Measurements of Shrinkage Induced Pressure (SIP) in Unsaturated Expansive Clays

A.J. Puppala, T. Wejrungsikul, V. Puljan[1] and T. Manosuthikij[2]
[1]*Department of Civil Engineering, The University of Texas at Arlington, Arlington, Texas, USA, 76019*
[2]*Petroleum Engineering and Geology Department, Petroleum Authority of Thailand Public company Limited (PTT), Thailand, 10900*
E-mail: anand@uta.edu

ABSTRACT: Unsaturated expansive soils are located in many regions of the world. Expansive soils can swell more than 100% and shrink more than 50% of its original volume. When these soil movements are totaled, they often result in significant distress to low overburden structures such as pavements and residential buildings. Cracking occurs when the shrinkage or desiccation induced pressure inside the expansive soil matrix exceeds the tensile strength of the same soils. In general, practitioners use soil tests such as linear shrinkage strain and Atterberg limit tests to determine shrinkage strain potentials of soils. However, these tests do not provide shrinkage induced soil pressures generated within the soil. The main objective of this paper is to present a new technique to measure the shrinkage pressure inherently induced inside the matrix of clays. This test termed as Shrinkage Induced Pressure (SIP) is evaluated for providing repeatable and reliable measurements. SIP test results are compared with Indirect Tensile (IDT) strength test results to explain the shrinkage mechanisms in the soils.

KEYWORDS: expansive clay, cracking, curling, shrinkage, shrinkage induced pressure (SIP), indirect tensile (IDT) strength

1. INTRODUCTION

Natural expansive soil is a clayey soil normally found in many places worldwide. These soils experience substantial volumetric changes due to moisture content changes from wet and dry seasonal periods. The 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 be around several billions of dollars annually (Nelson and Miller 1992).

Expansive soils owe their volume change 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. Soils with smectite clay minerals such as Montmorillonite exhibit the highest swelling properties (Manosuthikij, 2008). Swelling behaviour can be expected when the percentage of expansive clay minerals in soil exceeds 5 percent by weight (Manosuthikij, 2008).

Expansive clay soils can also be easily recognized in the dry season by the deep cracks that form in the ground surface as shown in the Figure 1a. The zone of seasonal moisture content fluctuations can extend to depths of 1 to 4.5 m (3 to 15 ft) which can result in wider cracks at the ground surface and this typically leads to the pavement or foundation distress as shown in Figure 1b. This creates cyclic swell and shrink related volume change behaviours in the upper portion of the soil column, and cracks can extend to greater depths than imagined by most engineers.

Many roads constructed on expansive clay subgrade especially in the east and central Texas, USA though designed with various chemically treated soil layers, still experienced severe pavement cracking with short serviceability life periods. The costs of maintenance, in some cases, are even more than their construction costs (Chen, 1988).

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, and their volume change potentials should be considered and accounted for in the design and construction of the roads.

a) Measurement of desiccation crack in expansive soil area

b) Pavement distresses from expansive soil movements

Figure 1 Desiccation and pavement distress (Manosuthikij, 2008)

Total or differential soil movements caused by swell and shrinkage strains of expansive soils could cause extensive destruction and damage to the highways (Chen, 1988). Contrasting or differential soil movements could produce large moments and shear forces in the pavement infrastructure, which can lead to cracking and eventual failure of both rigid and flexible pavements.

Majority of the soil movements in highway environment are connected to subgrade moisture content conditions, which are influenced by the seasonal changes, widening of the right of way, and surroundings of the large trees near to pavement systems. The latter condition will extract more moisture from the underneath pavement subgrades, resulting in severe shrinkage cracking problems in soils, thereby causing pavement distress. Other factors including lack of sufficient roadside ditches for drainage, and poor drainage problems around the pavements also 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).

In the present-day practice, expansive soils are mainly characterized based on swell characterization tests and models. Shrinkage tests are limitedly used in the current practice. However, it is well known that the shrinkage cracking of expansive soils during 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 experience shrinking. Hence, when characterizing expansive clayey subgrades, it is important to recognize and consider volumetric shrinkage strain potentials and shrinkage induced pressures of soils along with their swell properties. This paper describes shrinkage nature of expansive soils and methods used to advance the understanding of shrinkage strains and pressures in the soils. Native expansive soils from Texas, USA are primarily used for these measurements. Also, a few directions for design improvements for structures built on expansive soils are mentioned.

2. BACKGROUND ON SHRINKAGE CHARACTERISTICS

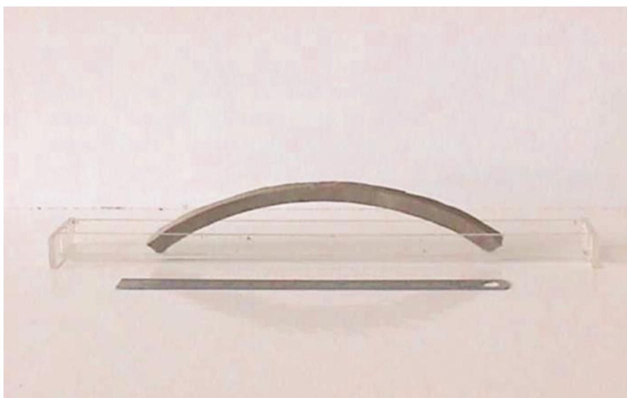
2.1 Shrinkage Cracking Mechanisms

Researchers and practitioners presently use linear shrinkage strain and Atterberg tests to measure and interpret shrinkage strain potential or cracking behavior of soils. These measurement methods are ineffective 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, a new test method using a cylindrical compacted soil specimen was introduced to drying to measure 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 unit weight conditions (Puppala et al., 2004).

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 2a. 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 of a soil layer is presented in Figure 2b, which is quite different from the irregular patterns that are shown in Figure 2a. 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) Soil curling at the edges (irregular pattern)



b) Curling of soft Werribee clay (ideal pattern) (Kodikara et al, 2004)

Figure 2 Soil curling patterns

Degree of curling is attributed to the differences of strain rates between top and bottom soil layers and also soil modulus of elasticity during shrinkage or drying process.

The shrinkage cracks would take place and lead to the curling deformation (stress relaxation) if an 'unsteady shrinkage induced stress' caused by the 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 cracking followed by soil hydration and swelling are considered to be major contributors to the distress of the light structures, e. g., pavements and residential structures.

2.2 Shrinkage Induced Pressure

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 during 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 and the structure thus leading to structural damage in the form of cracks.

Schematic of the shrinkage induced pressures and their influence on the slab foundation system are shown in the Figure 3. In this case, the lifted soil (contracting soil) is due to lateral shrinkage cracking which can be identified by the presence of cracks. Consequently, the shrinkage soils curl upwards and will attempt to lift the structures. Due to differential shrinkage movements from the outer region to interior core region, the infrastructure often experiences shrinkage cracks.

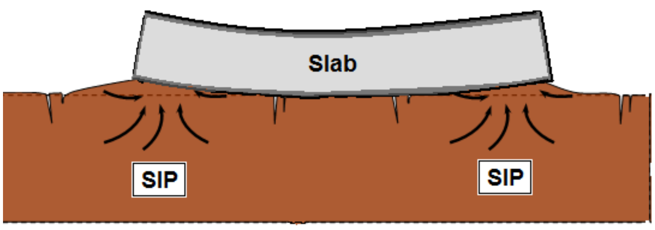


Figure 3 Schematic of SIP on a slab foundation

2.3 Shringake Modelling Studies

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) simulated laboratory shrinkage strain patterns using the finite difference modeling of this cracking behavior with a non-linear elasticity unsaturated soil model. Both shrinkage and curling behaviors to develop the shrinkage strains are explained in their research. These modeling methods advance our state of understanding of soil curling and shrinkage pressure developments. Yet the laboratory based state of the art work for measuring shrinkage pressure development in expansive soils is still being evolved due to complications of measuring this internal pressure.

New innovative sensors such as 'Force Sensing Resistors' or 'Force Sensors' (FS) can be embedded in the wet soil mixture and then they can be used to determine the shrinkage pressure while the wet soil mixture undergoes drying. This sensor 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. Assumptions are still needed for understanding and evaluating this sensor in the laboratory investigation. For example, the shrinkage pressure is assumed to be isotropic as this pressure can occur in any direction due to moisture depletion in a wet soil. In this paper, the use of this Force Sensor to measure shrinkage induced pressure in expansive soils is evaluated. This was considered as the main objective of this research.

3. EXPERIMENTAL PROGRAM

In the experimental program, two soil specimens from Fort Worth and Paris districts from Texas were used. These soils represent high plastic soils. Basic soil properties and three dimensional swell as well as shrinkage tests were performed and these properties are summarized in Table 1.

Table 1 Basic soil properties

Property	Soil Type	
	Fort Worth	**Paris**
Passing #40(%)	100	100
Passing #200(%)	85	81
Liquid Limit (LL, %)	61	60
Plastic Limit (PL, %)	24	23
Plastic Index (PI, %)	37	37
USCS Classification	CH	CH

For the three-dimensional or 3-D shrink and swell test, three soil samples from two sites were compacted at three different compaction moisture content conditions related to Optimum Moisture Content (OMC) condition. These three levels are Wet of OMC, OMC, and Dry of OMC with their corresponding dry unit weights as measured from a standard Proctor test on the soil. Volumetric shrinkage strain and swell tests were also measured to study the swell and shrinkage behaviour of the soil. Test results are expressed in percent values in terms of original volumetric strain. Initial volume in radial and vertical directions was first recorded and these results are used to determine volumetric shrinkage and swell strains which consist of vertical, radial, and volumetric strain parameters. Table 2 presents the average shrinkage and swell strain of the test results from a three-dimensional free swell test.

Table 2 Volumetric swell and shrinkage strain test results

Moisture Condition	Parameter	Shrinkage Strain (%)		Swell Strain (%)	
		Fort Worth	**Paris**	**Fort Worth**	**Paris**
Wet of OMC	Vertical	8.43	8.78	3.63	1.43
	Radial	8.87	9.45	1.95	1.51
	Volumetric	23.59	24.66	7.71	7.50
OMC	Vertical	5.29	4.92	9.28	7.37
	Radial	2.47	4.91	3.46	3.60
	Volumetric	12.51	14.04	16.97	15.25
Dry of OMC	Vertical	2.17	2.41	14.13	14.35
	Radial	0.97	1.46	4.26	5.36
	Volumetric	5.22	6.15	24.07	26.93

From the results, Paris soil provided the higher values of volumetric shrinkage and swell strain, which is considered to have high shrinkage strain behavior. Soil from Fort Worth showed the lower shrinkage and swell strain potentials.

3.1 SIP Test

Shrinkage Induced Pressure (SIP) test devolves around the direct measurement of the shrinkage force of curling soil specimen during the drying process when the soil is restrained by a rigid container. A few assumptions are needed in this test. For example, the 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. Typically the sensor is attached to rigid walls of the test setup. One of the problems is to know exact points where curling of the soil will occur and how they will be in contact with the container walls. The best SIP measurements can be achieved when the sensor is in direct contact with the curling soil medium. This research has focused on this aspect by investigating various sensor configurations in the test setup.

The sensor for detecting force inside specimen is called Force Sensor (FS) used in the study. 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 9.5 mm (0.375 in.) diameter circle at the end of the sensor. The sensors are made of two layers of substrate. This substrate is composed of polyester 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.

Figure 4 Force Sensor model A201 (Tekscan, 2010)

This test was a newly developed one and hence it is necessary to ascertain the quality of test result by studying the repeatability of test results. This section describes the present of Shrinkage Induced Pressure by using horizontal orientation (SIPH) results for soil in liquid limit + 10% (LL + 10%) condition. For the container used in this test, 76 mm (3.0 in.) diameter moisture can with a 20 mm (0.8 in.) slit on the side was made for the placement of Force Sensor (in horizontal orientation) as shown in Figure 5.

3.2 Test Procedure

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. Approximately 250 g 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%. The prepared soil mixture was placed into the moisture can and the Force Sensor was inserted in a horizontal position as presented in the Figure 6.

a) Side view of SIP test setup

b) Top view of SIP test setup

Figure 5 Moisture can with a horizontal slot for FS

Figure 6 SIP test in horizontal orientation (SIPH)

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. 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 400C in order to simulate slow drying process. Prepare and place the soil specimens at each moisture contents in the oven chamber. Each soil specimen (include container) shall be weighed every 4 - 6 hours so that the moisture changes with time can be established. The schematic of the test setup used for horizontal orientation SIPH tests is presented in Figure 7.

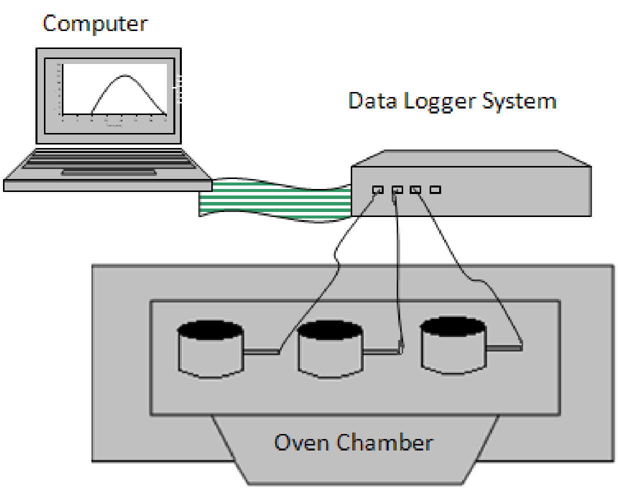


Figure 7 Schematic of the Test Setup

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 8 shows shrunk soil specimens from SIPH tests.

a) Fort Worth clay specimen

b) Paris specimen

Figure 8 Fort Worth/Paris soil specimens after drying

3.3 Indirect Tensile (IDT) Strength Test

Indirect tensile (IDT) strength test was also conducted in this research to measure the tensile strength of soil or rock specimens and the photograph of the test setup is presented in the Figure 9. 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 application (ASTM D3967- 95a, 2001). The indirect tensile (IDT) strength or splitting tensile strength test is hence chosen. The IDT strength is a representative of the force that breaks the physical bond between soil particles, while the shrinkage induced pressure measures the internal force that physical bonds are developed during drying process. Hence, due to their similarities, an attempt is made here to compare the tensile strengths of soils with shrinkage induced pressure results at the same conditions.

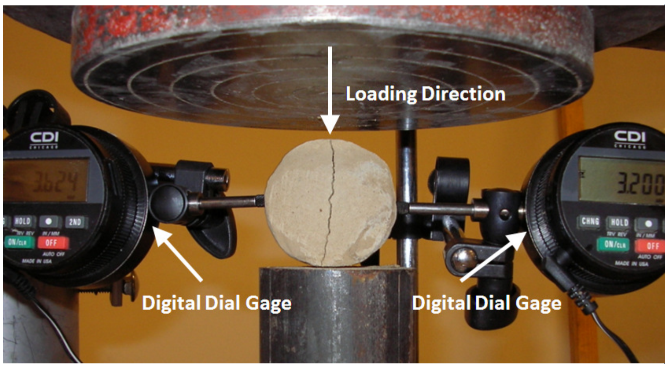


Figure 9 Indirect tensile (IDT) strength testing on dry soil specimen

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 expansive soils. In addition, the specimens used in this test must have no cracks and have a ratio of thickness to diameter (t/D) between 0.2 – 0.75 as per ASTM D3967.

Prior to the IDT test being performed, the sides of specimen were ensured to have smooth surfaces. Therefore, trimming the side of specimens is needed for unsmooth surfaces or edges. The dry specimens used for this test are shown in the Figure 10. Tests were conducted to determine the load (P) that induced a cracking along the vertical lining as shown in the figure.

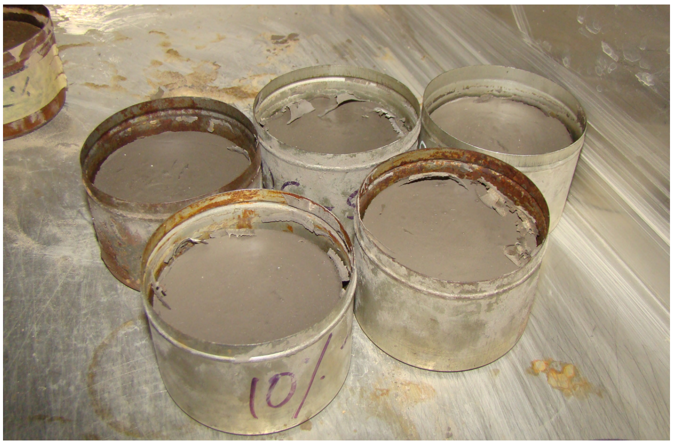


Figure 10 Dry soil specimens used in IDT testing

4. ANALYSIS OF TEST RESULTS

4.1 SIP and IDT Results and Discussion

All the individual shrinkage pressure test data of Fort Worth and Paris soils are shown in Figures 11 and 12. In one set of the results, shrinkage induced pressure results versus time and the change of water content versus time are presented. These results show very good repeatability of the SIP data versus time profiles. 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 superior and is able to capture SIP values during drying tests. The summary of the test results is presented in Table 3.

Table 3 Summary of shrinkage pressure test results

Moisture Condition	Type of Specimen	Average of IDT Strength, kPa (psi)	Standard Deviation, kPa (psi)
LL+10%	Fort Worth	290.54 (42.14)	43.57 (6.32)
	Paris	498.06 (72.24)	45.85 (6.65)

Paris soil has the higher shrinkage pressure which is 498 kPa (72.2 psi) and Fort Worth soil has the shrinkage pressures of 300 kPa (42.0 psi). From the standard deviation of both tests, it shows that 90% of the data lie within the 'Mean + 1 Standard Deviation' range, which can be used to infer that the SIPH test has provided repeatable measurements. Indirect tensile (IDT) test results that describe the tensile strengths of various soil types at different moisture conditions were measured and these results are presented in Table 4. As shown in the Table 4, the IDT results indicate that Fort Worth and Paris site soils yielded IDT values of 343.2 and 512.3 kPa (49.8 and 74.3 psi), respectively.

Table 4 Indirect tensile (IDT) strength test results

Moisture Condition	Type of Specimen	Average of IDT Strength, kPa (psi)	Standard Deviation, kPa (psi)
LL+10%	Fort Worth	343.22 (49.78)	34.34 (4.98)
	Paris	512.25 (74.31)	126.5 (18.34)

4.2 Correlations between SIP and IDT Test Results

The shrinkage induced pressure is a direct method to determine the pressures from shrinkage conditions in soils. In the past, the indirect tensile (IDT) 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 and these results were compared with SIP results as shown in Table 5. An attempt is also made to develop correlations between SIP and IDT values.

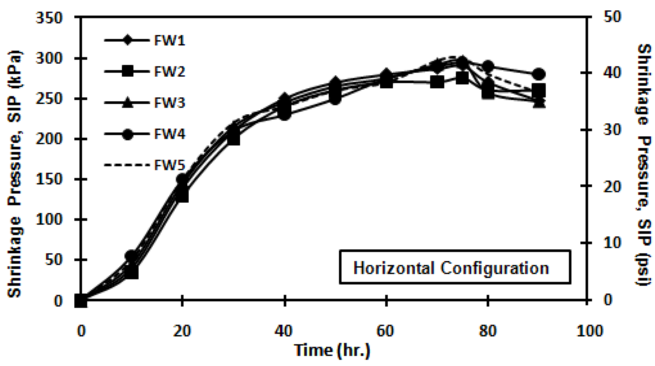
Table 5 Comparisons of the results of shrinkage induced pressure (SIP) tests and indirect tensile (IDT) strength tests

Moisture Condition	Soil Type	Average of Maximum SIP, kPa (psi)	Average of IDT value at Final Condition, kPa (psi)	(SIP/ IDT) x 100 (%)
LL+10%	Fort Worth	290.54 (42.14)	343.22 (49.78)	84.65
	Paris	498.06 (72.24)	512.25 (74.31)	97.21

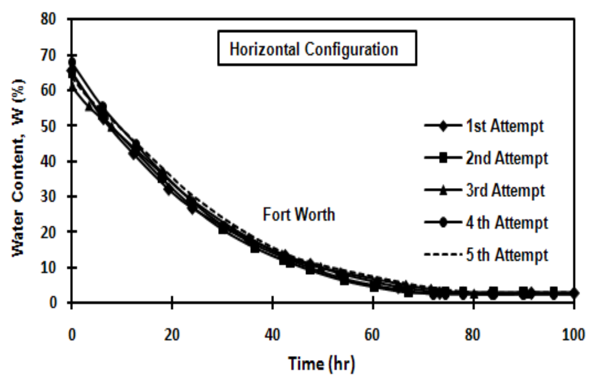
The comparison results show that the percentage of the ratios between SIP and IDT values is close to 100%, which means the actual SIP value is lower than the indirect tensile (IDT) strength of a soil. Also, SIP values provide direct shrinkage pressure measurements from volumetric shrinkage strain conditions and hence more appropriate for design of civil structures on expansive soils that undergo inherent volume change fluctuations from seasonal changes.

4.3 Design Recommendations

Conventional designs do not include the effects of shrinkage induced pressure (SIP) into the infrastructure design due to unavailability of technologies to properly measure the shrinkage pressures. In this research, it is well demonstrated to use Force Sensors to determine SIP values of expansive soils from large moisture content conditions. Thus, it is necessary to include the SIP values into the design practices and then address the flexural related design issues due to differential shrinkage movements occurring underneath the infrastructure. Several examples of various infrastructure designs using SIP values are presented and these include slab-on-grade foundations supporting residential and industrial buildings, pavement infrastructure, and retaining walls with expansive clay backfills. These results are shown in Figures 13a, b, and c.

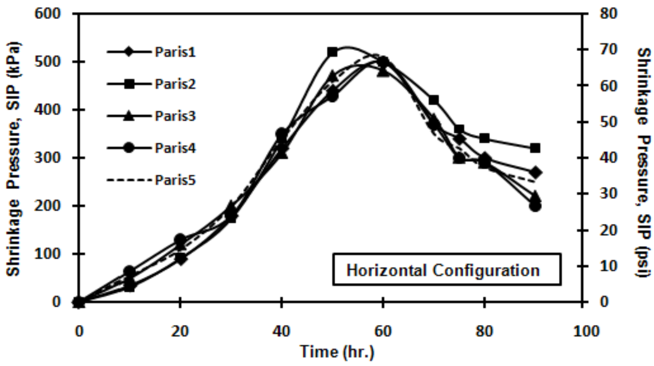


a) Time (hr) versus SIP

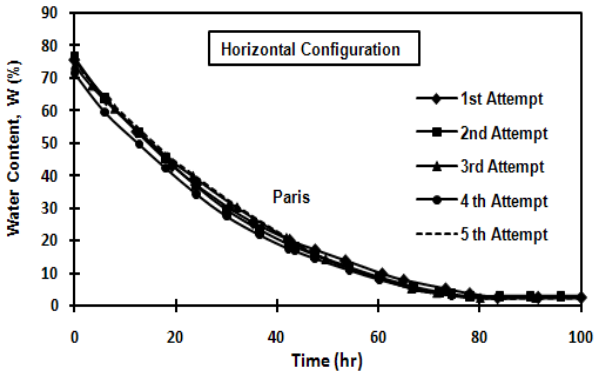


b) Time (hr) versus Water Content (%)

Figure 11 Shrinkage induced pressure test results of the Fort Worth soil at LL+10% condition

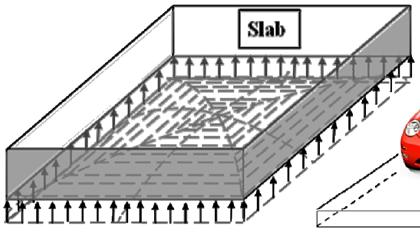


a) Time (hr) versus SIP

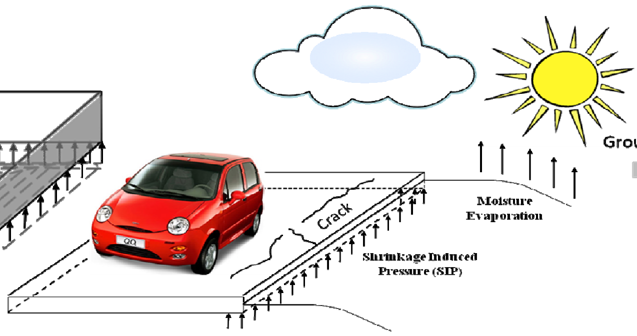


b) Time (hr) versus water content (%)

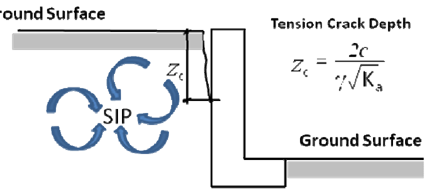
Figure 12 Shrinkage induced pressure test results of the Paris soil at LL+10% condition



a) SIP distributions in slab



b) SIP to induce distress in the pavement



c) Development of tension crack in retaining walls used with expansive clay fills

Figure 13 Examples of design recommendations

In the case of slab foundations, the large SIP values may be developed around the edges of the slab and low values or even zero values will occur at the middle of slab. Due to differential SIP values underneath the foundations, flexural moments will be generated and they should be accounted for the structural design of the slab.

In the case of pavements, the SIPs will be developed along the unpaved shoulders which are exposed to moisture variations and temperature changes. When the moisture contents in road shoulders evaporated, the soils will shrink and will induce SIP values in the adjacent pavements. SIP values along the pavements can result in both longitudinal and transverse cracking.

In the case of retaining walls, the tension crack depth has been included in the geotechnical design using the traditional Mohr Coulomb model. However, in reality, based on the clay mineralogy, swell and shrink behaviors vary and this will induce forces that needed to be used in the structural design of walls in the upper parts of the wall. It should be noted that though specifications require the use of granular fills, it is common practice to use the local mixed clayey soils with expansive properties for backfills. Most designs do not consider the expansive soil induced pressures.

Overall, the present SIP measurements of the field soils will there offers an opportunity to refine the traditional geotechnical and structural design of civil infrastructure built on or/and in expansive soils.

5. CONCLUSIONS

The following summarizes the major conclusions from the present research:

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

2) 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 at LL+10%. It should be noted that IDT test were performed on the soil samples prepared at LL+10% after oven drying. 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.

3) From the SIP results conducted on two soils at LL+10% condition at horizontal FS orientation show that the Paris soil has exhibited higher SIP value of 498 kPa (72 psi) and Fort Worth has SIP value of 300 kPa (42 psi), which is corresponding the degree of shrinkage and swell of the soil preformed in the laboratory section.

5. ACKNOWLEDGEMENTS

Authors would like to acknowledge the Department of Civil Engineering of University of Texas at Arlington for providing teaching assistantship to Thornchaya Wejrungsikul. We also acknowledge various funding agencies including Texas Department of Transportation.

7. REFERENCES

ASTM D 3967-95a, Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens.

Brown, S.F. (1996). “Soil mechanics in pavement engineering.” Geotechnique, v 46, n 3, Sep, 1996, p 383-426.

Chen, F.H (1988). Foundations on expansive soils 2nd Ed., Elsevier Science Publications, New York.

Chakrabarti, S. K. and Kodikara J. (2006). Modelling of curling due to drying shrinkage in Cement stabilised crushed rock, Construction Materials, Proc. the Institution of Civil Engineers, vol. 159, CM2, 65-75.

Fredlund, D.G. and Rahardjo (1993). Soil Mechanics for Unsaturated Soils, John Wiley & Sons, Inc., New York.

Jones, D.E., and Holtz, W. J. (1973). “Expansive soils: The hidden disaster.” Civ. Eng. (N.Y.) 43(8), 49–51.

Kodikara, J.K., Nahlawi, H., and Bouazza, A. 2004. Modelling of curling in desiccating clay. Canadian Geotechnical Journal, 41(3): 560–566. doi:10.1139/t04-01

Manosuthikij, T. (2008). ”Studies on Volume Change Movements in High PI Clays for Better Design of Low Volume Pavements” Doctor of Philosophy Dissertation.

Nelson, D. and Miller, D. J. (1992). Expansive Soils Problems and practice in Foundation and Pavement Engineering, John Wiley & Sons, New York, 1992., 40- 80.

Poor, A.R. (1974). ‘‘Experimental residential foundation design on expansive clay soils.’’ Rep. No. TR-3-78, Final Rep., Construction Research Center, The Univ. of Texas at Arlington, Texas.

Puppala, A.J., Griffin, J.A., Hoyos, L.R. and Chomtid, S. (2004a). “Studies on Sulfate-Resistant Cement Stabilization Methods to Address Sulfate-Induced Soil Heave.” Journal of Geotechnical and Geoenvironmental Engineering, 130, pp.391- 402.

Puppala, A.J., Katha, B., Hoyos, L.R. (2004). “Volumetric Shrinkage Strain Measurements in Expansive Soils Using Digital Imaging Technology.” ASTM Geotechnical Testing Journal, Vol. 27, No. 6, November, 2004.

Tekscan Inc. homepage. FlexiForce® Force Sensors. http:// www. tekscan.com /flexiforce /flexiforce. html. Accessed September 3, 2010.

Wray, W.K.; Ellepola, C.B. (1994). “Stresses Developed by Laterally Shrinking High-PI Clay.” Proc. of 8th International Conference on Computer Methods in Advanced Geomechanics, vol. 4, p 1515-1526.