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# Effect of plastic and silty fines on the shear behavior and pore water pressure generation in sands

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**Effect of plastic and silty fines on the shear behavior and pore**

**water pressure generation in sands**

by

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The student author, whose presentation of the scholarship herein was approved by the program of study committee, is solely responsible for the content of this thesis. The Graduate College will ensure this thesis is globally accessible and will not permit alterations after a degree is conferred.

Iowa State University

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2020

## DEDICATION

This thesis is dedicated to

my family members, friends and well-wishers.

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## NOMENCLATURE

| | |
|---|---|
| psi | Pounds per square inch |
| kPa | Kilo Pascal |
| eg | Granular void ratio |
| Rd % | Relative density |
| esk | Skeletal void ratio |
| PWP | Porewater pressure |
| p’ | Mean effective stress |
| q | Deviator stress |
| fc | Fines content |
| FCth | Threshold fines content |
| $\varphi$ | Angle of internal friction |
| $\tau f$ | Shear stress |
| c’ | Cohesion |
| $\sigma' f$ | Effective stress |
| PL | Plastic fines |
| NP | Non-plastic fines |
| LL | Liquid limit |
| PI | Plasticity index |
| $\sigma 3$ | Confining stress |
| w% | Water content |

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## ABSTRACT

The purpose of this study was to investigate the influence of plastic and silty fines on the shear strength and the porewater pressure (PWP) response of Ottawa Sands. The sands have been mixed with both plastic and silty soils at different percentages. The fines were added ranging from 5% up to a threshold value by weight for constant dry unit weight and sheared in consolidated undrained triaxial compression (CU) test at a constant small strain rate. The advantages of using skeletal void ratio in cases where fines are present within pure sand samples are also discussed.

The properties of plastic fines such as the plasticity of the fines, the percentage of fines content, and the clay mineralogy are studied to understand the influence on the undrained behavior of sands. In order to have a better understanding of the generation and dissipation of PWP during undrained shearing in sandy and sandy soils with plastic and non-plastic fines, a novel PWP measurement system was designed to measure PWP at the center of a triaxial specimen. Factors influencing the development of PWP such as strain rates, the relative density of the sands and the percentage of fines content along with their plasticity index were investigated. Results from the study indicated that the effect the fines on the soil response varies on the amount and type of fines and also on the initial relative density of the host sand.

# CHAPTER 1. INTRODUCTION

The study of engineering properties of soil has always revolved around research conducted on two distinct types of materials namely pure clays and pure sands. These two materials are on opposite ends of the spectrum when discussing the very basic fabric of soil. Soils are very different in their properties related to particle size, hydraulic conductivity and hence also differ in behavior when sheared. The pore water pressure generation and dissipation behavior within the soils are also very different during shearing and hence should be analyzed separately.

Most of the studies in the literature focus on typically pure clay or pure sand. However, soil mixtures such as clayey sand, silty sand, etc. are more naturally occurring than pure sands or silts or clays. Previous research has shown that the presence of the fine particles of silt or clay within the sands affects the behavior of sand due to their small size and smaller void spaces (Carraro, Prezzi, and Salgado, 2009; Maleki et al., 2011; Murthy et al. 2007; Thevanayagam, Ravishankar, and Mohan, 1997). In sand mixtures, the change in the internal void ratio due to the presence of different materials of different sizes are influential in affecting the generation of pore water pressure. The behavior of the pore water is especially important in order to understand the behavior of the soils under various loading conditions. The issue of initiation and generation of excess pore-water pressure is critical in understanding the prediction of liquefaction, slope stability, the behavior of foundations, the stability of embankments, etc. in sands and silty or clayey sands.

Previous studies paint a complex picture of nature the fines play in the behavior of soils leading to liquefaction Zlatovic and Ishihara, (2013). With regard to the effect of plastic fines, there is a general assumption that the presence of plastic fines should increase the stability and resistance to liquefaction Bayat et al. (2014). Numerous studies have also reported that the effect of the plastic fine on the behavior of the soil depends mainly on the plasticity index and the number of fines (Ishihara, 1993). Nevertheless, there have been observations made at various earthquake sites where the ground has undergone liquefaction phenomenon and plastic fines have been observed within the sand present Ishihara (1993). Therefore, it can be shown that the behavior of sands in the presence of fines is more complex than previously thought.

### 1.1. Need for Testing

During moderate or strong earthquakes, extensive damage can occur to infrastructure such as earth dams, embankments, ports, building foundations, etc. due to the liquefaction phenomenon. A significant amount of research has been done in the past on the effects of plastic fines and silts on the behavior of the shear strength and liquefaction resistance in addition to sands Carraro, Prezzi, and Salgado (2009). It has been understood in the 1960s that the presence of non-plastic and plastic fines in some manner resisted the liquefaction of sands Polito, (1999). However, numerous case histories concerning failure due to liquefaction-induced by earthquakes were observed where the soils contained plastic fines present within its sand deposits Carraro, Prezzi, and Salgado, (2009). The Mino-owar (Yang, 2015) in Japan consisted of soils with 10% fines Kishida (1969), while it has also been reported that sands with clay content have been seen to liquefy at many instances during earthquakes Kishida (1969). Many of the results acquired after research carried out in this area have provided conflicting results especially in terms of soil behavior with respect to their potential to liquefy. This has led to questioning the basic assumptions we had regarding the behavior of soils with fines.

The role of fines especially on the behavior of sands and resistance to liquefaction and static liquefaction has been a topic of debate for many years. The reason for the debate has been regarding what factors affect the behavior of soils and how they affect it. The nature of the sand behavior changes in the addition of fines. However, in the case of plastic fines, more parameters than non-plastic fines, such as plasticity of fines, fines content, clay mineralogy, and pore water chemistry, may influence the undrained behavior of sands. One of the most important effects of fines on the undrained behavior of sands is their effects on the instability of sands. The fines can have an important effect on the fabric of the soil which can influence the resulting stress-strain response. To evaluate the potential strain softening/ hardening response, and its effect on the collapsibility of sands, it is useful to study the effects of fines content on sands, variation in plasticity and gradation on the behavior of soil samples.

Liquefaction is a natural phenomenon that takes place in saturated sands due to the increase of excess pore water pressure to a particular point where it equals the normal stress causing the effective stress to become zero. During research and engineering site investigations, liquefaction analysis is carried out using results from undrained laboratory shear tests. Liquefactions occur when for any given loading condition, the excess pore water pressure equals the effective stress. Many natural soils contain a significant amount of fines and may lead to instability phenomena to occur due to the presence of these fines. Saturated soils lose some of their shearing strength during cyclic loading as well Carraro (1999). But it is important to know its behavior under monotonic loading first before dealing with a more complex phenomenon of cyclic loading. Therefore, an investigation of the effects of different fines, their characteristics on the generation of pore water pressure is necessary in order to correctly analyze its effects on the behavior of soils under shear stress.

### 1.2. Objectives of the Present Investigation

The purpose of the present investigation was to observe the behavior of mechanical strength and behavior of pore water pressure of sand samples with different proportions of fines with different characteristics. Different types of soil such as silty sands, clayey sands, and pure Ottawa sands were tested in this study. The soils were characterized based on particle size analysis, Attenberg’s limits, hydrometer, etc. The main objectives of this research were to:

1. Measure the static undrained response of sands at various void ratios;
2. Experimentally study the behavior of pore water pressure within the soil simple by applying pore water pressure transducers in the middle of the length of the sample and investigate the factors affecting PWP generation;
3. Analyze the pore water pressure behavior with the presence of plastic, non-plastic fines and in pure sands;
4. Analyze the transition of soils from stable to unstable during monotonic loading based on the type of fines added to the sand specimens;
5. Analyze the transition of soil stability due to the presence of fines; and
6. Create a framework to describe the patterns of the presence of fines in the sand.

# CHAPTER 2. LITERATURE REVIEW

## 2.1 Introduction

Unlike other construction materials used like steel, concrete, etc. soil behaves differently. It does not have a unique shear strength associated with it and depends upon boundary conditions, external factors, stress history, etc. But the mechanical behavior of the soil is an important aspect that needs to be studied in order to correctly understand their behavior on the application of stresses and a triaxial test has been used for a long time in determining the various variables associated with it.

The mechanical behavior of clean sands was first observed by Coulomb (1776) where he worked on problems related to earth pressures against retaining walls. The most commonly accepted theory of shear strength is one of the major contributions to geotechnical engineering. Terzaghi (1925) almost 150 years later first presented the first textbook on soil mechanics building his work on works done previously by engineers namely Collin, Darcy, and Rankine. These studies continued over the years and gave us the definitions that led to the analysis of engineering behavior wide range of spectrum from pure clays to pure sands. These soils differ in properties due to the basic fundamental structure and hence behave differently. They have opposite extremes of hydraulic conductivity and particle sizes; sands being more permeable as compared to clays and having bigger grain sizes as well. These hydraulic conductivities affected other properties of soil such as consolidation and generation and dissipation of pore water pressure during the presence of a load Carraro, Prezzi, and Salgado, (2009); Holtz and Kovacs (1981).

**2.2. Shear Strength of Soils**

If the load on the soil increases above a certain limit so that the deformations become more than the accepted value, the soil is assumed to have failed in such a case. In this case, the strength of the soil is measured as the maximum or ultimate stress a soil can sustain. In the case of soils, failure leads due to excessive applied shear stress. The shear strength of soils is generally defined as the limiting resistance or ultimate resistance offered by the material during the loading and unloading. The shear stress in the soil can be determined in two ways, by in-situ soil tests like cone penetrometer tests or vane shear test or by carrying lab tests on soil samples. In the in-situ tests, the results obtained are indirect, i.e. mostly from correlations with the tests done in the laboratory or via back calculations from actual failures. Laboratory tests are done with fair accuracy although the sample runs a high risk of being disturbed during extraction.

During the turn of the $20^{th}$ century, Mohr hypothesized a criterion for failure plane in a material by a critical combination of normal stress and shear stress. The relation between normal stress and shear stress and the failure plane can be given by:

$$\tau_f = f(\sigma_f) \tag{1}$$

where $\tau_f$ is the shear stress at failure and $\sigma_f$ is the normal stress on the failure plane. The failure defined envelope defined by the failure plane is shown in Figure 1.

The component "c" of the shear stress which is equal to the intercept on the X-axis is called apparent cohesion and the angle which it makes with the Mohr circle is determined with Φ or the angle of internal friction. As shown in Figure 2, if a normal and shear stresses occur on point on a plane in a soil mass, a shear failure will not occur on that plane (Point A). If the combination of normal and shear stresses occurs at Point B then shear failure will occur

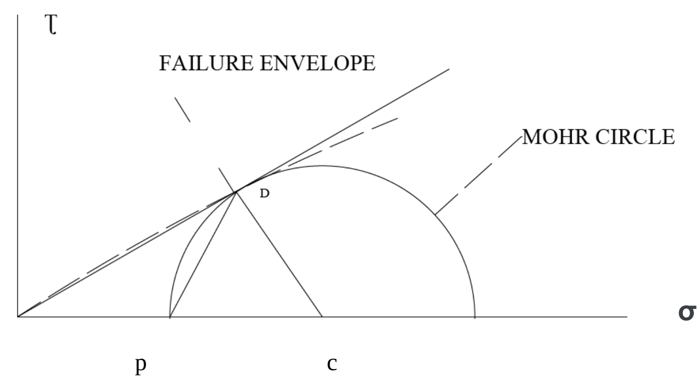



**Figure 1. Mohr's circle and failure envelope**

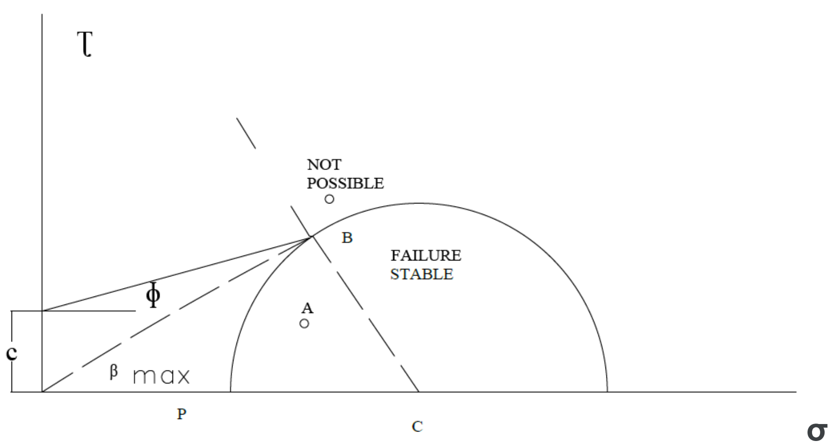



**Figure 2. Position of stress on Mohr's circle**

along that plane. A state of stress cannot occur on Point C since it lies outside the failure plane which means that the state of failure has already occurred before that condition was reached. Equation 2 defines the shear stress of failure.

$$\tau_f = c' + (\sigma'_f) \tan \Phi' \qquad (2)$$

where:

$\tau_f$ = shear stress at failure plane
c = cohesion intercept in terms of total stress
c' = cohesion intercept in terms of effective stress
$\sigma_f$ = total normal stress at failure

$\sigma'_{f}$ = total effective stress at failure
$\Phi$ = friction angle in terms of total stress

Effective stress parameters between soil particles are accepted as the basic factor influencing strength. The total stresses and parameters related to it are generally considered where drained conditions on shear strength are more critical than that of undrained conditions such as stability of slope problems. Total stress parameters are used where undrained conditions are more critical in shear strength such as bearing capacity problems in shallow foundations Ölmez, (2008).

### 2.2.1. Shear strength of cohesionless soils

The shear strength of a cohesionless soil maybe is represented by Equation 3. This equation is a modification of Equation 2 which is a special case as c= 0.

$$\tau_f = (\sigma_f) \tan \Phi \quad (2)$$

The shear stress of cohesionless soils depends on the angle of internal friction, which is caused by the interlocking of coarse, angular grains. The friction angle increases with the increase of relative density and decreasing the void ratio. It also depends on the gradation of soil particles where well-graded sand has a higher friction angle than poor, or gap graded as it leads to a better interlocking of grains. The shear strength of the soil is affected by the presence of pore water pressure within the samples and hence the shear strength of saturated cohesionless soil is given as:

$$\tau_f = (\sigma_f - u) \tan \Phi \quad (3)$$

where u = pore water pressure.

When the pore water pressure approaches the value of the total stress within the soil, the shear strength reaches zero. This is when the soil is said to have achieved instability and might undergo failure at slopes or create a sand boil phenomenon. The shear strength of cohesionless soils is conventionally measured using a direct shear test and triaxial test.

**2.3. Undrained Triaxial Compression Test (CU)**

The triaxial test device is one of the most commonly used tests to measure shear stress and pore water pressure within a soil sample Figure 3. It is much more versatile than any other test in controlling drainage, applying principle stresses and controlling stress paths for the test. In such a test, a cylindrical soil specimen is subjected to an axial compression stress sigma 1 ($\sigma_1$) and radial confining stress sigma 3 ($\sigma_3$). A triaxial test is a relatively simple test used for measuring numerous properties of soil. The success of the triaxial device lies in the fact that it allows the user to vary the drainage while running a test as it facilitates in running both a drained and an undrained test. Drainage is provided to the soil specimen as water flows from bottom to top under the desired pressure. In a drained test, volume change in the soil can be measured whereas in an undrained test, the change in pore water pressure is calculated. In cases of soils, just by controlling the drainage values the volume change is prevented and the soil is sheared undrained. In the case of drained tests, the volume change in the soil occurs due to the applied pressure sigma 1. In the case of sands, when loaded statically the sand drains instantaneously because they have higher permeability as compared to silts and clay. Unlike a direct shear test, one can measure the pore water pressure within the sample while running an undrained test in a triaxial test with much ease. Also, unlike a direct shear test, one can apply principal stresses in known direction using air or water or oil in a triaxial cell. The soil is placed in a rubber membrane placed in a cylindrical chamber. In most cases, water is used to apply desired confining stress. The membrane is used to prevent the water from entering the soil specimen and making sure that the principal stresses are

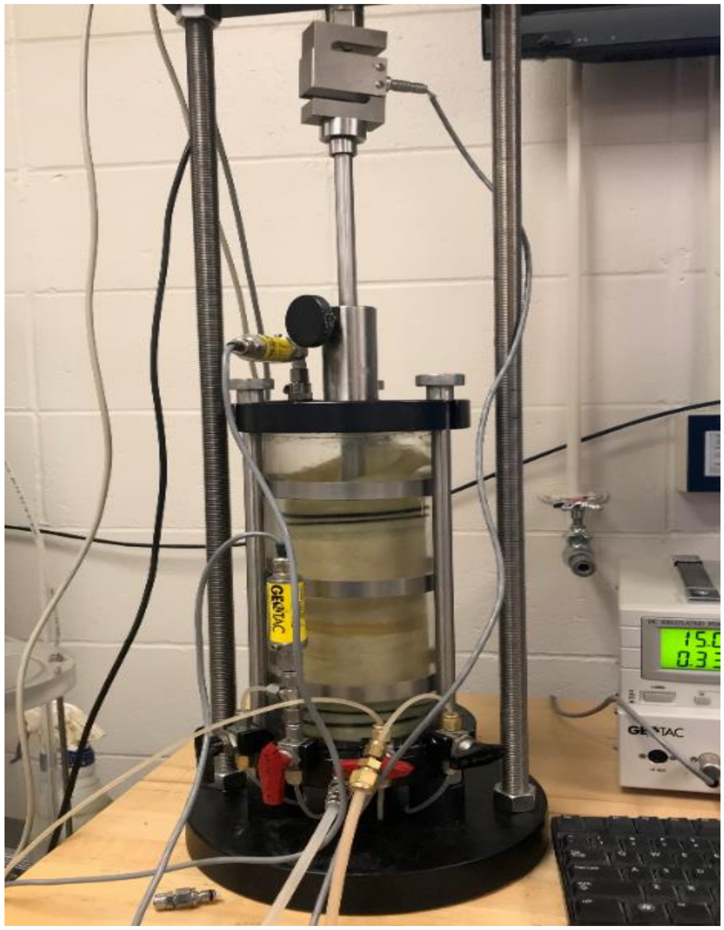


**Figure 3. Monotonic Triaxial Testing Chamber**

separately applied to the specimen. A conventional triaxial testing system is used in this research to measure the various properties of the soil. Remolded and undisturbed specimens can be used for both testing of both cohesive and cohesionless soils (Baldi, Hight, and Thomas, 1988; Holtz, 1981).

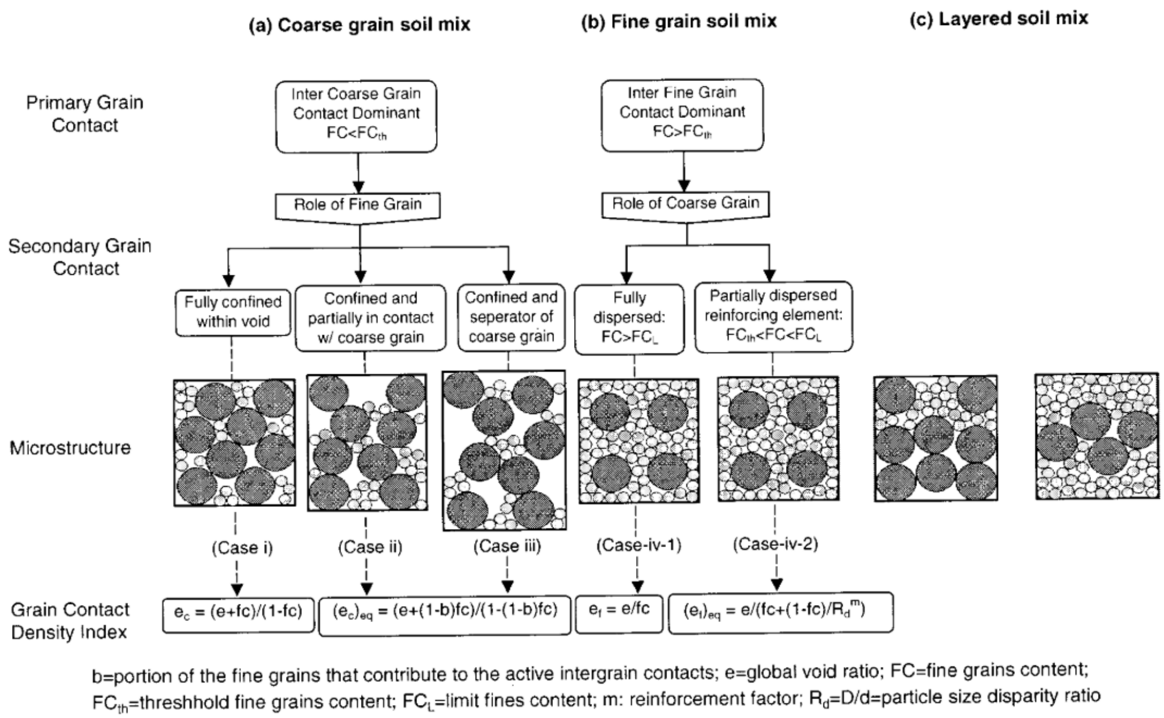
Conventionally, the pore water pressure within a sample in a triaxial test is only measured at the two ends of the sample. However, that does not give us a complete picture of the behavior of soil, due to the development of strains and pore water pressure at various points within the sample. Additional pore water pressure measurements distributed throughout the height of the specimen would allow for additional understanding of the generation and dissipation of pore water during loading. The results of the tests carried out by in this thesis provide insight into the behavior of PWP during a monotonic triaxial test at various points along the length of a sample to provide a more accurate understanding of silty and clayey sands during shearing. The testing program described in this research utilizes triaxial tests to observe the difference in the behavior of Ottawa sand with the presence of Kaolinite clay and loess fines.

**2.4. Microstructure of the Soil Fabric**

"Fines content" refers to the particles with grain size less than 0.075 mm which is made of the same or different parent rock. The microstructure of the sand and fines with size D and d of the particle mixes can be arranged in three extremely limiting categories. The different arrangement in the structure lead to different intergrain contact and may lead to a different stress-strain response. According to the intergrain state concept, a transitional/ threshold/limiting fines content $FC_{th}$ exists: sand-dominated behavior passes to fines-dominated behavior when the fines content is beyond the $FC_{th}$.

The 3 extreme categories by Thevanagayam et al. (2002) are displayed in Figure 4. In the case of (a) ($FC < FC_{th}$) the fines present within the system are divided into 3 parts depending upon the amount of fines present.

- Case (i): The soil is mostly coarse-grained with fines confined within the voids between them. These fines have little or no contribution towards supporting the soil skeleton.
- Case (ii): As the fines content increases within the sand specimen, the fines within the voids increase and they start partially supporting the coarse-grained skeleton.




Key: (a) The coarse grains are in contact with fines present in between them, case (i-iii)
(b) Primarily the fines are in contact, case (iv)
(c) Layered system.

Source: Thevanayagam et al. (2002)

**Figure 4. Variation in the arrangement of the microstructure in a sand and fines mix**

- Case (iii): As the fines content increases further, they start separating the coarse grains and play a considerable role in the stress distribution within the soil skeleton. The fines reach a threshold limit at the case (iii).
- Case b: ($FC > FC_{th}$) The coarse-grained soils are fully dispersed in fines.
- Case c: The soils are layered as fines and coarse-grained one below the other.
- $FC_{th}$: The same framework used to describe the small-strain stiffness and the shear strength of clean sands may be used for silty sands, provided that the fines

content remains below some limit also called threshold fines content (Salgado, Bandini, and Karim, 2001).

As the amount of fines in the coarse grain matrix increases, there comes a point where fines start to dominate the soil behavior. This was further explained in detail by Thevanayagam et al. (2002) and Zlatovic and Ishihara (2013), where they explained the intergranular void ratio characterized by the stress-strain response of sand and silty/clayey sand mixes.

**2.4. Intergrain State Concept**

The intergrain state concept was proposed by Thevanayagam (1998), where the author defined the intergranular void ratio in order to define the state of sand instead of the global void ratio. It was later redefined as a skeletal void ratio by several researchers, such as Salgado, Bandini, and Karim (2001). For soils with a low fines content or $FC < FC_{th}$ the skeletal void ratio is defined as:

$$e_{sk} = \frac{e+fc}{1-fc} \tag{5}$$

where fc is the % of fines content in decimals.

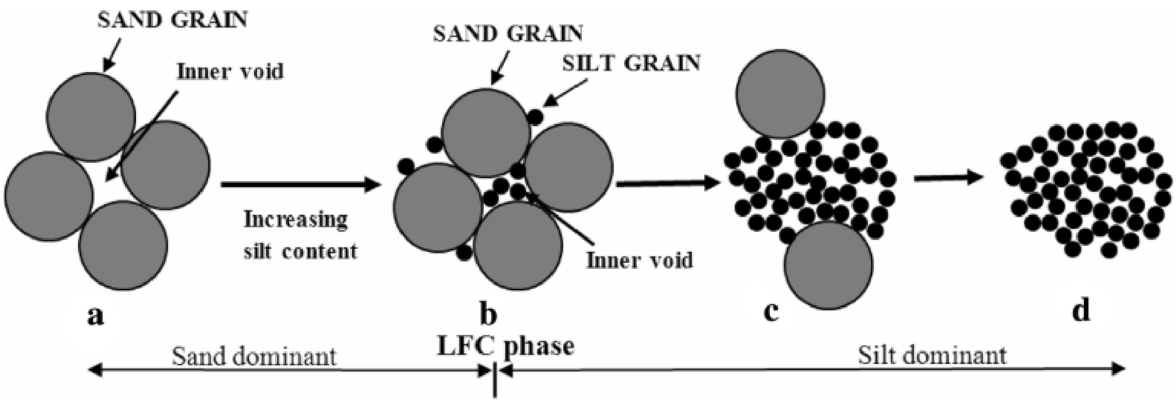
For sands with high fines content where $FC > FC_{th}$, the skeletal void ratio is defined as

$$e_{sk} = \frac{e}{fc}\,. \tag{6}$$

Thevanayagam et al (2002) proposed that, at low fines content, the sand particles are in contact with each other and majorly constitute the soil skeleton while the fines only make a secondary contribution. At high fines content, the sand particles only make a secondary contribution to the soil behavior whereas fines majorly contribute towards the stress distribution. In other words, in the intergrain state concept, the fines are defined as voids and are assumed to participate partially or not at all in the resistance to shearing whereas when the fines are high, then the coarse grains are assumed to be voids and are hence considered to participate only partially to the void ratio.

For the investigation of the effects of the addition of fines on the behavior of sand, many different approaches have been carried out by many researchers. Kubis et al. (1998), and Pitman, Robertson, and Sego (1994a) used a constant void ratio of the host sand, whereas Georgiannou, Burland, and Hight (1990) and Thevanayagam, Ravishankar, and Mohan (1997) used the concept of granular void ratio. Therefore, it can be seen after the research on sands and on sands containing fines that for proper evaluation of experimental results and, in order to correctly determine the engineering properties of the sand with fines matrix, a proper parameter needs to be determined to quantify soil density. Contradictory conclusions can be obtained on the failure of choosing a correct parameter to measure soil density (Carraro, Prezzi, and Salgado, 2009; Maleki et al., 2011; Murthy et al. 2007; Thevanayagam, Ravishankar, and Mohan, 1997).

The level of participation of different soil particles of different sizes within the soil matrix and their ability to transfer stress and strains within the interparticle and their ability to offer resistance is what dictates the behavior of a soil body. As shown in Figure 5, Karim and Alam (2017) explained how the sand with silt interact upon at different silt content. The behavior of the sand changes after reaching the limiting fines content also called threshold fines content. Void ratio and relative density have been used as the index properties for a mechanical response correlation, but they are not sufficient as they account for the void spaces but fail to consider the presence of smaller particles within the voids.




Source: Karim and Alam (2017)

**Figure 5. Visualizing limiting fines content**

Mitchell and Soga (1976) and Kenny (1977) realized that for mixed soils with fine particles, because of their size, nature, or position, they may not participate in the force transfer mechanism, and thus the space they occupy should be considered as void. This led them to introduce another index known as the granular void ratio ($e_g$). While void ratio (e) is defined as Vv/Vs and is the most commonly used method to calculate the void ratio in a soilelement, they do not consider the volume occupied by the fines. The conventional definition of void ratio (e) as used in clean sands, can be misleading since e value of void ratio cannot describe samples with different fines contents. Putting it another way, samples with the same value of $e_{sk}$ will have different void ratios depending on their fines contents. However, in the same way, that void ratio (or relative density) is inadequate for characterizing the behavior of sands, so the skeletal void ratio is inadequate on its own for characterizing clayey sands since it gives no indication of the distribution of clay within the soil (Georgiannou, 1988; Lade and Yamamuro, 1997; Pitman, Robertson, and Sego. 1994b; Thevanayagam, 2007; Thevanayagam et al., 2002; Vaid and Negussey, 1988; Zlatovic and Ishihara, 2013).

The skeletal void ratio considers the volume occupied by the fines as if it were void space. It is particularly better in representing sand with fines system as compared to the void ratio according to the studies carried out in the past (Thevanayagam, 1998). Although more versatile than void ratio in order to evaluate the states of soils, it fails to take into consideration the plasticity of the fines present in the system and hence have its own shortcomings (Carraro, Prezzi, and Salgado, 2009; Murthy et al., 2007; Thevanayagam, 1998; Thevanayagam et al., 2002). The sand with plastic fines has a higher granular void ratio than the void ratio of sedimented clean sand. It modifies the fabric of the sand, including contact conditions, so that changes in undrained brittleness can take place at constant granular void ratio; increases in clay fraction at constant e, reduce the stability of the fabric (Carraro, Prezzi, and Salgado, 2009; Thevanayagam, 2007; Thevanayagam et al., 2002; Vaid and Negussey, 1988). Sand skeleton void ratio is the void ratio that would exist in silty sand if all the silt particles were removed, leaving only the sand grains and voids to form the soil skeleton. The liquefaction of the soils also could be seen is independent of the amount of fines in the soil system.

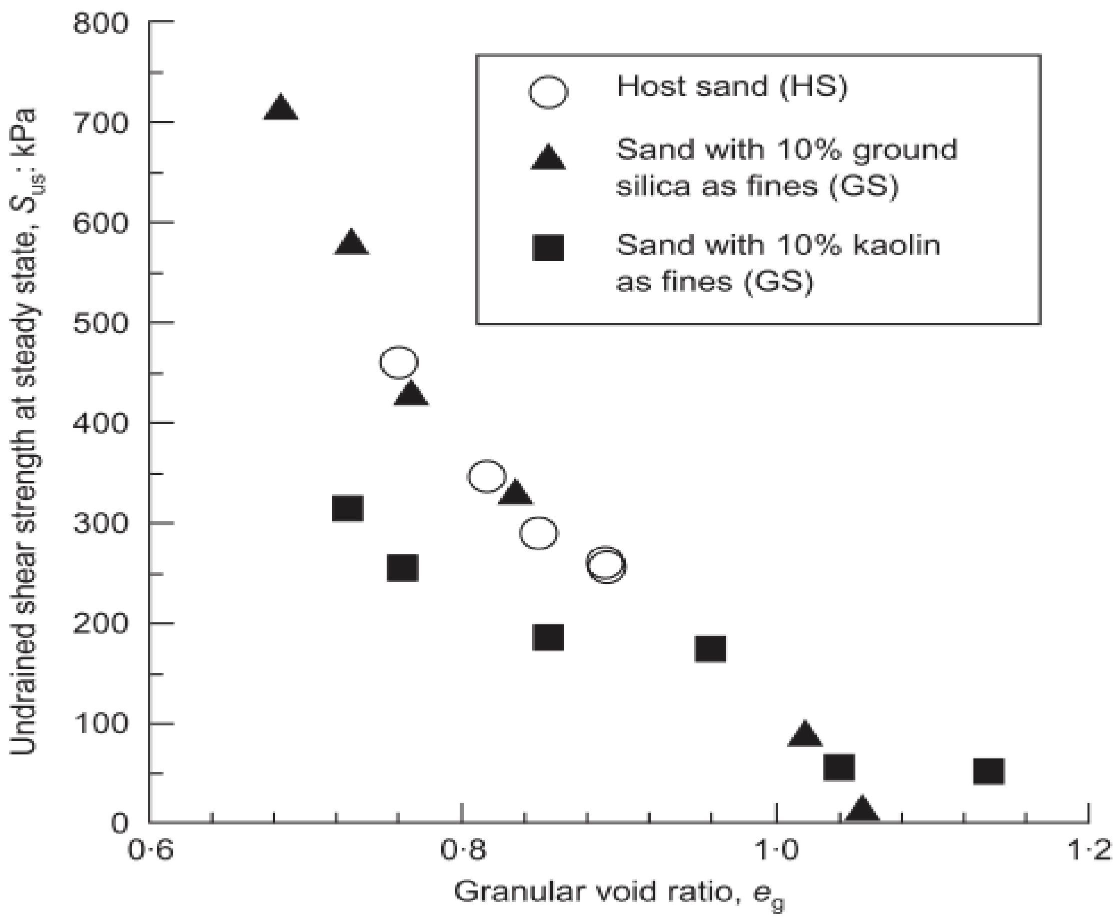
Although different researchers used different equations for calculating the intergranular void ratio, Chu and Leong (2002) showed that different equations used to calculate the intergranular void ratio are essentially identical. They also noted that the definition of the intergranular void ratio by assuming that fines are completely nonactive is not universally applicable for the entire range of fines content.

### 2.5. Contribution of Fines towards the Behavior of Mixed Soils

The majority of the investigations in granular soils have been carried out on pure sands (Kenney, 1977; Mitchell and Soga, 1976). However, most natural soils contain some amount of clay content as well as silt content. The nature of granular material during shear is determined by the stress levels, initial conditions of soils, void ratio, and effective stress-strain behavior strongly in the presence of small strains whereas at intermediate and high strains it is the void ratio that governs the behavior of soil.

In the case of sands, the behavior is modified depending on the void ratio or relative density (Lade and Yamamuro, 1997). For example, loose saturated sands can exhibit a strain-softening behavior under undrained loading leading to liquefaction under lower effective stresses. However, the addition of fines or their presence in sands affects their fabric and can influence their stress-strain behavior as well. Previous work has investigated the behavior of sands due to the presence of fines (Georgiannou 2006a; Georgiannou, Burland, and Hight 1990; Ni et al. 2004Pitman, Robertson, and Sego 1994a; Thevanayagam 2007; Thevanayagam et al. 2002). Thevanayagam et al. (2002) investigated the strength response due to the effect of sand mixed with silica and kaolin with samples of different granular void ratio. Each sample was prepared with the same sample preparation techniques as well as the same fines content. However, they observed different behavior showed by the soils in the presence of the different types of fines especially with respect to the undrained shear strength at steady state at the same granular void ratio in the soils. This means that the only factor affecting the outcome of the stress-strain behavior was the properties of the fines. In other words, the fines do not act merely as voids but act better or worse than the voids that they encompass. Therefore, it must be the properties of the fines that lead to this dichotomy.

The results as provided by Thevanayagam et al. (2002) are presented in Figure 6. They show different relations for the host sand, sand with kaolin and sand with silica. As shown in Figure 6, the strength of the silica fines with host sand is higher than the host sand




Source: Thevanayagam et al. (2002)

**Figure 6. Sus vs eg for silty sand at various fines content**

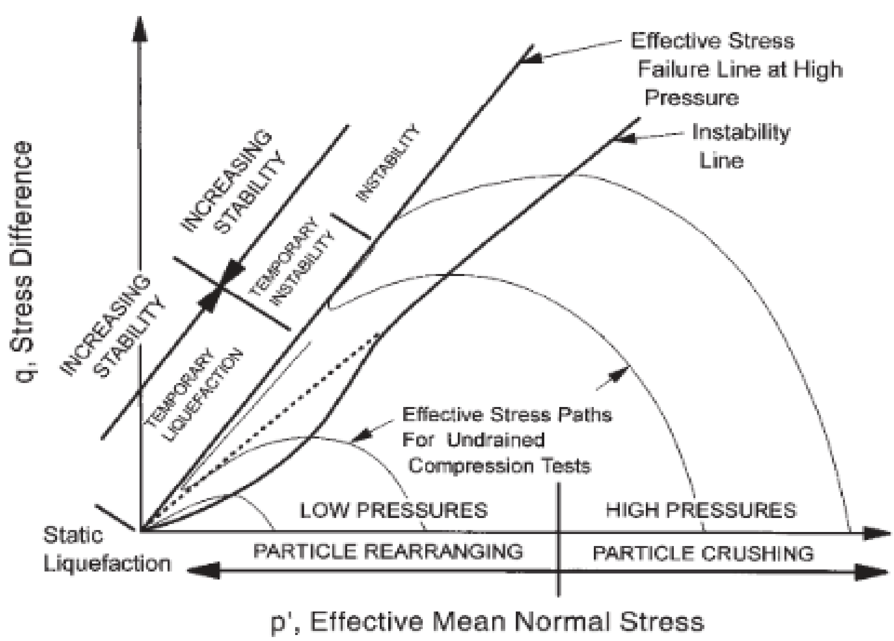
at the same granular void ratio whereas that of kaolin mixed with sand is less than that of the host sand. The observation made for plastic fines is consistent with findings reached by Georgiannou, Burland, and Hight (1990). This means that the fines present in the voids do not act as voids but may act better or worse than voids. It depends not only on just the properties of the fines but also on the granular particle interaction in the presence of fines.

## 2.6. Characteristics of Soil States

Lade and Yamamuro (1997) identified four characteristic types of undrained behavior in sands under monotonic loading: static liquefaction, temporary liquefaction, temporary instability, and instability. Static liquefaction occurs at low confining pressures and is characterized by large pore water pressure development resulting in zero effective stress and an increase in confining stress in this region leads to an increase in the friction angles. Temporary liquefaction occurs in the stress region above static liquefaction and is characterized by stress difference that reaches an initial minimum value and then increases at a particular point to a value much higher than the initial peak. This increase is caused due to the dilatant tendencies in the behavior of the soil which further causes the PWP to decline and thus increasing then effective stress again.

Temporary instability is similar to temporary liquefaction except for the fact that the stress difference is lower in temporary instability than during temporary liquefaction. Another aspect of temporary instability is that the soil undergoes decreasing dilation with the increase in confining pressure as opposed to soil showing increasing dilation in increasing confining pressure in temporary liquefaction. The stress path commonly observed in the sand (Figure 7 and 8) was first introduced by Yamamuro and Lade (1998) to describe the behavior sands under stress. There are three types of undrained behavior sand undergoes and this behavior is called "Normal Behavior". Under low-pressure sand undergoes contractive behavior and then undergoes suppressed dilation after passing the phase transformation line. This line is also called a steady-state line. The sand undergoes dilation caused after attaining negative pore water pressure where the stress path is directed towards the higher effective confining pressure as shown in the part of Figure 7 and, under this case, the sand is entirely stable.

The middle-stress path as shown in part B of Figure 7 for clean sands, is experienced at medium-high confining pressures. This results in greater positive pore water pressure which is large enough to cause a "wrap around" the top of the yielding surface. The stress difference increases and reaches an initial peak before a decrease in the pore water reduced it. The undrained sand behavior during the declining part of the curve is unstable in the sense that the sand cannot sustain a constant stress difference. However, as the effective stress path crosses the phase transformation line, the suppressed dilation produced negative pore water and it leads to a higher stress difference. At this point, the sand becomes stable again. Thus, temporary instability is observed in the middle ranges.




Source: Lade and Yamamuro (1997)

**Figure 7. Four distinct stress paths in sands**

The third effective stress paths which is also observed at higher confining pressures indicate large contractive tendencies (Figure 8 and 9). Under this case, the pore water pressure increases continuously under strain and the effective stress path reaches a peak and after which it keeps declining with the increase in PWP. In this case, the soil undergoes a constant decline and the phase transformation line is not encountered and the eminent decline continues till it reaches the failure surface. This behavior pattern is generally considered liquefaction observed in part b of Figure 9.

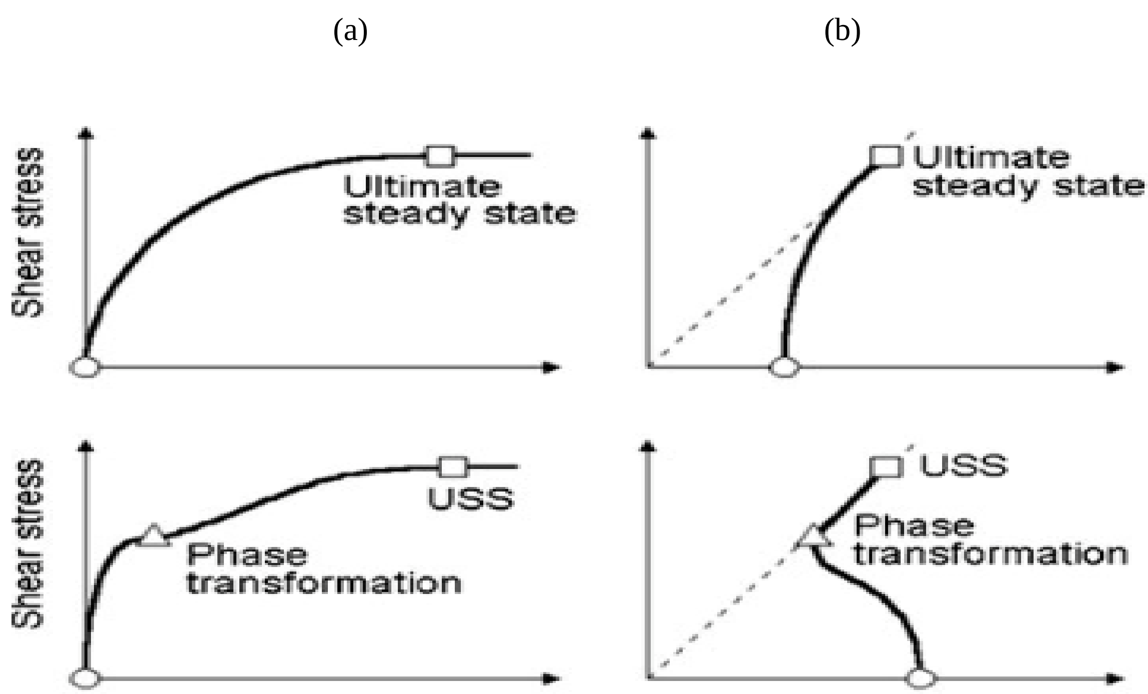



**Figure 8 (a) Dilative behavior of sand causing stable deformation; (b) Phase transformation and unstable stress-strain curve**

(b) (b)

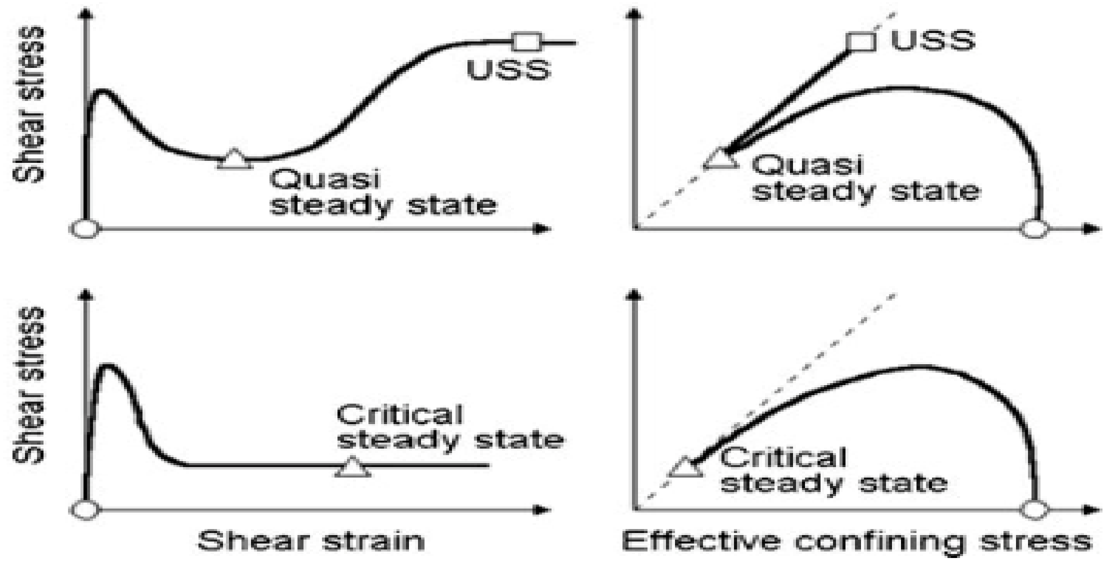



**Figure 9. (a) Unstable behavior of sand**
**(b) Liquefaction of sand**

**2.7. Effects of Plastic Fines**

Unlike the non-plastic fines, the behavior of plastic fines depends on more factors like clay mineralogy, fines content, the plasticity of fines, pore water chemistry, etc. which influences the fabric of the sands along with the density and stress states. Therefore, it is important to study the behavior of sand in the presence of plastic fines as much as behavior in the presence of silt and other non-plastic fines. Georgiannou (2006b) showed that the particles of mica fines which are flat in shape lead to the different behavior of sand than the addition of kaolin fines which are more rounded in shape. The importance of shape, size, and location of the particles is critical in determining the behavior and hence the consideration of soil fabric is very important. At the same granular void ratio, an increase in fines content would eventually force the coarser grains to disperse fully in the finer grain matrix. In effect, it is the modification of the structure by the added material that results in the observed modified behavioral response Georgiannou (2006b).

However, numerous studies have reported that the behavior of the fines is affected majority in predominantly soils with plastic fines by the plasticity of the fines added and their amount. Georgiannou et al. (1990) investigated the effect of increasing clay content on the undrained behavior of the sand with kaolin clay. They observed that as the clay content increased from 4.6–10% the initial small strain response is unaffected. However, at a constant skeleton void ratio as the clay content increased to a critical value, the undrained shear strength of the sand kaolin matrix at a quasi-steady state decreased. Thus, the undrained brittleness for the strain at the quasi-steady state increased with the increase in the fines content. The quasi-steady-state was defined by Ishihara (1993) as the state when the soil behavior changes from contraction to dilation. Ishihara also observed that this trend reversed when the clay content increased more than 20% and especially about 30%, the soil is no longer dilatant in nature and shows behavior similar to that of clays (i.e. the clay dominates the behavior in the host sand). However, the author had not made any comment with respect to the threshold limit of the fines content

Pitman et al. (1994a) observed that hardening occurs at a very high fines content of 40% strain. The soil changes from its brittle behavior to ductile which implies that at 0% fines content the soil has more potential for large deformations due to strain softening. The steady-state condition which is represented by constant state stress can only be achieved at a very large value of strain which could be difficult to achieve in a lab due to equipment limitations.

Georgiannou (1988) conducted an investigation on the behavior of clayey sands under monotonic and cyclic loading and concluded that as the fines content of the soils especially in loose state increase the dilatant behavior of the soils is suppressed and the response gradually becomes controlled by the fines matrix at a certain amount of fines content, also called threshold fines content. Georgiannou, Burland, and Hight (1990) performed further experimental studies using Ham river sand into a kaolin suspension concluded that this method creates a material which is markedly less stable, which has a higher granular void ratio and exhibits a higher undrained brittleness behavior if compared with the same sand that is sedimented through clean water (i.e. contains no clay) Ölmez, (2008). These authors also observed that as the granular void ratio decreased as the plastic fines within the soil increases up to a certain critical value and also affected the undrained shear strength of the soil at quasi-steady state decreases. Ölmez also stated that the presence of clay fractions did not reduce the angle of shearing at critical state for the sands but affected the stability of the fabric, thus reducing the undrained shear stress.

A strain-controlled approach is used to run the test as the increase in porewater is controlled by the amount of induced stress Bayat et al. (2014). The study inculcated the results of tests run on sand specimens with the addition of 0–30% kaolin fines with the same relative density and consolidation with a mean confining pressure of 50psi. According to the authors, the peak strength decreases as the fines content increases up to a threshold content of fines.

Ovando-Shelley and Pérez (2009) investigated the undrained behavior of moist tamped sands with fines at 3%–7% fines at moderate to high strains and found that the presence of fines reduces the strength and stiffness and potentially increases the porewater pressure within the soil sample. However, it depends upon the granular void ratio and the stress state relative to the failure envelope and loading path. They found that the fines in small quantities do not contribute to stress bearing capabilities of the host sands but they do modify the fabric of the sand, generating more loose and unstable structures. Sands consolidated under different stress ratios, the potential for generating PWP during undrained loading depends on the initial intergranular void ratio, on the relative position of the stress state with respect to the failure envelope. The relationship between granular void ratio and the stress-strain relationship is important in many practical situations in which the initial state of NC clay sands occur at different varying effective stress ratios and can be used to understand the behavior of slopes, embankments or base of dams and footings.

**2.8. Effect of Silts and Non-plastic Fines**

The occurrence of silty sand around the world has led to more research on the influence of fines on sand behavior. Many researchers have carried out research in the past by testing a different combination of fines content with sands at different relative densities to determine the understanding of both the stability of sand and also its effect on the sand fabric.

Broadly speaking, the observations noted by these researchers were directly related to the amount of fines present in the sand matrix. For example, at low fines content, it was observed that the host sands dominate the behavior of the soil under stresses due to the dominance of the sand in the soil matrix and since the fines present do not completely acquire the void spaces present in the soil. When the fines exceed a certain value in the sand, the soil behavior is dominated by the fines and the sand no longer controls the behavior during shear. This limit is also called as the limiting threshold of fines, after which the fines start dominating the behavior of the soil and the behavior changes. This also leads it to affect the instability of the soil matrix. Thevanayagam and Mohan (2000) proposed that during the presence of low fines content in the soil, the fines do not completely occupy the void space present between the sand particles and hence act secondary during the overall soil behavior on loading.

Pitman, Robertson, and Sego (1994b) tested Ottawa sand with non-plastic crushed quartz and kaolinite at a confining pressure of 350 KPa with varied fines content ranging from 0–40%. They concluded that the effect of non-plastic fines was to create slightly more dilative responses resulting in the sand that is less likely to liquefy. Data from Pitman et.al. (1994b) showed the variation in undrained shear strength of soils with fines under constant effective stress with varying fines content. The data also showed that sands with plastic and non-plastic fines with the same amount of fines content vary in behavior and also in the undrained shear strength. This means it is not just the amount of fines but also their behavior and other properties like plasticity are an important factor affecting the strength of the soils.

While undertaking a comprehensive study to explain shear strength, small scale stiffness critical state and fabric of sands containing fines, Carraro et al. (2009) concluded that in case of low fine content where fc<<15% where the fines can be either non-plastic or kaolin observed that the addition of non-plastic fines to the host sand increases both the peak and critical state friction angles of the soil and make it more dilative than clean sand for loose states. On the other contrary, the addition of fines of kaolin clay to the host sand decreases both the peak and critical state friction angles of the soil and imparts a less dilative and more contractive response for soils in all states. In other words, they observed that non-angular silt particles tend to interact with the irregularities and produce a jamming response during shearing whereas kaolin clay produces more lubrication or smoothening response during shearing of the soils.

These studies on sand revealed that the silt makes the soil more unstable or even liquefiable on its addition to sands whereas others specify that the silt initially decreases the resistance to liquefaction potential and increases after a specific quantity of silt is added to the mixture. It was also observed that the presence of fines in the sand makes the sand more dilative and hence the strength of sand at 50% silt content is less than that of the sand at the same dry density.

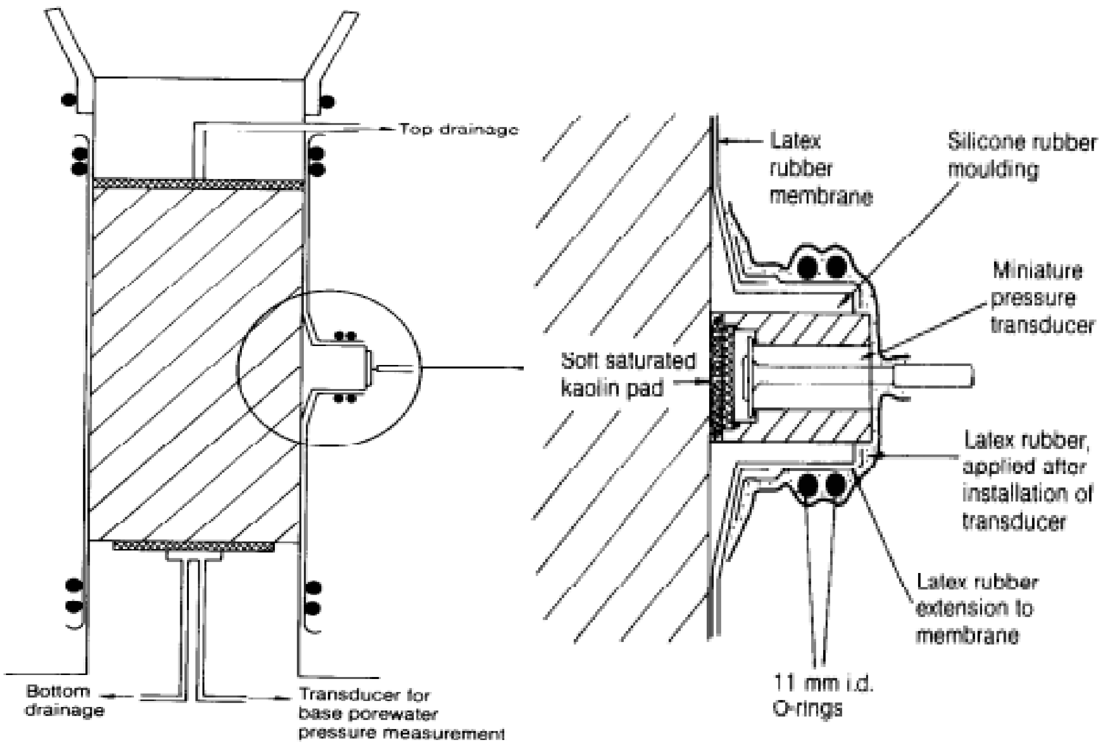
# CHAPTER 3. MEASUREMENT OF PORE WATER PRESSURE GENERATION AT SPECIMEN MID-HEIGHT

Conventionally pore water pressure is measured at the ends of the sample during any undrained triaxial tests. However, there are two methods of measuring the pore water pressure developed in the sample:

Method 1: Run the test at a constant small strain rate especially in a cohesive soil sample and measure the pore water pressure hence developed at the ends of the soil sample.

Method 2: Place a localized pressure measuring device (pressure transducer) in an anticipated failure zone or at any point in which PWP measurement is desired. (Baldi, Hight, and Thomas 1988; Cherrill, 1990; Hight 1982)

The conventional approach of choosing method 1 arises from the fact that it is easy and reliable and, in most cases, gives optimum information expected from a triaxial test. The second method requires more skill and is used only in cases where extensive information regarding PWP generated within the soil samples are needed. It is also important to have a fast measuring device that is sensitive enough to measure a small change in the pressure developed within the sample. The use of transducers to measure both the pressure at both ends and at mid-height gives extremely valuable information with respect to controlling the rate of consolidation, strain rates necessary for shearing, etc. The success of a transducer relies on its capability in measuring pore water pressure at a high rate and with accuracy and with minimum interference with the sample. Figure 10 illustrates the miniature probe design as used by Hight (1982) in one of the first tests carried out using a piezometer probe (Baldi, Hight, and Thomas, 1988; Fourie and Xiaobi, 1991; Hight 1982).




Source: Hight (1982)

**Figure 10. Miniature mid-height piezometer probe**

There are many advantages of using a probe along the length of the soil sample. According to Hight (1982), it is advantageous to use a probe since it saves time as it reduces the time for the pore pressure to achieve equilibrium while performing consolidation in cohesive soil samples. It improves the data interpreted with respect to PWP measurement in cyclic as well as monotonic tests. This could help in a better understanding of PWP flow in soils in case of cyclic loading which helps further understanding of the liquefaction phenomenon. The combination of pore pressure measurement at the mid-height and also at the ends is advantageous during stress path testing as the rate of stress change can be controlled to maintain an acceptable low gradient of PWP in the sample during drained stress path testing. In addition, consolidation can be carried out at any PWP gradient to simulate field conditions.

One particular problem with measuring pore water pressure at the ends of the sample is end restraints which have a considerable effect on the measurement. End restraints are caused by the rough ends at the end platens and are a recognized problem for a long time. The use of lubricated platens has been in practice since and despite it they are not 100% smooth and lead to discrepancies in the data. Errors usually occur while measuring the PWP because it is conventionally measured at the end of the sample which is not representative of the PWP throughout the specimen. In an undrained test, the excess pore water pressure is not allowed to dissipate and no flow of water is allowed from the specimen or towards it. Hence the pore pressure generated is non-uniform and depends on the rate of loading especially when dealing with fine cohesive materials with low hydraulic conductivity. The equalization of pressure takes place by the flow of water at high PWP areas to low PWP areas and a complete equalization only takes place when the test is allowed to run at a very slow rate. There has, however, not been a lot of research done on pore pressure developed within a triaxial specimen and the ones available are the ones carried on an undrained unconsolidated triaxial test.

Fourie and Xiaobi (1991) used a mid-height pressure probe while measuring the effects of end restrains on pore water pressure measurement within the sample in a UU test. The authors observed that the testing rate did not affect the stress-strain curve while working with high plasticity clays. They also observed that the PWP at the base of the probe was 50% higher than the midpoint in an isotopically consolidated clay and the magnitude of the difference was dependent on the rate of testing. The authors stated that the effective stress-strain parameters could be obtained from undrained testing provided the piezometer probe is placed at the midpoint.

# CHAPTER 4. EXPERIMENTAL PLAN

## 4.1 Material Properties

**Kaolinite:** The material used in this research was pulverized EPK kaolin clay from Edgar Minerals™. The chemical analysis of kaolin clay is provided in Table1, which describes the chemical, whereas Table 2 describes the ceramic properties of the material (Maher et al. 2018). Kaolin is extremely plastic, in which the LL = 53 and PL =35. Figure 11–14 provide sieve analyses and semi-logarithmic plots of water and silt content.

**Table 1. Chemical Components of Kaolin Clay Fines**

| Mineral type | Percentage | Mineral type | Percentage |
|---|---|---|---|
| $SiO_2$ | 45.73% | CaO | 0.18 |
| $Al_20_3$ | 37.36 | MgO | 0.098 |
| $Fe_2O_3$ | 0.79 | $Na_2O$ | 0.059 |
| $TiO_2$ | 0.37 | $K_20$ | 0.33 |
| $P_20_5$ | 0.236 | LOI | 13.91 |

**Table 2. Physical and Ceramic Properties of EPK Kaolin Clay**

| | |
|---|---|
| Median Particle size (microns) | 1.36 |
| Specific surface areas (m2/g) | 28.52 |
| Mineral content (X-Ray Diffraction) | Kaolinite ($Al_2O_3$,$2SiO_2$,$2H_2O$)-97% |
| Specific Surface Area ($m^2$/g) | 28.52 |

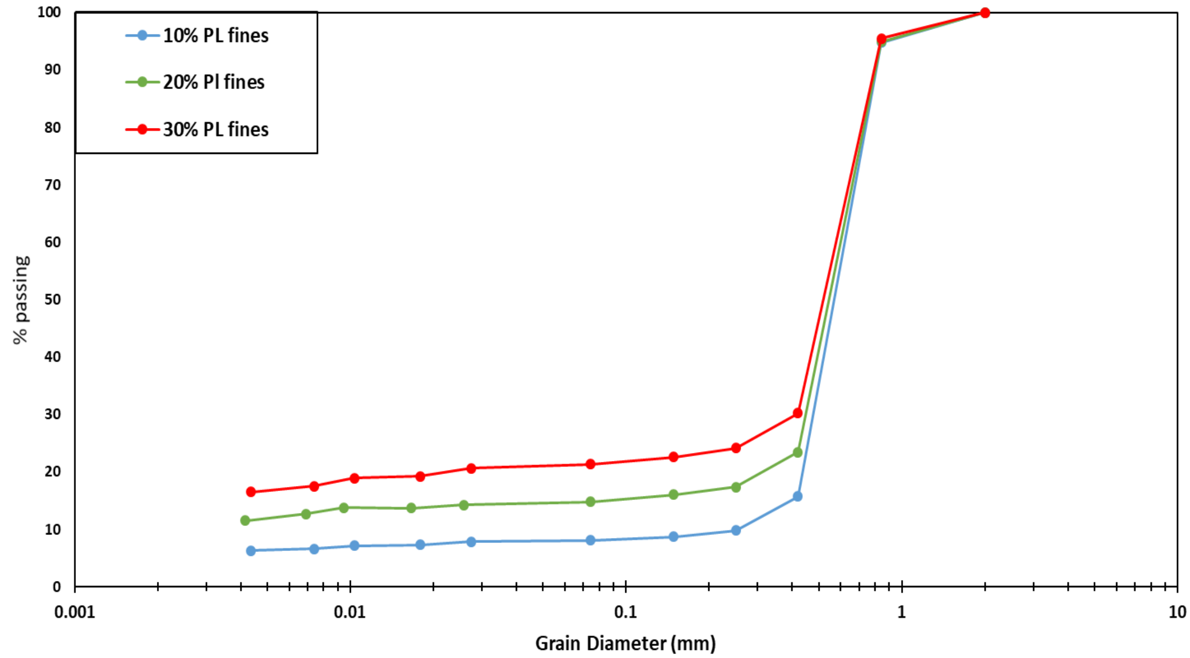



**Figure 11. Sieve analysis of Kaolin clay**

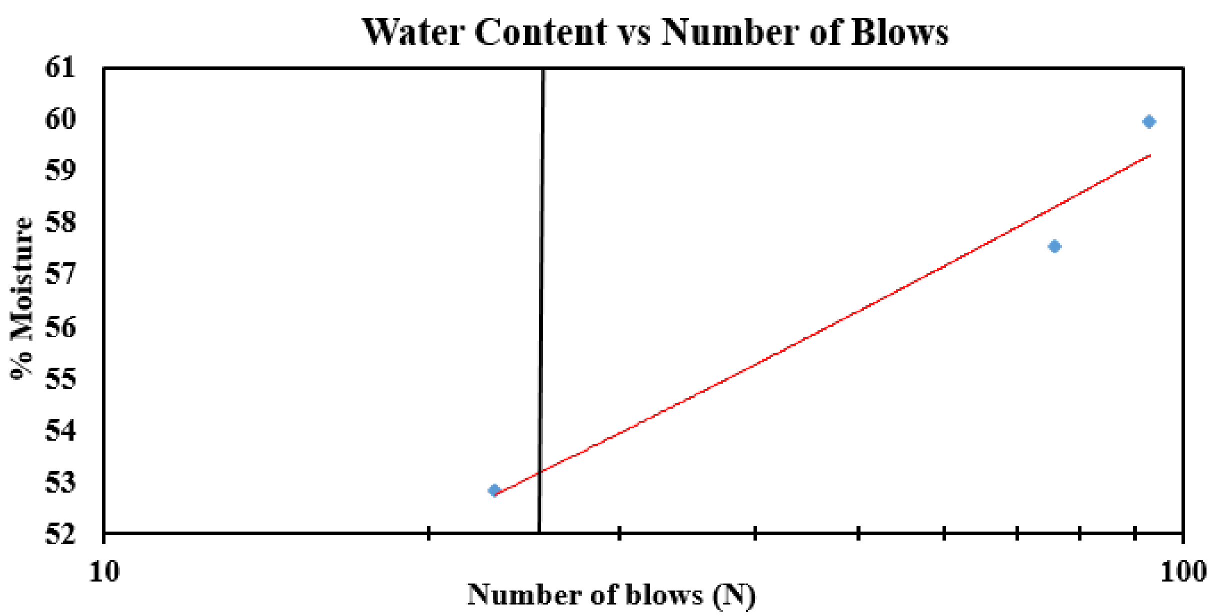



**Figure 12. Semi logarithmic plot of water content vs the number of blows with N=25 indicated for Kaolin clay**

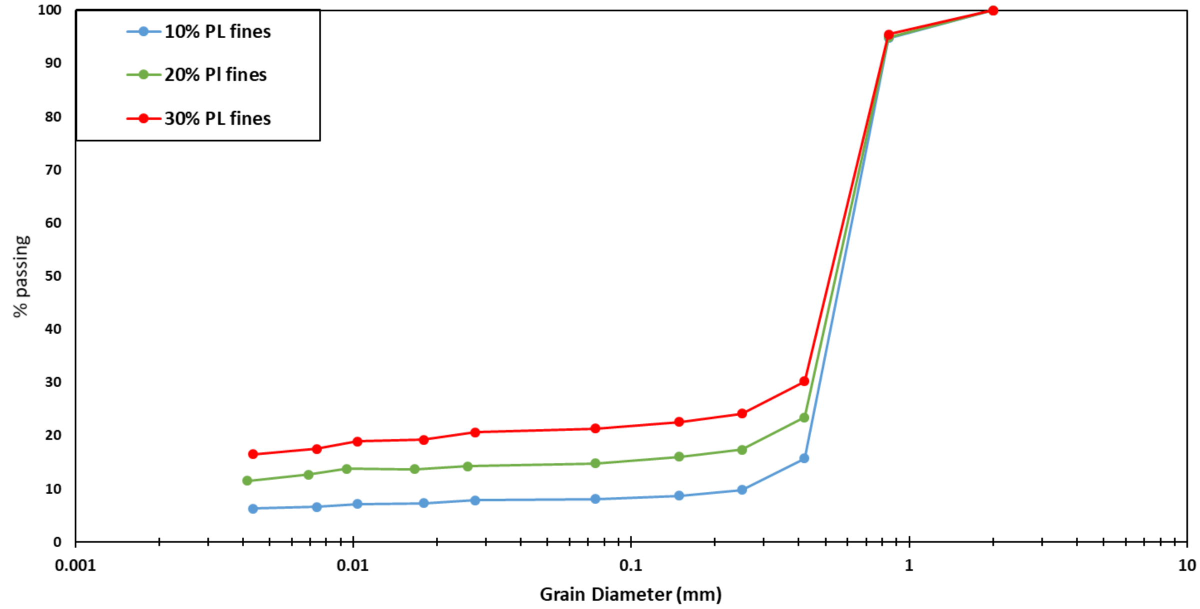



**Figure 13. Sieve analysis of Loess silt**

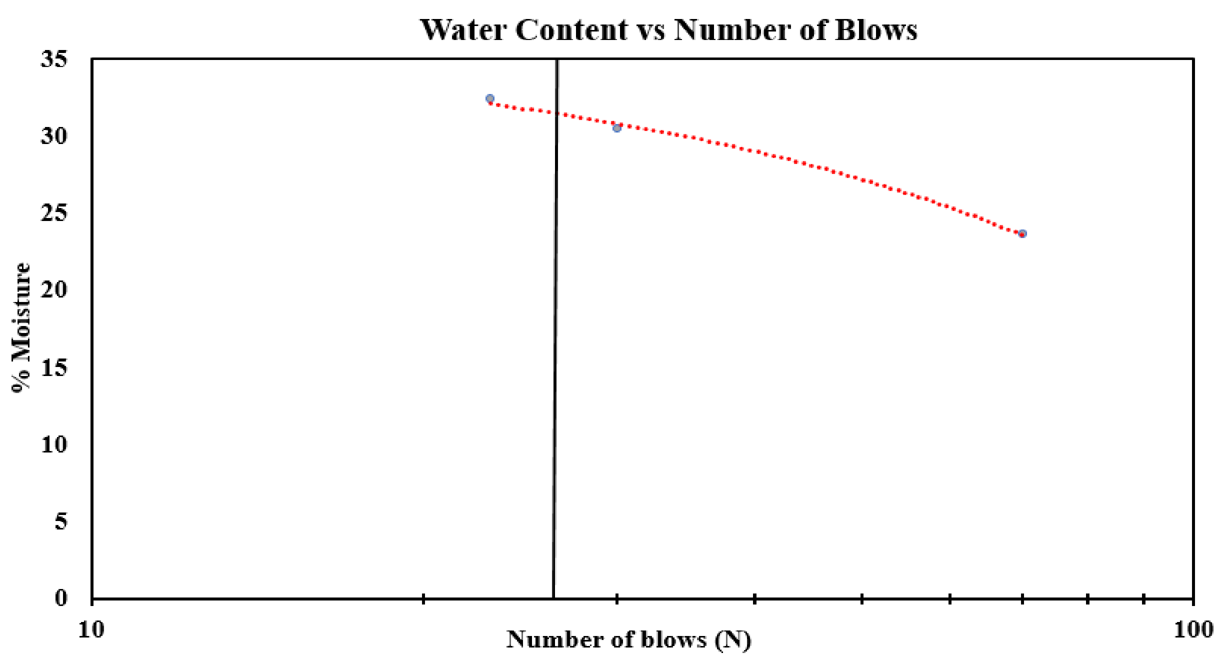



**Figure 14. Semi logarithmic plot of water content vs the number of blows, with N = 25 indicated for Loess silt**

**Loess**: Loess soil was used to carry out tests on silty sand. The loess soil used in this test was collected from the Loess Hill areas in Monoma County, Iowa. Approximately 82% of the soil has fine particles (< 0.075mm), with other sand-like particles. The soil is brown in color and silty in nature. The specific gravity of the soil is 2.74 with a Liquid Limit (LL) = 32 and Plasticity Index (PI) = 6. The soil was classified as (ML) according to USCS unified soil classification system. Mahedi, Bora, and Dayioglu (2019).

## 4. 2. Test Apparatus and Software

After a brief review of the advantages and limitations of triaxial testing on cylindrical specimens, the following topics are discussed:

- methods of measurement, including axial load, pore pressure, and axial displacement, with special emphasis given to axial deformation measurements;
- methods of preparing and installing specimens;
- methods of specimen saturation;
- consolidated undrained testing; and
- Measurement of pore water pressure the mid-height of the specimen.

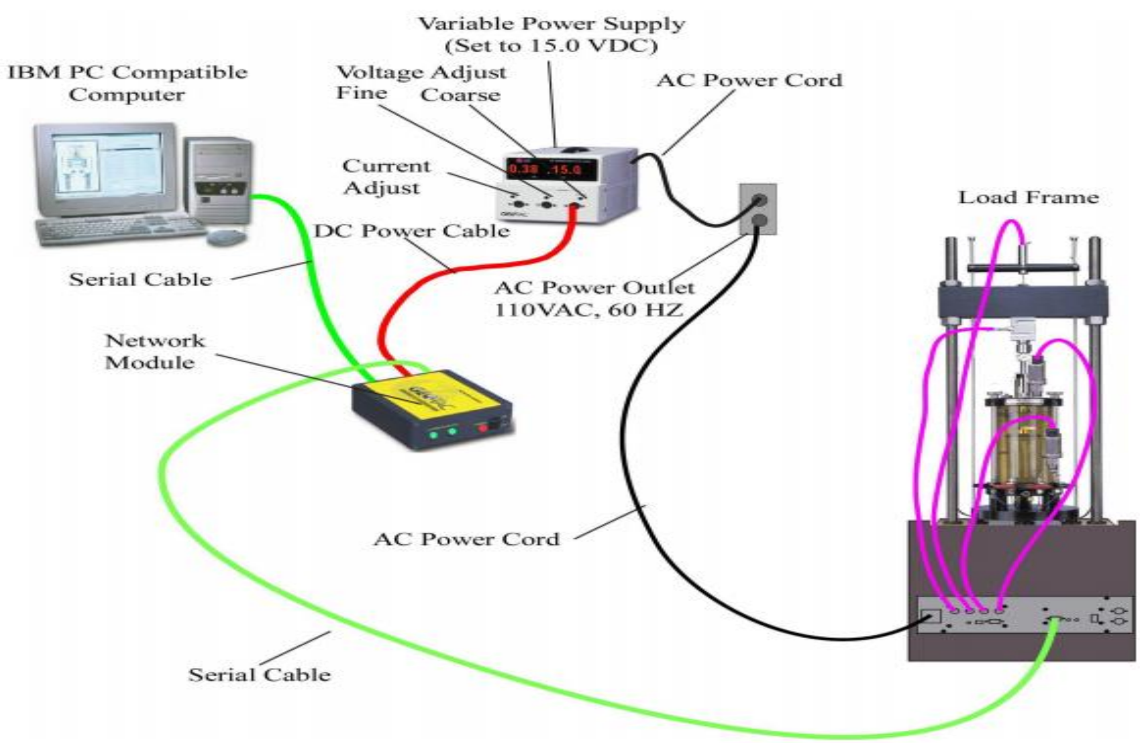
### 4.2.1. Triaxial test device

Geotac Sigma1 System (Figure 15) was used for carrying out tests on the cohesionless soils. The system consists of a load frame used to apply external loading with the help of a loading jack. The samples tested in the test were 1.4" * 4" dimensions for testing the mid-height pore water pressure whereas 2.8'* 5.6" specimens were used to conduct tests on the effect of fines. Drainage is provided from both top and bottom platens in the triaxial cylindrical cell using water and a pressure panel by Trautwein.


u

**Figure 15. Geotac Monotonic Triaxial testing setup**

Conventional triaxial tests were conducted due to the relative ability with respect to other tests in measuring the principal stresses in all dimensions. Backpressure was provided to the system using the pressure panel with the pressure values depending on the tests and the type of sample. The top and the bottom platens were lubricated using a silicone gel in order to reduce the non-uniformity of stresses within the specimen caused due to end restrains. The best solution to the problem of measuring the pore water pressure within the length of the sample was solved by modifying the top cap of the triaxial device. Special modifications were made in the top cap of the triaxial cylindrical cell for this particular research in order to carry out this particular study (Figure 16). The top cap was drilled with plastic caps with openings to pass 1/4" diameter tubes to facilitate the measurement of PWP within the soil chamber. These were later connected to the pressure transducers placed above the top cap. Special care was taken to prevent leakage through these openings so that the pressure within the chamber is maintained during the tests. Pressure transducers were connected with the system to correctly measure the PWP within the soil at both the top and bottom of the chamber and to measure the cell pressure within the chamber.




Source: http://www.geotac.com/products/truepath/

**Figure 16. Systematic view of Geotac Setup Digital Image**

**4.3. Twin Towers and Mid-height Porewater Pressure Measurement Device**

It is important to make sure that the needle connected to the membrane at the mid-height does not get disturbed during sample deformation. Hence it was necessary to provide support for the needle hanging freely from the tube connected to the top cap as shown in Figure 17–19). This was carried out by 3-D printing a mold which can be fixed at the bottom cap without affecting the water tubes connected to the top pedestal (Figure 17). The mold has circular holes at a specific height for the basic to pass through and stay in place. The heights of the opening of the holes were such that the needle can be inserted at mid-height, top, bottom or 1/3rd, the length from the top or bottom of the sample. Tests were done to make sure the needle does not affect the outcome of the tests. During the initial tests where the needles had directly penetrated within the sample without the twin towers, it was observed that the needles were disturbed during the sample preparation process which led to punctures in the membrane (Fourie and Xiaobi, 1991). This led to the influx of cell pressure within the sample which led to the failure of the samples being created at it could not keep the 2 separate pressure systems apart.

As shown in Figure 17, the twin towers provide stability to the needles which do not budge during the sample preparation process and hence produce the required stability by preventing leakages and thus reduce the risk of sample failures. The design consists of a circular ring base with 2 towers based on its upper face of the ring. The towers consist of 3 circular openings each, with the dimensions equal to the dimensions of the crown of the needles. This ensures the needles are placed intact. The openings are placed at positions equal to the 1/4th, 1/2 and 3/4th the length of the sample on one side and 1/3rd, 1/2, and 2/3rd the length of the sample on the other. The lower base of the sample consists of an

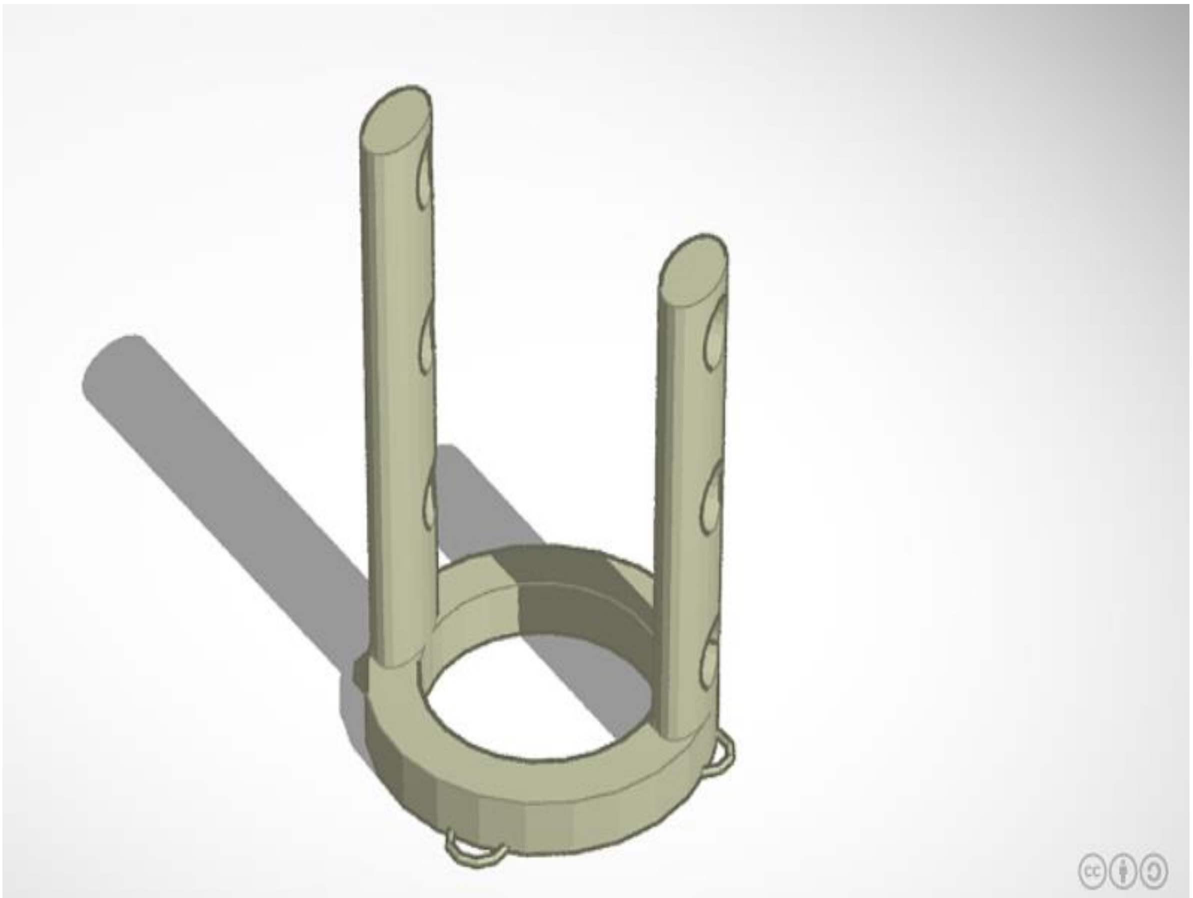


**Figure 17. Twin Towers**

arrangement for attachment to the base of the triaxial base for further stability to prevent any angular displacement (Figure 18 and 19). The design was 3D printed using the Iowa State University's CTLT Department at a very minute monitory cost.

**DATA acquisition system**: A USB directly powered data acquisition system named OM_DAQ_2400 by OMEGA Engineering was used to acquire data from the pressure transducer. The system has a 16bit single-ended analog input used to collect the data (Figure 21).

**Figure 18. Setup for measuring mid-height PWP**

**Figure 19. Setup for insertion of the needle at mid-height**

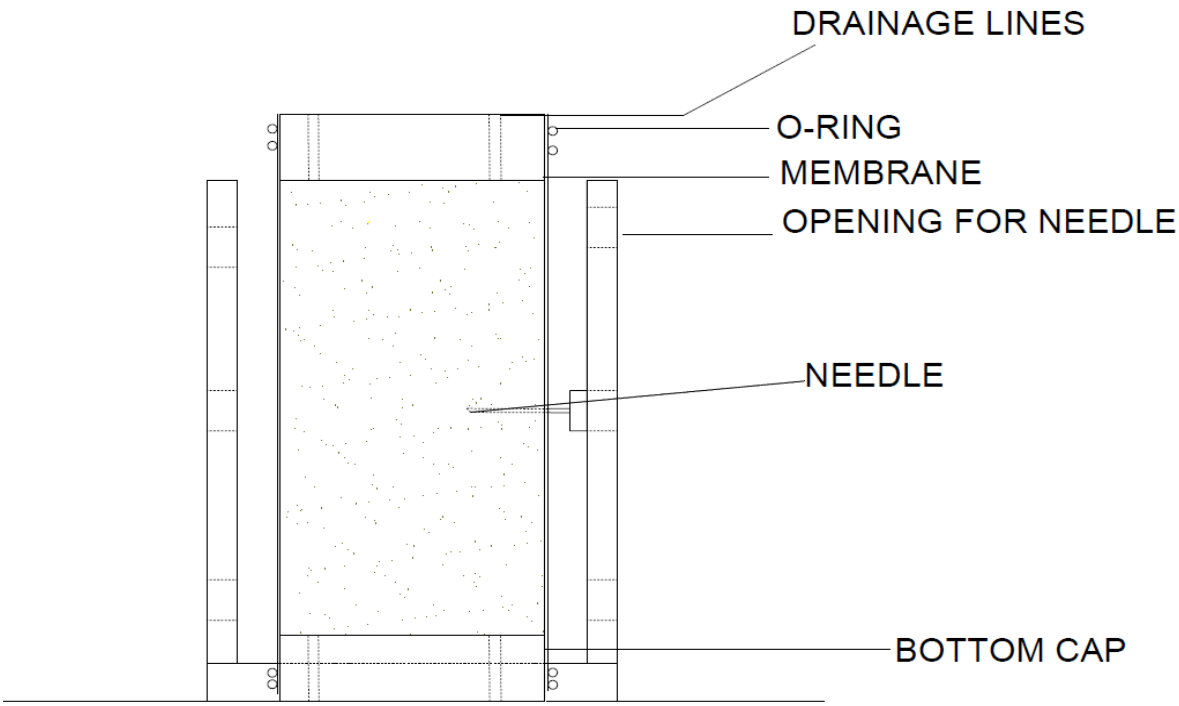



**Figure 20. Test setup for measuring mid-height PWP**

### 4.4. Internal Pore Pressure Measuring System

**Pressure transducer:** PX 119-100AI pressure transducer by Omega Engineering was used to measure the pressure at the center of the soil sample (Figure 21). It is a piezo-resistive transducer used with an accuracy of 0.5%. The PX119 pressure transmitter series is ideally suited for material handling, industrial and mobile equipment applications where space constraints require a small body size.


**Figure 21. Data acquisition system and pressure transducer**

# CHAPTER 5. TESTING PROCEDURE

## 5.1. Test Preparation for Cohesionless Soils

Testing of homogenous soil samples under uniform stresses or strain is required for the study of many fundamental properties of soils along with analysis of their behavior under various boundary conditions. It is of vital importance to be able to produce several reconstituted samples having the same soil fabric with the desired relative density in order to be able to carry out multiple tests. The preparation techniques used for the cohesionless samples greatly affect the properties of the cohesionless soil sample. Special care needs to be taken for specimen preparation so that the properties of the soil achieved at the end of the tests do not have any errors be it mechanical or human. Every step of the process needs to be carried out as precisely as possible. However, in the case of cohesionless soils like sand, it is very difficult to achieve non-disturbed samples in-situ. Therefore, in this study reconstituted sand samples were used. The sands tested were Ottawa sands which were mixed with fines of either clay or silt. Pure sand samples were prepared using the air pluviation method where changing the height of the drop leads to denser materials. However, when dealing with sand with fines, wet tamping was used in order to achieve a thorough mix to prevent any segregation during deposition. In order to prepare the soil sample, the sand was mixed with the desired number and type of fines.

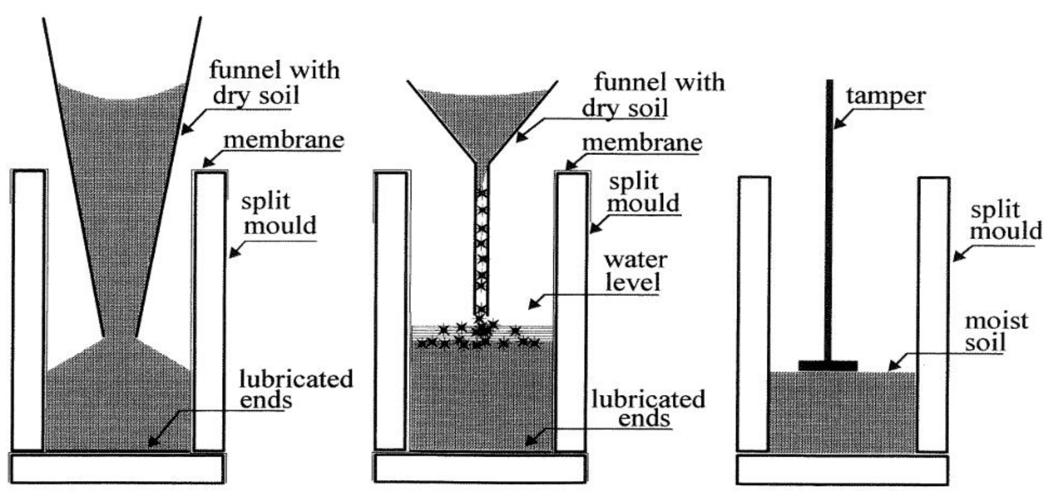
## 5.2. Influence and Comparison of Different Specimen Reconstitution Methods

There are only two ways of obtaining soil samples required for laboratory testing. One technique is to obtain an in-situ undisturbed sample from the ground in thin-walled tubes with proper sealing on both ends with paraffin wax and then later carefully extruded in the lab. These samples are later trimmed to the desired size and various tests are conducted on it to understand its behavior and obtain the engineering parameters necessary. However, this is a laborious and expensive technique and cannot be used to carry a myriad of tests for research purposes on a regular basis. It is also difficult to obtain undisturbed samples in cases of granular soils such as sand as they cannot be sampled without disturbance unless they are frozen in-situ (using liquid nitrogen), cored and then later shaped in suitable specimen size all in the frozen condition using a lathe and later tested after it has been thawed within the latex membrane. This technique also required a cold room and freezing temperatures.

Past research has indicated that the technique used for sample preparation has a strong influence on the stress-strain response and the mechanical behavior of the specimen (Carraro et al., 2009; Murthy et al., 2007). However, experimental data related to the depositional effect has been observed to be very limited as most of the research was focused more on clean sands. Efforts can be made so that the reconstitution technique simulates the depositional process of natural soils, with the soil fabric being similar to natural soil deposits of similar nature. However, almost all of the processes do not accurately depict the natural soils, and more research needs to be carried out to make sure the reconstitution process depicts the natural soil deposits. Uniform and repeatable clay specimens can be obtained by trimming the clay deposits naturally obtained from a sampler. However, this is not possible in case of sand samples except by using frozen samples which is an impractical and expensive process. The goal in each of the reconstituting technique is to achieve soils samples which simulate the same fabric, void ratio as that of naturally occurring in–situ depositional process. These techniques become more important in cases of fines content as a uniform mixture of the host sand and the fines are the key and segregation are prevented as it could lead to incorrect results.

The main goal in using reconstituting soils is to make sure that the technique is repeatable and the desired average density throughout the length of the sample is acquired. The most commonly used techniques are Wet Tamping (WT), Air Pluviation (AP) and slurry deposition (SD), water sedimentation (WS) techniques. All of these techniques have their own advantages, however, none of the techniques truly create a sample simulating naturally occurring soils.

Figure 22 provides an illustration of different soil reconstitution methods. In the work carried out by Yamamuro and Wood (2004), wet deposition methods appeared to indicate a more volumetrically dilatant or stable response, while dry methods appeared to exhibit a more contractive or unstable behavior. It was also observed that the effects were more pronounced at lower densities than higher densities. This is probably due to the fact according to the authors that to achieve a high density in a specimen mold, a fairly high amount of energy must be applied to the soil as it is deposited. Thus, the effect of the depositional method is negated, because the resulting soil fabric is probably very similar.




Source: Zlatovic and Ishahara (2013)

**Figure 22. Comparison of different specimen reconstitution methods**

### 5.2.1. Air Pluviation (AP) technique

In the air pluviation technique, dry soil is pluviated through a fixed height. The idea being that with an increase in height, the velocity of the soil grains is used to achieve densification and hence as the drop height increases the void ratio increases as well and densification can be achieved. The pluviation of sand appears to create samples that are closest to simulate the sedimentation of depositional processes found in nature as the main factors controlling the deposition of sands are wind or water. The void ratio and the structure of the sand fabric are the factors determining the behavior of sands (Lade, 2016).

Air pluviation samples are used to create samples right within the membrane jacket by sand raining. The setup might include a container with a shutter at the bottom, a diffuser screen, a long tube easily made with cardboard with the inside diameter equal to the split mold which is covered with a latex membrane. The tube is placed on the split mold and the sand is rained on the bottom of the split mold containing a saturated porous stoned and filter paper. A variation in height is used to achieve sands of different densities. In order to achieve the loosest of the samples, specifically for liquefaction tests and to achieve samples with low relative density. Terminal velocity is reached at a very small drop height, and homogeneous samples of the same initial density tend to be formed by pluviation of uniform sand in water. Different sand densities are achieved by changing the rate of drop of sand poured in the tube. This can also be achieved by changing the number of holes and the size of the holes in the shutter and thus achieve a controlled deposition rate. However, the rate of drop and the relative density thus achieved along with the drop height changes for every sand and also on the value of the desired void ratio, hence it should be noted that experiments need to be carried out for each individual sand Lade (2016). Preparation of triaxial sand samples by pluviation in water is also recommended by some researchers as it results in initially saturated specimens, and homogeneous samples of desired densities can be replicated without difficulty (Y. Vaid and Negussey 1988; Yamamuro and Wood, 2004).

#### 5.2.2. Dry Funnel Deposition (DFD)

Specimens prepared by this technique are initially formed by placing sand or sand and silt mixture in a funnel and placing the spout of the funnel to the bottom of the split mold. The spout of the funnel is raised slowly, and the soil is allowed to deposit along with the height of the mold in a symmetric manner. To achieve higher densities of soil, the funnel is raised at a greater height quickly. In order to achieve higher densities, the split mold is tapped gently from all sides evenly at various intervals.  Yamamuro and Wood (2004) state that air pluviation and dry funnel deposition method can be used for making sand with silt mixtures, but the tests carried out during this research found particle segregation a reoccurring concern in case of sand and fines specimens. The AP would be a recommended technique for making reconstituted specimens of pure sands Wood, Yamamuro and Lade, (2008).

#### 5.2.3. Moist tamping

Moist tamping is used to prepare reconstituted samples when needed to test on liquefaction. Loose samples of sand are possible especially due to the capillary effects between grains and are mostly observed to have more contractive stress-strain behavior which may unjustly lead to a liquefaction behavior (Vaid et al., 1999). The technique used for carrying out moist tamping has evolved with passing years. A known weight/volume of soil is mixed with a known weight/volume of water and mixed thoroughly and placed over several layers. In the test consisting of fines and sand mixture moist tamping was used in 5 layers. Each layer was tamped using a tamper of a known weight and equal blows were applied so that a uniform density throughout the specimen could be achieved. Casagrande (1936) noticed that sometimes moist tamped samples create bulked, honeycombed structures that liquefy upon saturation. During these tests a few samples, similar problems were faced for a few samples, but the technique was found satisfactory to the needs required for the tests especially uniformity and consistency in achieving repeatable samples quickly. It has been argued that the under-compaction technique promotes the reconstitution of uniform specimens but direct evidence showing the void ratio profile over the specimen height has rarely been presented. The technique is known however to create specimens with a non-uniformity in their void ratio throughout the length with variation ranging up to 10% in either direction (Vaid, Sivathayalan and Stedman, 1999). However, this technique was used for preparing the samples of sand and fines mix mostly to prevent segregation of the two different materials and the mixture obtained through it was found to be more uniform and consistent as compared to any other technique.

#### 5.2.4. Slurry deposition

To ensure full saturation, the sand sample is boiled the night before testing, similar to the WS technique. Rather than depositing the soil directly into the split mold, it was first placed into a mixing container, where it was thoroughly mixed before being deposited into the mold. The soil and mixing container is inserted into the mold and then the mixing container is extracted leaving the soil in place. According to Vaid et al. (1988), the slurry deposition method has the following advantages:

1. The method produces loose to dense samples observed commonly among the density range of in-situ sands.

2. The sample saturation is easier as compared to others.
3. The fabric and void ratio of the samples are more or less uniform throughout.
4. The particle segregation is minimum regardless of gradation or fines content.
5. The method is closest to simulating the natural ways of deposition of soils and easy to duplicate.

#### 5.2.5. Undercompaction

Undercompaction is the soil compaction technique developed by Yamamuro and Lade (1998) in order to create soil samples which do not segregate when a soil mixture containing fines exist. It is claimed to achieve soil samples with uniform relative densities. The technique was developed for soil samples to undergo cyclic triaxial tests specifically for liquefaction study. The technique was different from moist tamping in one particular aspect that the tamping of the subsequent upper layer does not further compact the layers below it. Thus, each layer is typically compacted in lower densities thus overcoming the non-uniformity of compaction caused due to tamping processed where the density of the lower material increases due to compaction blows at the upper layers.

### 5.3. Sample Installation

Before the commencement of the consolidated undrained test, the membrane has placed the sides of the triaxial cylinder was lubricated with silicone gel to make sure the cell pressure and backpressure do not get mixed. The top and bottom platen were lubricated with sufficient Vaseline or silicone grease to prevent friction between the membrane and the platens. This friction leads to non-uniform distribution of stresses in the soil samples as it introduces non-desired stresses and affects the angle at which resultant principal stress is applied. Such effect due to the friction is called as end restrains. It is important to make sure that the stress distribution throughout the system is uniform so that effective stresses across the material being tested will be reasonably uniform and meaningful test results can be obtained. A vacuum pump is activated to make sure the membrane achieves the entire dimensions of the split mold due to suction. The porous stoned completely saturated along with wetted filter paper is introduced at the bottom platen. Soil is later introduced in the system according to the method of sample preparation chosen. Once the soil is completely filled, another saturated porous stone and filter paper is introduced at top of the soil layer and a lubricated top platen is placed.

In order to measure the pore water pressure inside the sample at various heights, 1.4" long hollow needles are penetrated within the samples at the desired heights from the outside of the membrane. These needles were punctured in the soil specimen at any desired height at which the readings need to be made. To prevent leakage through the puncture the needles at its tail were connected to a ¼" tube which was connected to the modified opening made to the top cap of the system. It should be noted that all the efforts were made so that cell pressure does not influx in the sample as that would lead to the failure of the test before any external axial load is applied. However, when the samples were initially made and tests were run, it was observed that the process of sample preparation and placement of the top cap to the cylindrical triaxial chamber caused disturbance and displacement of the needles which led to the puncture in the membrane to increase in size thus leading to the soil sample to be inundated with water in the pressure chamber thus creating leakage problems. Thus, it was necessary to make sure that the needles remain undisturbed when the triaxial chamber is prepared and the top cap is placed. The initial runs showed that the needles caused leaks in the membrane due to constant disturbance created while setting up the sample. Thus, modifications were made in the preparation of the samples. A special base was designed and 3 D printed to make sure the needles were not disturbed when the triaxial chamber was put in place. The design of the base is provided in the twin tower section. The holes keep the needles in place and caused minimum disturbance thus reducing leakage problems considerably. These membrane punctures created by the needles are covered with watertight tape to prevent the water from entering the soil through the gap created. These needles are further connected to tubes that come out from the top cap of the triaxial cell with the help of specially made grooves that are sealed with water sealant and O-rings in order to maintain the cell pressure and prevent leakage of water and pressure.

The pore pressure and cell pressure transducers were calibrated to make sure accurate values have been obtained. The cell pressure is first introduced as a seating pressure to ensure that there is no leakage within the soil membrane. And the cell is filled with water as a medium for applying the sigma 3 on the soil sample. The soil is back pressured using the flushing method where Carbon Dioxide gas is passed through the soil sample to get rid of the air trapped between the voids The soil is then back pressured with water to achieve saturation, In many cases, the air is flushed out by passing carbon dioxide gas through the system and then passing the de-aired water through it. This is a better procedure as carbon dioxide is lighter than air and is easily flushed out from the system. The water in the tank is de-aired for at least a period of 24 hours to prevent the influx of extra gases from the outside. Another added advantage is the prevention of the possibility of chemical reactions that could occur in the soils which might affect the results. This is especially important for long-term tests in order to keep the influence of the pore fluid purely mechanical. A small pressure gradient is applied between the bottom (inlet) and the top pore pressure circuit (outlet) by leaving the exit valve open. The backpressure is typically increased in several stages on both specimen faces and is maintained for several hours to days. The confining pressure is increased simultaneously in such a way as to maintain the effective stress that has been established during the flushing phase.

### 5.4. Saturation

The soil was saturated using the Trautwein pressure panel to achieve a B-value of 0.95 and was usually achieved at around 30-35 psi (Figure 23). A B-value test was run on the sample to make sure that the change in pore water pressure on a change of at least 5 psi of cell pressure is 0.95 times the value of cell pressure.

$$B = \frac{\Delta u}{\Delta \sigma 3} \tag{7}$$

where:

$\Delta$ u = Change in the PWP of the specimen as a result of cell pressure change, and

$\Delta$ σ3 = Change in cell pressure.

Once the saturation of the soil was completed the soil would later be consolidated by opening he valves and allowing the soil to consolidate for up to 30 minutes minimum as recommended by ASTM D4767-11.

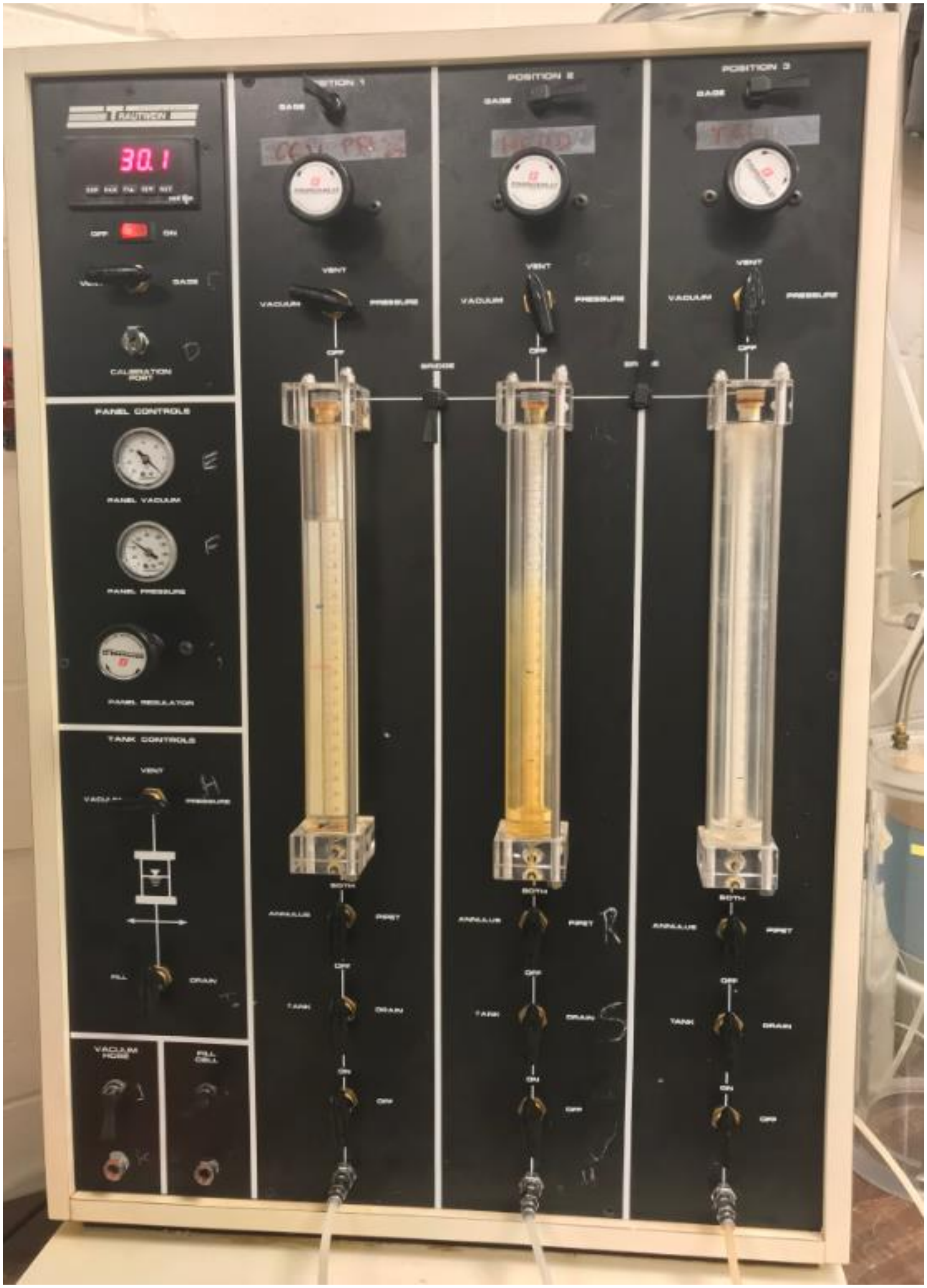



**Figure 23. Trautwein Pressure Panel**

# CHAPTER 6. TEST RESULTS AND DISCUSSION

## 6.1. Behavior of Soils with the Addition of Silty Fines

### 6.1.1. Responses at 15-18% relative density

The relative density of the soils was kept nearly constant throughout the tests. At a constant relative density, the fines were added to the soils, which led to an increase in their skeletal void ratio. The soils were tested at 15%. Results are provided in Table 3.

**Table 3. Loose Sand with the Addition of Silty Fines**

| Sample | Relative Density | Type of Fine | % Fines | $e_{sk}$ | Initial Confining Pressure |
|---|---|---|---|---|---|
| 29 | 15 | None | 0 | 0.78 | 50 |
| 70 | 15 | Loess | 10 | 0.94 | 50 |
| 53 | 18 | Loess | 20 | 1.50 | 50 |
| 58 | 14 | Loess | 30 | 1.21 | 50 |

For the first series of tests loose sands, Ottawa sands were mixed with silt fines at 0%, 10%, 20%, and 30%. The results for deviator stress vs strain % of 4 undrained monotonic triaxial tests are shown in Figure 24. The soils have been tested at a constant relative density of approximately 15%. The purpose was to verify the effect of fines content at moderately high confining stress of p’ = 50 psi or (345 KPa) on the behavior of loose sands. The clean sand exhibited a highly dilative response when sheared as it usually expected from loose sands under shear Holtz (1981). The soil showed a dilative behavior also called flow type by Ishihara (1993) where the soil behavior is unstable where it achieves a maximum shear strength then undergoes dilation on achieving optimum void ratio and the strength declines steadily. However, when the sand was mixed with silts it showed a more rapid hardening

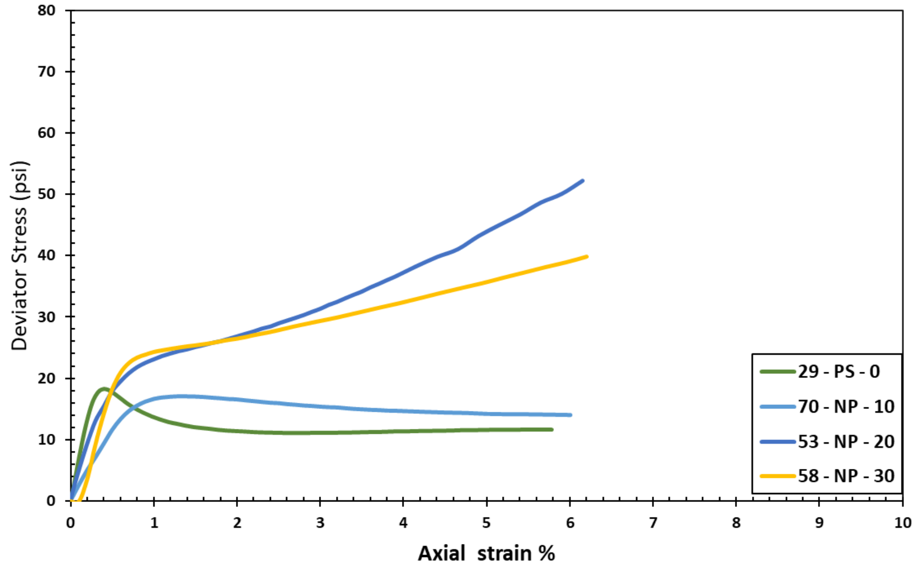



**Figure 24. Deviator Stress vs Axial Strain (%) for loose sand with silty fines**

behavior where the soil showed signs of contraction. As the number of silt fines increased to 10% rapid hardening response was observed within the soil and without affecting the stability of the soil. However, as the fines content was increased within the host sand, the soil mixture underwent contraction with the reduction in the residual strength observed. It was observed that at fines content up to 10%, the soil silt mixture showed a rapid reduction in the behavior with respect to the change in the dilation to undergo contraction without the loss in hardening properties of the soil. Similar behavior was observed for the soils with fines content up to 20%. For the soil with 30% fines, the relative density or the void ratio of the host sand before the introduction of the fines was rescued to 5%. For the same amount of fines but with a lower void ratio or relative density and at the same confining pressure of 50 psi, the soils were observed to change their behavior with the increase in fines. The dilation tendency is steadily reduced and the soil starts to undergo contraction. This phenomenon occurs due to the increase in the silts in the voids and they participate in the distribution of loads for the sand grains.

The p-q' curve (Figure 25) revealed that the sands with the addition of silts undergo an increase in its tendency to contract and try to achieve a temporary instability phase from temporary liquefaction. It was observed that the mean effective stress at which the soil achieves temporary increases thus making it more stable with and provided resistance towards possible liquefaction. The phase transformation stage is the point at which the curve achieves its minima after which it undergoes an increase in deviator stress or a quasi-steady state is achieved at a higher mean effective stress with an increase of the fines content. This follows the observation of the behavior of excess pore water pressure generation within the soils during its shear stage.

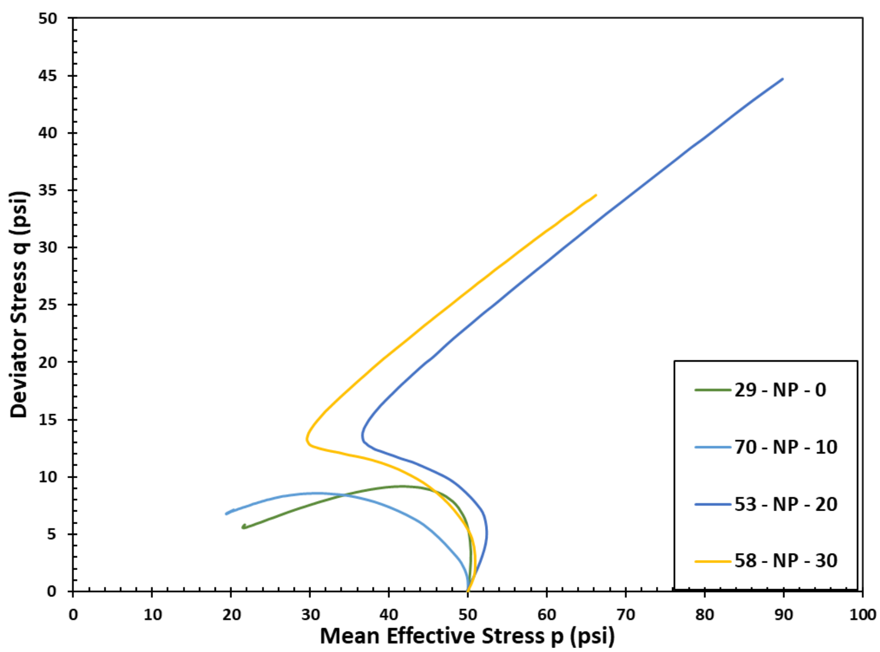



**Figure 25. Effective stress paths (p' vs q) for loose sands with silty fines**

With the increase in fines contents the behavior, it was also observed that the increase in the silt content in the sand increased the generation of excess pore water pressure buildup within the soil specimen changes its behavior from a more loose soil towards medium dense sand. As shown in Figure 26, the excess PWP generated curves behave like loose sand in the absence of silts. The increase in the silt content and in the case of loose sands the PWP increased to a maximum and then kept steady state as the soil achieved a steady state. The increase PWP curves on increasing the silt content in the soil tend to behave similarly to medium and dense sands.

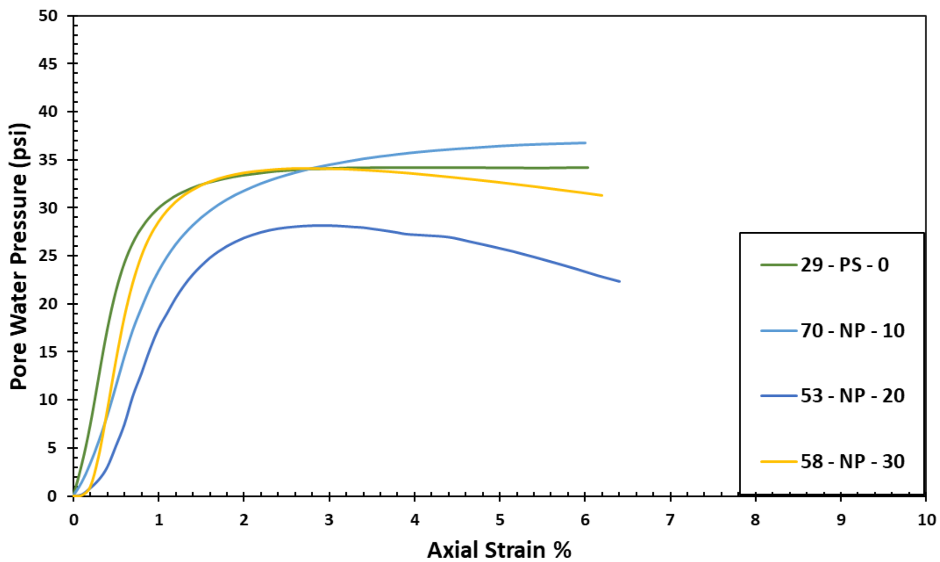



**Figure 26. Excess PWP vs Strain (%) for loose sands with silty fines**

Why does the silty sand mix behave differently than pure sands? When clean sand is mixed with increasing amounts of non-plastic fines, the minimum, and maximum void ratios, as well as the range of void ratios, change, and highly unstable and compressible particle structures are formed when gently deposited into loose configurations. The void spaces between the larger grains are relatively unoccupied, and the larger grains, which will make up the load-bearing skeleton, are held slightly apart by smaller silt particles. On the application of stress, these silt particles are forced into the void spaces (Figure 27). This leads to a metastable structure formation which was hypothesized by Terzaghi. It was also observed that the increase in the fines increases the angle of the steady-state line without affecting the point of phase transformation.

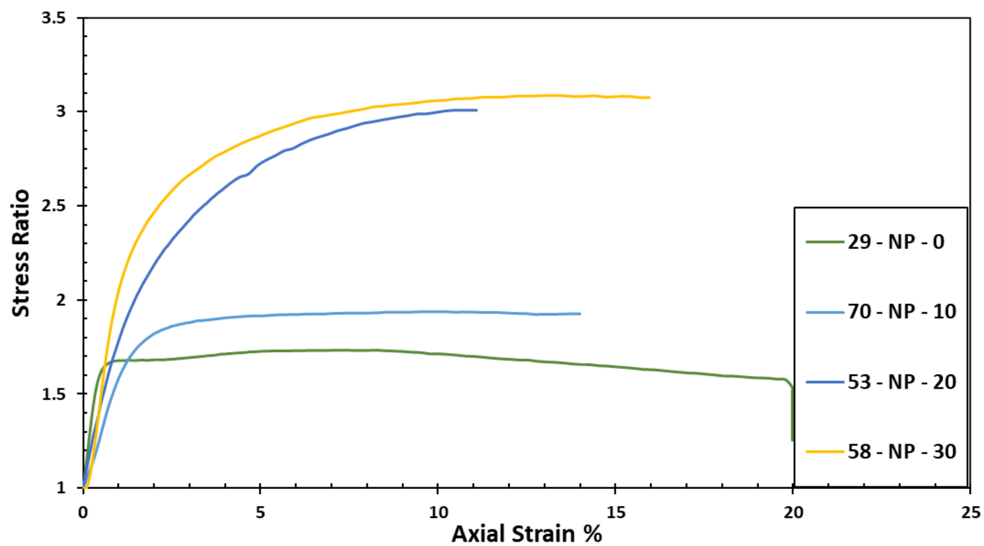



**Figure 27. Stress Ratio vs Axial Strain (%) for loose sands with silty fines**

### 6.1.2. Response at 30-60% relative density

Tests for this series were executed on medium dense to dense sands (Table 4). Medium dense Ottawa sands were mixed with silt fines at 0%, 18%, 28%, and 40%, whereas dense sands were mixed with 12%, 28%, and 40%. The results for deviator stress vs strain % of 7 undrained monotonic triaxial tests are shown in Figure 28. The soils were tested at different relative densities approximately ranging from 30% to 60% and variation of behavior in the addition of fines was observed. The purpose was to verify the effect of fines content at moderately high confining stress of p’ = 50 psi or (345 KPa) on the behavior of sands.

**Table 4. Medium Dense and Dense Sand**

| Sample | Relative Density | Type of Fine | % of Fines (fc) | $e_{sk}$ | Initial Confining Pressure |
|---|---|---|---|---|---|
| 33 | 30 | None | 0 | 0.64 | 50 |
| 57 | 60 | Silty | 12 | 1.07 | 50 |
| 84 | 30 | Silty | 18 | 1.3 | 50 |
| 86 | 30 | Silty | 28 | 1.1 | 50 |
| 88 | 60 | Silty | 30 | 1.48 | 50 |
| 98 | 30 | Silty | 40 | 0.91 | 50 |
| 61 | 60 | Silty | 35 | 1.4 | 50 |

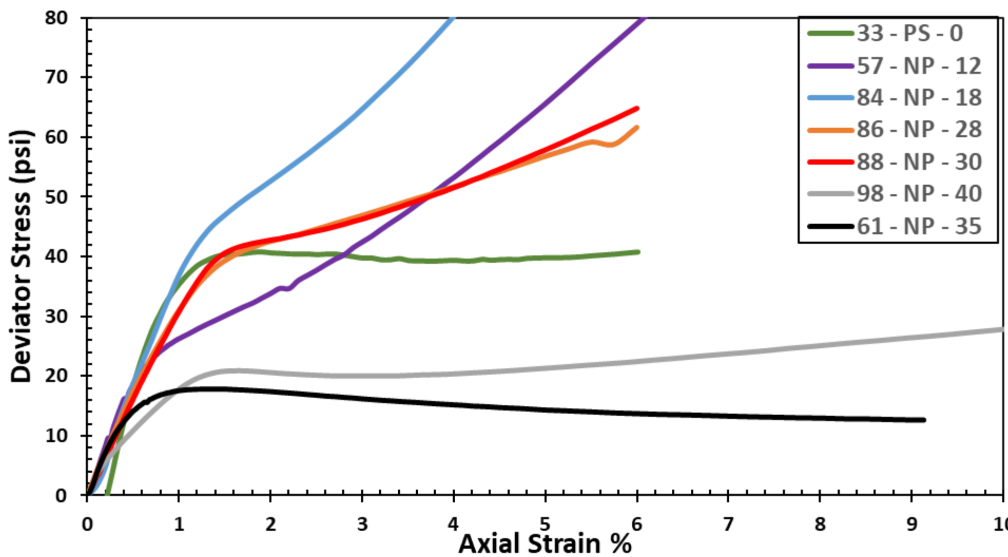



**Figure 28. Deviator Stress vs Axial Strain (%) for medium dense and dense sands with silty fines**

It was observed that the sand revealed a stable behavior of that of medium dense sand when sheared (Figure 29). The sand undergoes dilative behavior and achieves maximum shear strength at a 1.3% strain rate where it undergoes instability up to a particular mean effective stress, and then stabilizes which it traditionally shows in a triaxial test under a

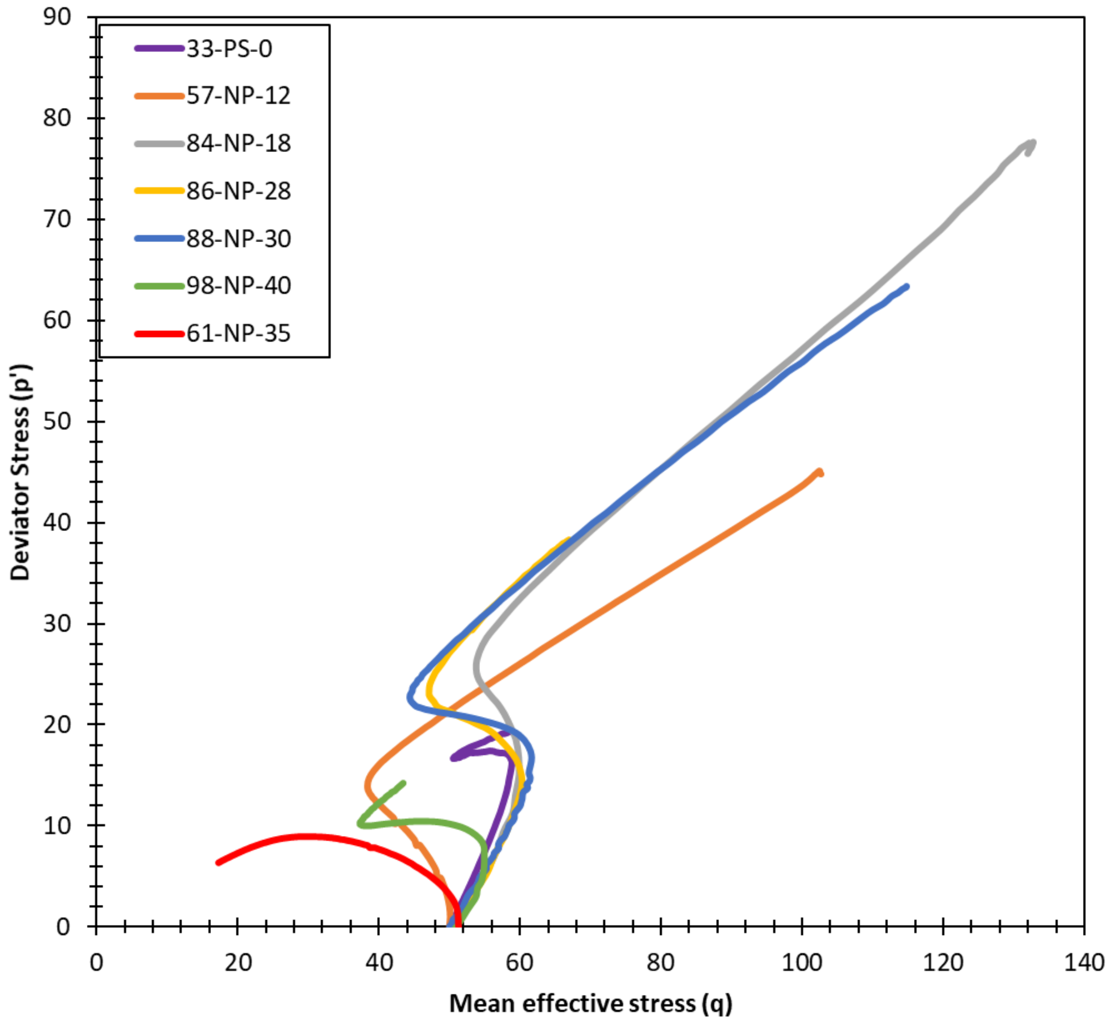



**Figure 29. Effective stress paths (p' vs q) for medium dense sands with silty fines**

medium consolidation pressure in undrained conditions. As seen in the tests for loose sands in section 6.1.1., the sand on the addition of 18% fines undergoes a contractive behavior and becomes more stable.

The excess pore water pressure which in the case of sand shown in Figure 30 decreases with the increase in silt content. However, with the addition of additional silt of 28%, dilation is observed as similar to loose sand with 30% silt content. The pore water pressure increases more steadily and the sands become less stable. As the silt content in the sands is further increased up to 40%, the sand shows dilative behavior again as was observed

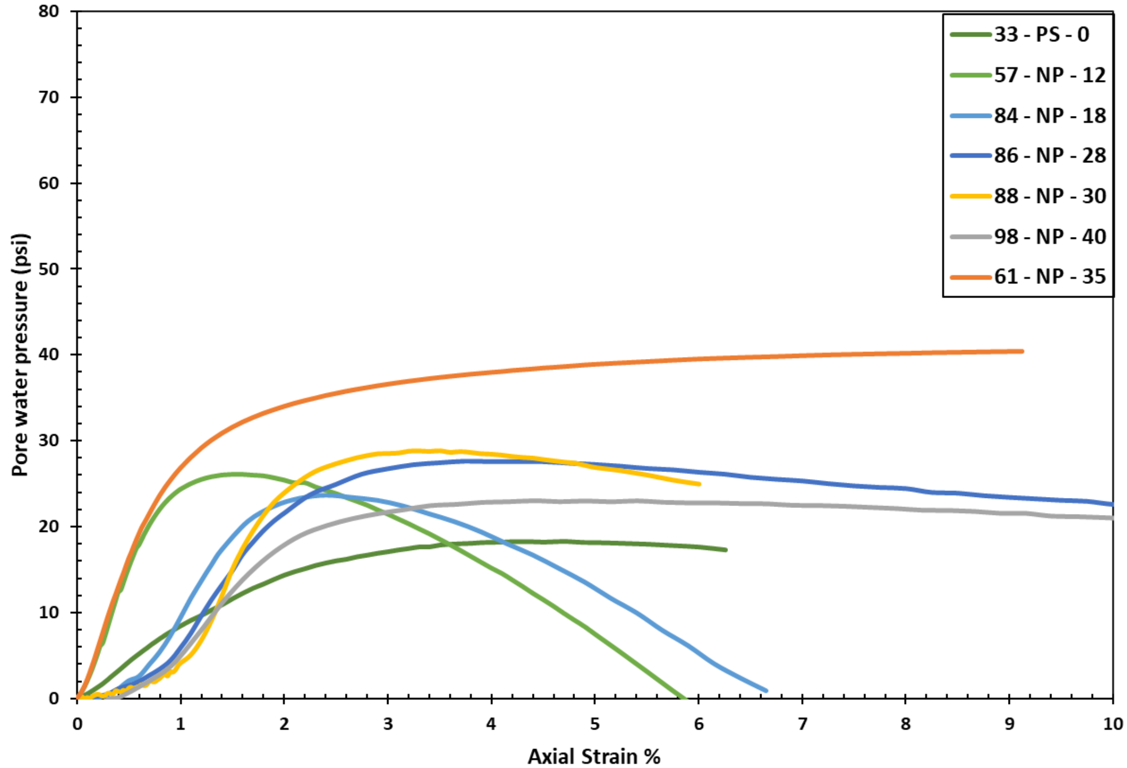



**Figure 30. Excess PWP vs Axial Strain (%) for medium dense sands with silty fines**

with loose sand. Similar behavior was observed for dense sands. In addition to 12% silt fines, the sand shows contractive behavior with a loss in pore water pressure and an increase in its stability. As the silt content increases. However, the increase of fines content reduces the rate of generation of negative PWP. As the silt content of the soil increases further, the excess PWP increases and stays positive throughout the test. The addition of fines into the sand increases the dilative dilatant in nature. Therefore, the relative density of sand on the addition of fines is an important factor determining its behavior. At a particular stage where the fines achieve the threshold limit, the fines start to dominate the behavior of the sand with silt mixture. The silt fines no longer fill only the voids but are responsible for the distribution of loads, even between the sand particle, thus explaining the change in behavior of the sands on the addition of fines content.

## 6.2. Behavior after Addition of Plastic Fines

### 6.2.1. Response at 12% relative density

For this series of tests on loose sands, Ottawa sands were mixed with kaolin fines at 0%, 5%, 10%, 13%, 20%, 24%, and 31% (Table 5). The results for deviator stress vs strain (%) of 7 undrained monotonic triaxial tests are shown in Figure 31. The soils have been tested at a constant relative density of approximately 12% and pure sand at the relative density of 25%. The purpose was to verify the effect of fines content at moderately high confining stress of p' = 50 psi or (345 KPa) on the behavior of sands.

**Table 5. Loose Sand Samples with Plastic Fines**

| Sample | Relative density | Type of fine | % of Fines (fc) | $e_{sk}$ | Initial Confining Pressure |
|---|---|---|---|---|---|
| 42 | 25 | Plastic | 5 | 0.86 | 50 |
| 66 | 12 | Plastic | 13 | 1.09 | 50 |
| 69 | 10 | Plastic | 20 | 1.04 | 50 |
| 65 | 10 | None | 0 | 0.78 | 50 |
| 74 | 12 | Plastic | 24 | 1.3 | 50 |
| 78 | 12 | Plastic | 31 | 1.58 | 50 |
| 81 | 12 | Plastic | 10 | 0.99 | 50 |

As shown in Figure 31, when the clean sand is sheared the sand shows a strain-hardening behavior where the shear stress increases with the increase in strain up to a particular point after which it achieves a steady state. This behavior of the sand is called the non-flow type behavior as defined by (Ishihara 1993) where the sand undergoes dilation. It can also be seen from the excess PWP vs strain % curve Figure 32 that the PWP in

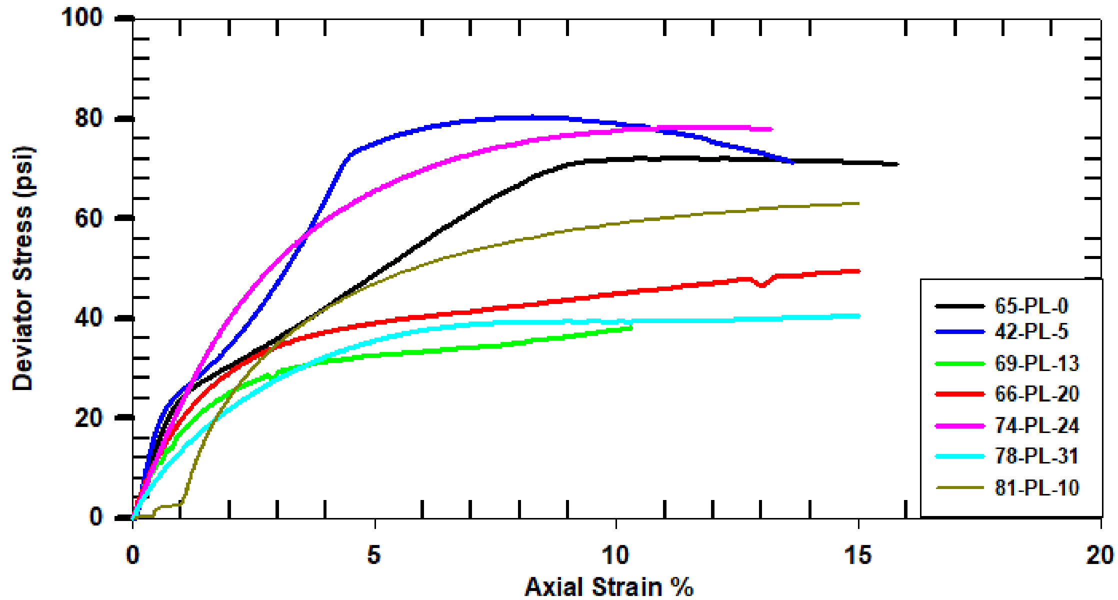



**Figure 31. Deviator Stress vs Axial Strain (%) for loose sands with Kaolin fines**

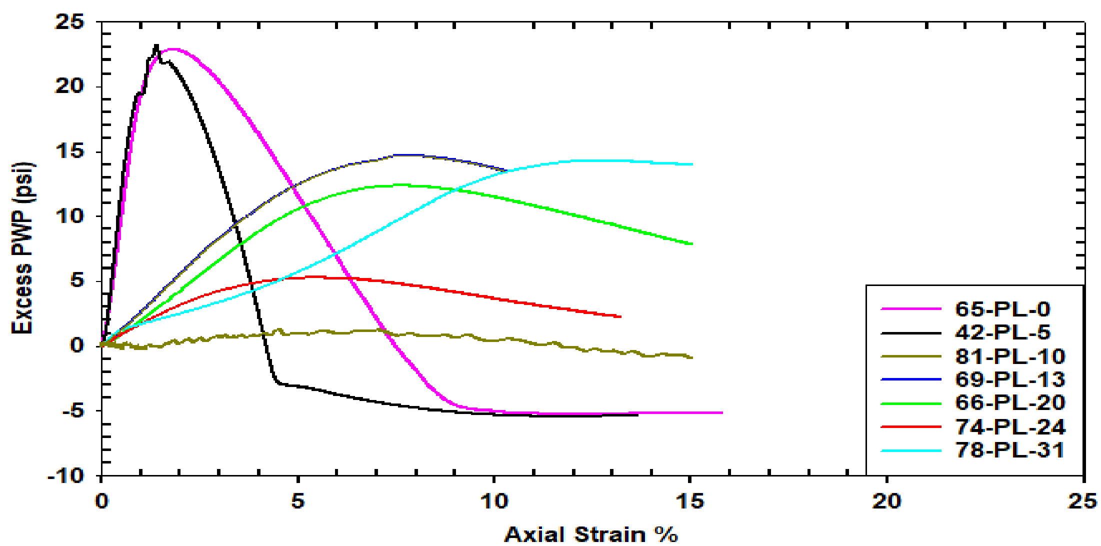



**Figure 32. Excess PWP vs Axial Strain (%) with loose sands with plastic fines**

the soil undergoes rapid increase and then a decrease after which the PWP becomes negative. The sand under such condition is not completely stable and undergoes a phase change at a relatively lower mean effective stress as shown in Figure 33. The sand is confined at a constant initial confining pressure of 50 psi then sheared. The sand is then

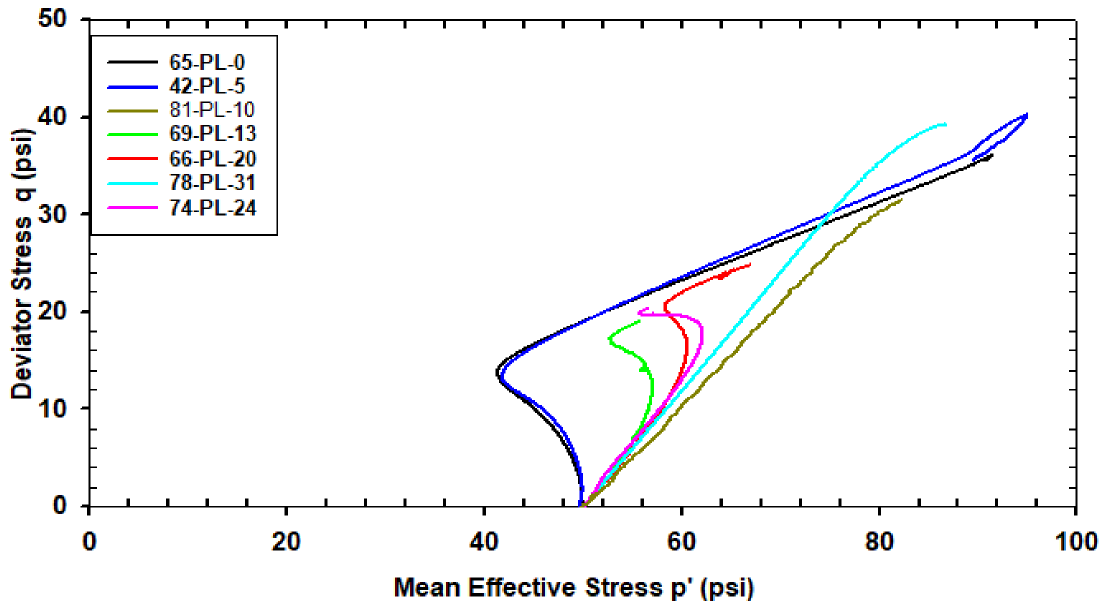



**Figure 33. Effective stress paths (p' vs q) for loose sands with plastic fines**

introduced with plastic fines up to 5% and sheared under the same conditions. The introduction of the plastic fines in such a small amount does not affect the behavior of the soil too much.

The behavior of the pore water pressure can also be seen to behave much like the pure sand on the addition of such small quantities of fines; however, the shear strength of the soil is higher than that of pure sand due to the higher relative density. Past studies have observed that the presence of plastic fines (Georgiannou et al., 1990; Ni et al., 2004) in the sand does not help the strength of the soil but mostly decreases it instead. In sand, the presence of plastic fines present in between the coarse grains encourages rolling and due to the size disparity undergo compression. Hence, the fines present between the voids no longer act as voids but act worse than voids. That is, they affect the strength of the soil negatively and lead to a decrease in the shear strength of the soils. The stability of the soil from Figure 33 does not change and the soil phase transformation point moves only very slightly.

With the addition of 10% fines to the sample with the same relative density, the soils start behaving less dilative and more contractive. The PWP of the soil increases and is no longer negative while the soil shows more contractive behavior. The shear strength of the soil is considerably lower than that of the pure host sand at similar void ratio/ relative density. The pore water pressure also increases with the increase of the fines content. It can be noted that the presence of plastic fines in the voids prevents the easy dissipation of pressure between the voids and hence leads to an increase in the pore water pressure.

This further leads to the contractive behavior of the soil where the fines undergo contractive behavior due to the plastic nature of the kaolin fines which prevent them from dilating. The platey shape of kaolin fines, as opposed to the rotund shape of the loess fines, make it easier for them to adjust their position and get out of the force-carrying skeleton in clayey sand. With the addition of both 20% and 24 % fines, it can be seen that the behavior of the soil behaves similarly; however, the maximum shear strength of the soil decreases on further addition fines. The mean effective stress at which phase transformation occurs increases for the 10%, 20% and 24% fines with the addition of fines (Figure 34). The instability behavior of the soil decreases and its resistance towards liquefaction potential increases. As shown in Figure 33, the excess PWP behavior of the soil increases drastically till up to 1.5% strain rate and then decreases as soil reaches a steady state.

As shown in Figure 31, for soil samples with the addition of both 24% and 30% sand, the soils have comparatively lower maximum shear strength and they achieve a contractive behavior with respect to the sands. Figure 34 indicates they have achieved a stable stress path. They do not undergo any phase transformation and have a straight stress path which shows that they have a stable behavior when compared to the sands with lower fines.

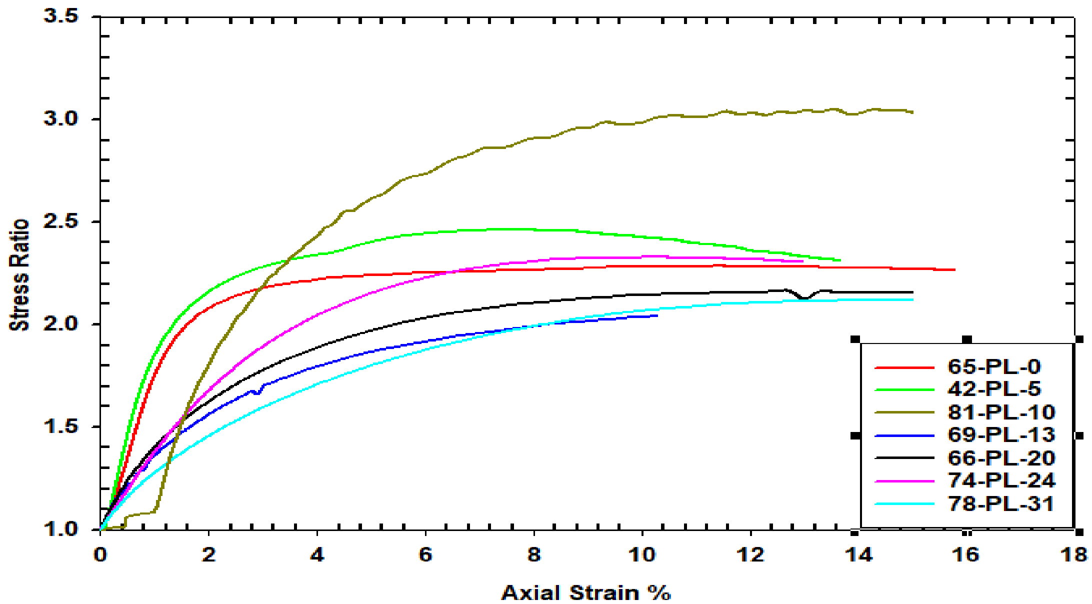



**Figure 32. Stress ratio vs strain (%)**

**6.2.2. Response at 40-50% relative density**

For this series of tests on loose sands, Ottawa sands were mixed with kaolin fines at 0%, 8%, 17%, and 30% (Table 6). The results for deviator stress vs strain (%) of 4 undrained monotonic triaxial tests are shown in Figure 35. The soils have been tested at a constant relative density of between 40-50 %. The purpose was to verify the effect of fines content at moderately high confining stress of p' = 50 psi or (345 KPa) on the behavior of sands.

Medium sand of 50% relative density was consolidated at 50 psi pressure and sheared. The sand shows contractive behavior where the soil undergoes strain-softening behavior and where it has a point of highest shear strength after which there is a gradual decline in strength. In this case, it can be seen from Figure 36 that the soil shows a very small instability behavior but is stable in most cases. With the addition of the plastic fines, the soils simply undergo a similar deformation and similar behavior without undergoing any instability (Figure 36 – 38). The only difference observed in all the soils after the addition of

**Table 6. Medium Dense Sand with Kaolin Fines**

| Sample | Relative Density | Type of Fine | % of Fines (fc) | $e_{sk}$ | Initial Confining Pressure |
|---|---|---|---|---|---|
| 63 | 50 | Plastic | 0 | 0.64 | 50 |
| 25 | 45 | Plastic | 8 | 0.82 | 50 |
| 44 | 45 | Plastic | 17 | 0.70 | 50 |
| 59 | 40 | Plastic | 30 | 1.25 | 50 |

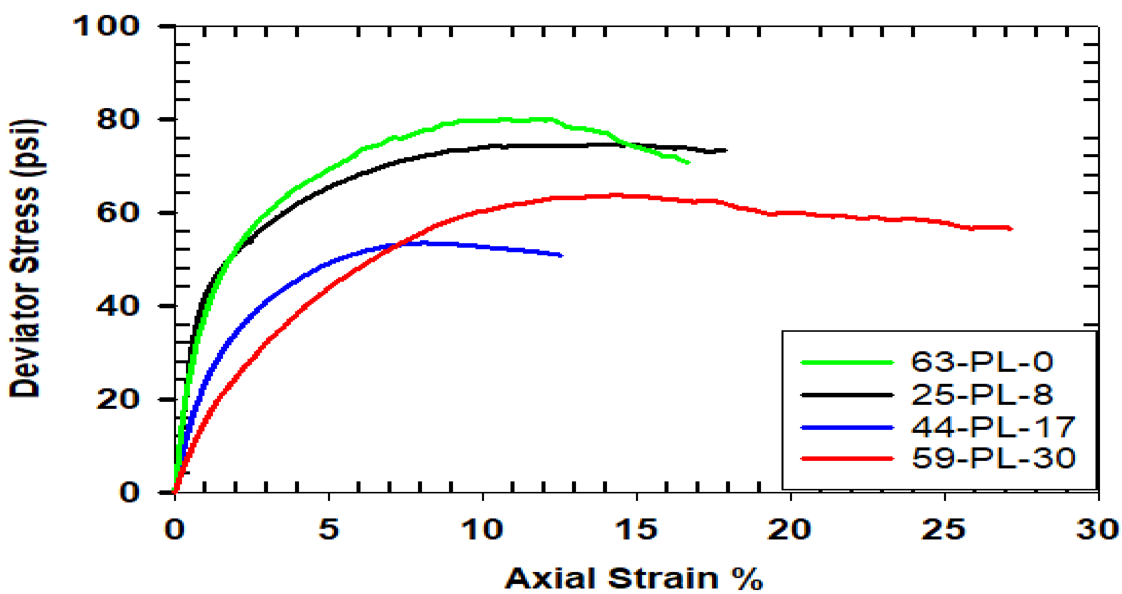



**Figure 33. Deviator Stress (psi) vs Axial Strain (%) for medium dense sands with plastic fines**

kaolin fines is the gradual decline in the value of maximum shear strength but there is no change in their behavior when compared to the pure sand. There is no observation in any change to the steady-state line either, and the stability is also retained. Therefore, it can be shown that, in the case of medium sands, the presence of kaolin does not affect the stability in any way but only affects its strength and does not promote or prevent liquefaction or instability.

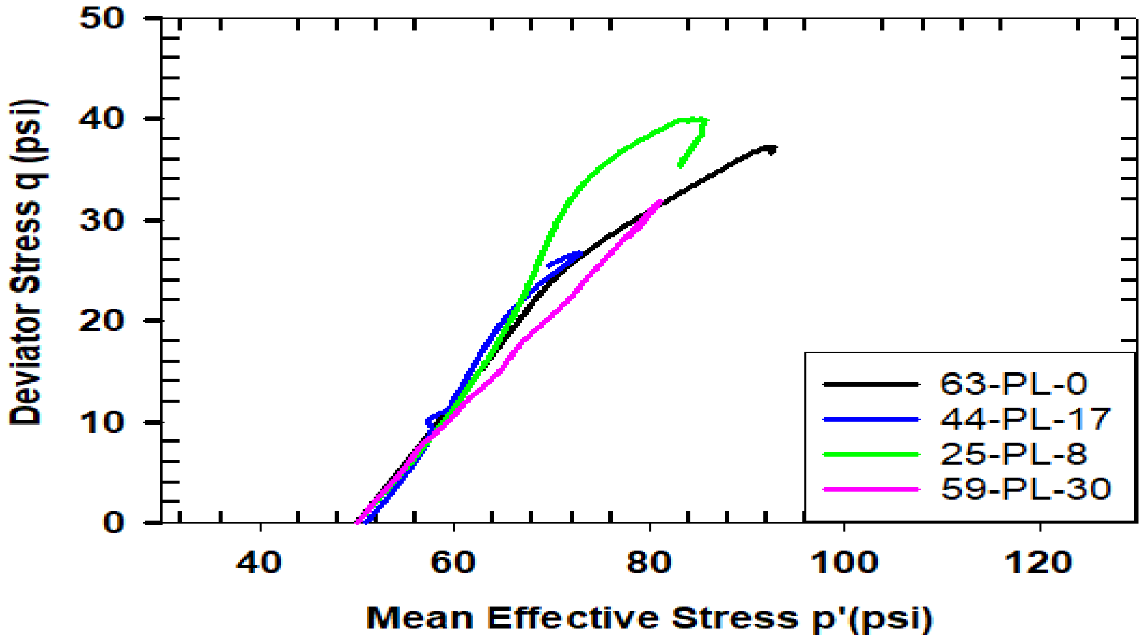



**Figure 34. Effective stress paths (p' vs q) for medium dense sands with plastic fines**

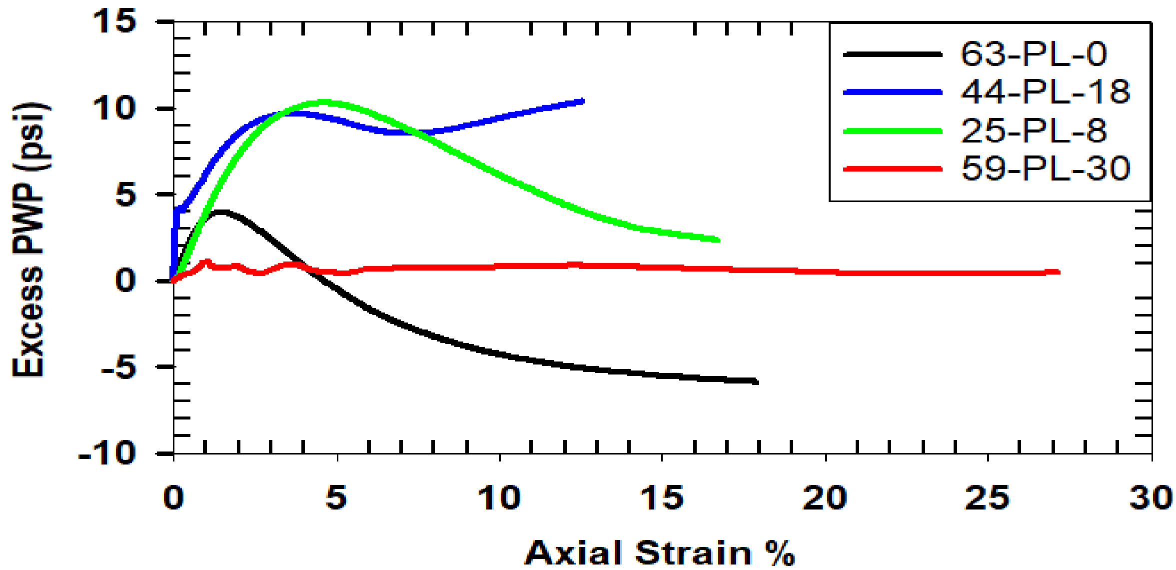



**Figure 35. Excess PWP vs Axial Strains (%) for medium dense sands with plastic fines**

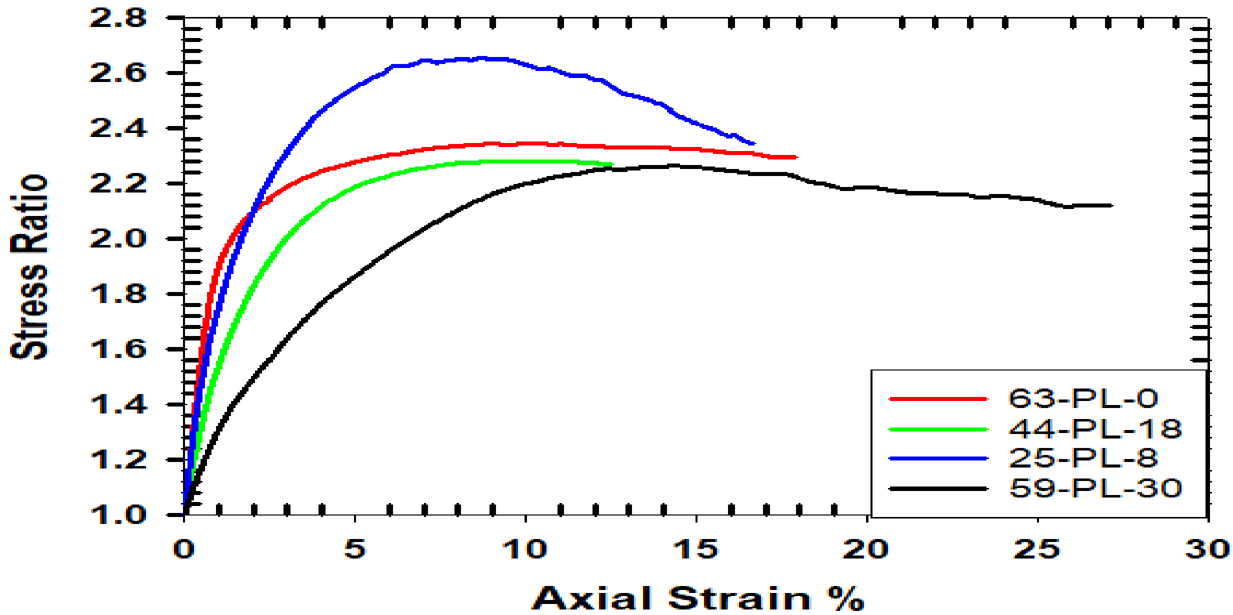



**Figure 38. Stress ratio vs axial strain (%) for medium dense sands with plastic fines**

**6.2.3. Determination of range of threshold fines content of silts in Ottawa sands**

The shear strength of various samples of sand compacted at different relative densities of compaction is shown in Figure 39. The natural behavior of the increase in fines content to the sand at a constant relative index is shown in the preceding graphs. It can be observed that for all the soil samples there is an increase in shear strength in the soils with the increase in the fines content. As the amount of fines in the soil increase, the sand matrix densifies as the fines replace the voids and the transfer of forces due to interparticle contact increase which leads to the increase in the shear strength of the soil sample. This is clearly observed in all the samples consolidated at different relative densities at fine content (%) equaling 10%. As the fines content increase further, the shear strength of the soil increases considerably. At a particular point in the increase of fines to the host sand, however, the soil reaches a shear strength and then drops. This is assumed to be the threshold fines content as theorized.

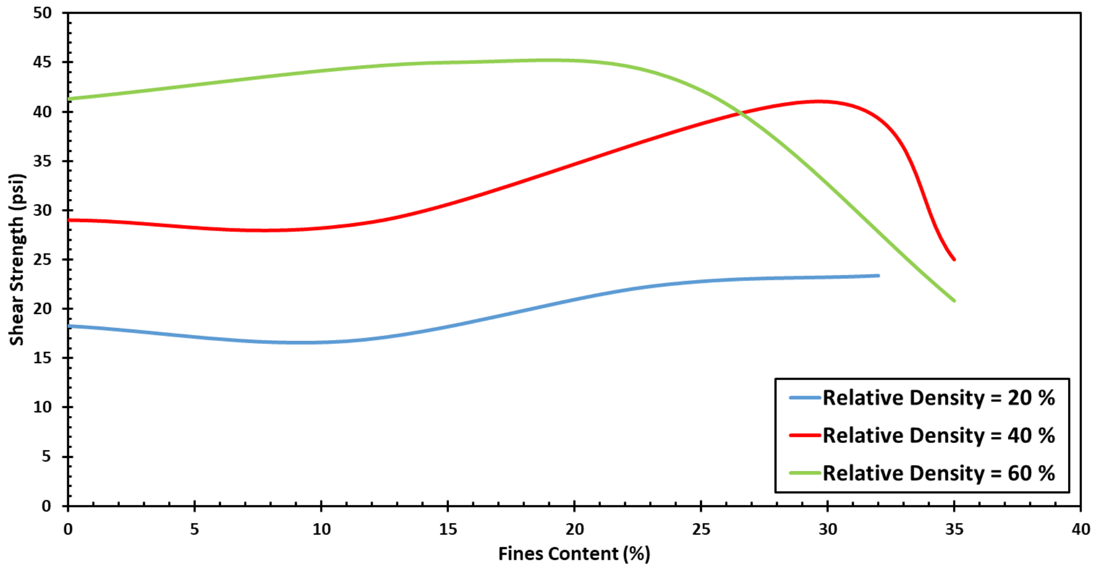



**Figure 36. Fines Content (%) vs Shear Strength (psi) at different relative densities in sand**

At the threshold fines content the fines overcome the voids and participate actively in the stress distribution. This causes in case of silt a drop in the shear strength due to slipping between the sand grains and the silt fines. It can be observed from the graph that the threshold fines content is achieved from 23 % to 35% for dense sands to medium dense sands. The loose sands do not show the behavior for fines content up to 35%.

### 6.2.4. Confining pressure effect on maximum stress at failure in very loose sands

For this series of tests on very loose sands, Ottawa sands were mixed with silt fines at 35% and 40% (Table 7). The results for deviator stress vs strain % of 3 undrained monotonic triaxial tests are shown in Figure 40). The soils have been tested at a constant relative density of approximately 5% at different confining pressures. The purpose was to verify the effect of fines content at different high confining stress in the presence of loose sands and high plastic fines of p' = 50 psi or (345 KPa), 90 psi or (620.5 KPa), 103 psi or (710.1 KPa) on the behavior of sands.

**Table 7. Sand at Various Initial Confining Pressures**

| Sample | Relative Density | Type of Fine | % of Fines (fc) | $e_{sk}$ | Initial Confining Pressure |
|---|---|---|---|---|---|
| 45 | 5 | Plastic | 35 | 1.76 | 50 |
| 47 | 5 | Plastic | 35 | 1.76 | 90 |
| 49 | 3 | Plastic | 40 | 2.03 | 103 |

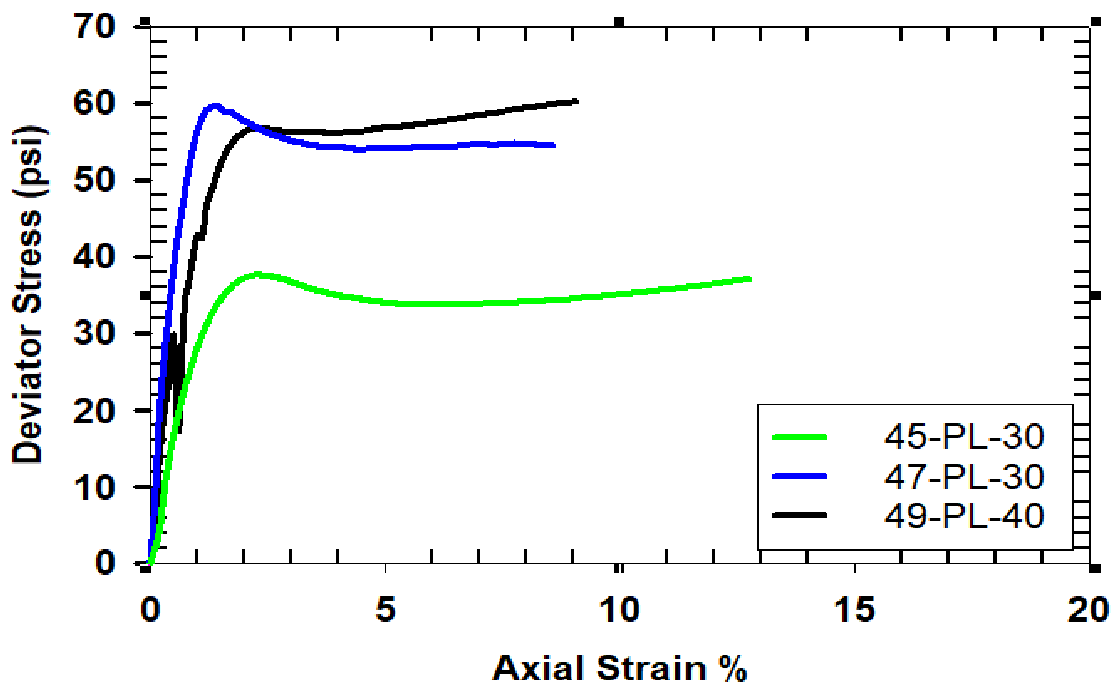



**Figure 40. Deviator Stress vs Axial Strain (%) for very loose sands with plastic fines at different confining stress**

Even though it was shown that the increase in the fines content stabilizes the soil and prevents its potential to liquefy or stabilize in case of plastic fines, at low relative density the soils do undergo instability despite the increase in the initial confining stress. It was proposed by Yamamuro and Lade (1998) that an increase in the initial confining stress increases the stability of the soil, it is the relative density at which the host sand is confined that determines the stability even though in presence of high plastic fines content. For the soils with the increase in the confining pressure, the shear strength at which soil collapses increases along with its initial confining pressure. However, the behavior of the soil does not change and it shows instability despite the presence of high plastic fines. It can also be seen that for the soil with the same relative density and % fines, the phase transformation point or the point at which quasi-steady state is observed does not change on the increase in the confining pressure. However, with the decrease in the relative density and the increase in the fines content in sample 49 is undergoing liquefaction at a lower mean normal stress. As shown in Figure 41, the soil undergoes temporary instability in sample 45 and 47 at different initial confining stresses and undergo a phase transformation at mean effective stress of approximately 35 psi > on the contrary sample 49 despite having a higher fines content and initial confining pressure undergoes instability and liquefaction at a lower value of effective stress of 5 psi.

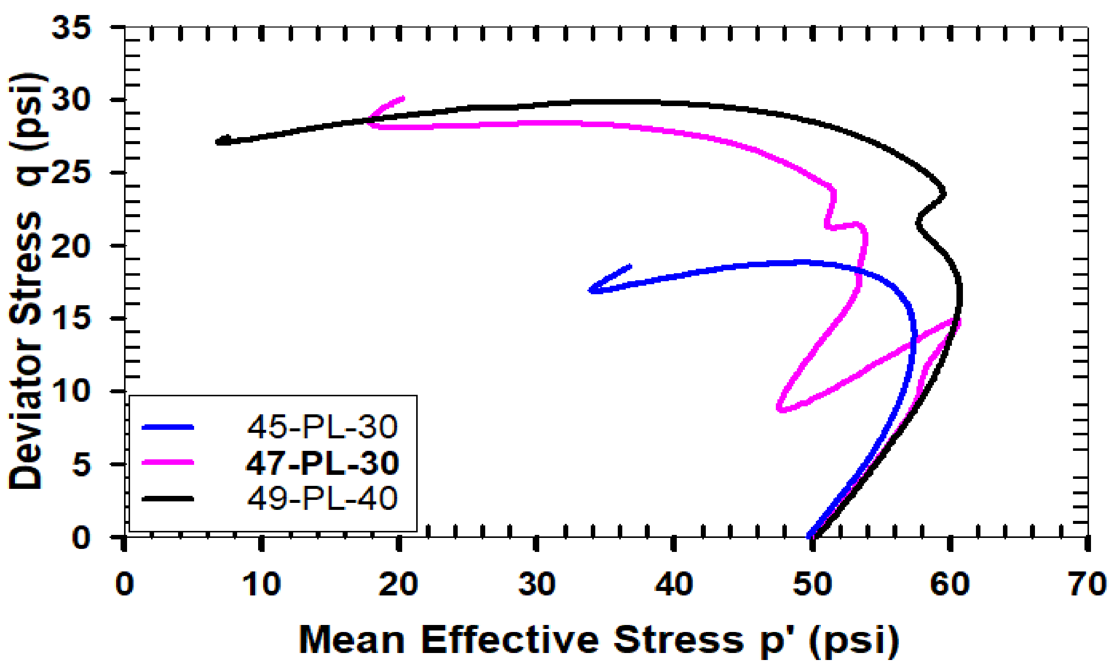



**Figure 37. Effective stress paths (p' vs q) for very loose sands with plastic fines at different confining stress**

### 6.3. Pore Water Pressure Measurement at the Mid-height of the Sample

Conventionally PWP is measured at the bottom and top of the soil sample. The pore water pressure was measured at the mid-height of the sample and the factors affecting it were observed. The test was run on pure sand with two different types of soils, i.e. pure sands and sands with plastic fines and the pore water pressure measurement were carried out at two ends and compared with each other. The test is run on six different samples the effect of relative density, amount of fines were the factors that were compared (Table 8).

**Table 8. Sands with PWP Measured at Two Different Ends**

| Sample | Type of Soil | % of fines | Relative density | $e_{sk}$ | $R^2$ |
|---|---|---|---|---|---|
| 75 | Pure Sand | 0 | 30 | 0.73 | 0.993 |
| 76 | Pure Sand | 0 | 60 | 0.62 | 0.9858 |
| 77 | Sand with Plastic Fines | 20 | 12 | 1.23 | 0.999 |
| 73 | Sand with Plastic fines | 15 | 40 | 0.98 | 0.835 |
| 68 | Sand with Plastic fines | 20 | 40 | 1.10 | 0.8463 |
| 67 | Sand with Plastic Fines | 20 | 75 | 0.56 | 0.328 |

For this series of tests, six Ottawa sand samples were tested with two pore pressure transducers, at the end and at the mid-height of the sample. The samples were made at different relative densities and fines at various quantities were added to it. Pure Ottawa sands were tested at 30 and 60% relative density whereas, sands with Rd = 12%, 40%, and 75% were mixed with kaolin fines at 20% and 15%. PWP vs time graphs for all samples for both PWP at the ends and PWP at mid-height are shown in Figure 42 – 58). The values obtained were best fitted on a regression line to determine how close the values obtained were to one another. Additional results and figures are provided in the Appendix.

It can be observed that, for the pure sand with a low relative density, the values obtained were extremely close to each other since the coefficient of determination was very close to 1. With the increase in the relative density for the sand sheared with all the other factors kept constant, the value of $R^2$ deviates from 1 although not by a large value. Thus, it can be determined that the relative density of sand affects the PWP within the soil however not immensely. With the addition of fines, the behavior of the PWP within the soil sample is significant. On the addition of 20% fines in the case of loose sand, the regression value is considerably close to 1. As the relative density of the sand increases and the number of fines is kept constant, it can be seen that the $R^2$ value drops immensely. Thus, as the soil densifies and the void spaces between the coarse grains decrease, the fines occupy the space in the voids and the porosity of the soil decreases. In addition, the presence of plastic fines makes the soil impermeable and the dissipation of pressure between the soil samples takes longer. It is theorized that if a similar test is executed at a much higher strain rate, the dissipation of PWP within the sample would be non-uniform and would lead to considerably different values for both the pressure transducers. For tests carried out at 0.5-0.8%, the time is permitted for the dissipation of pressure allowing uniformity throughout the length of the sample. For dense soils with 20% fines, as observed in Figure 52, the $R^2$ obtained is considerably lower providing significant proof that even within smaller samples, the pore pressure generated within the soil specimen is non-uniform.

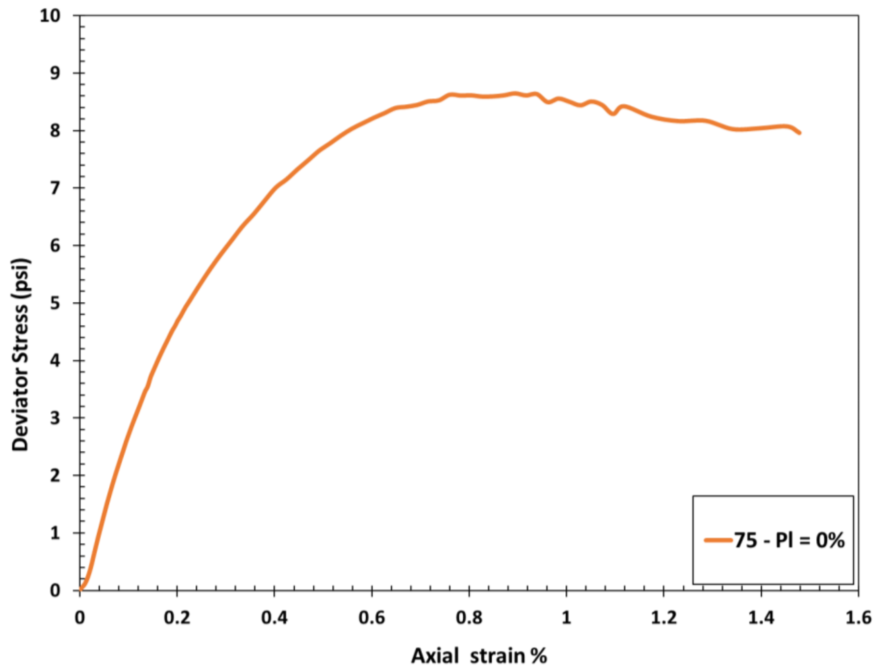



**Figure 38. Deviator Stress (psi) vs Axial Stress % for sample 75**

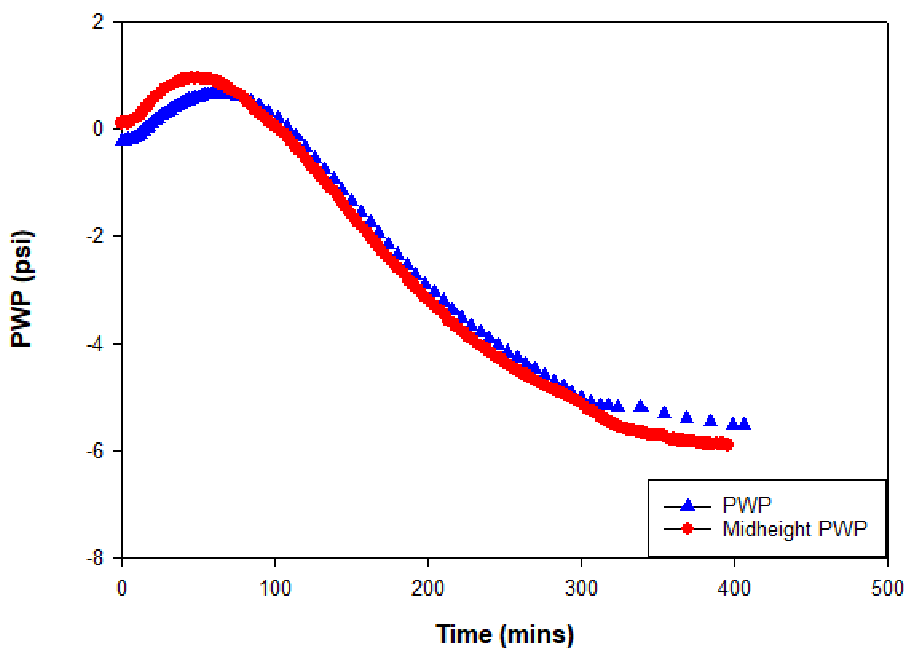



**Figure 39. Comparison between PWP (psi) at ends and mid-height vs time (min) for sample 75**

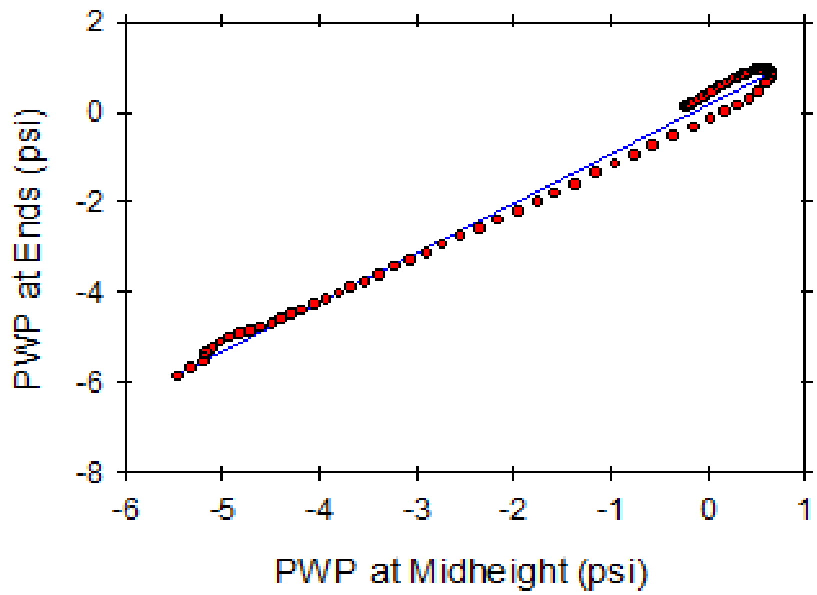



**Figure 40. Variation between PWP at ends and mid-height for sample 75**

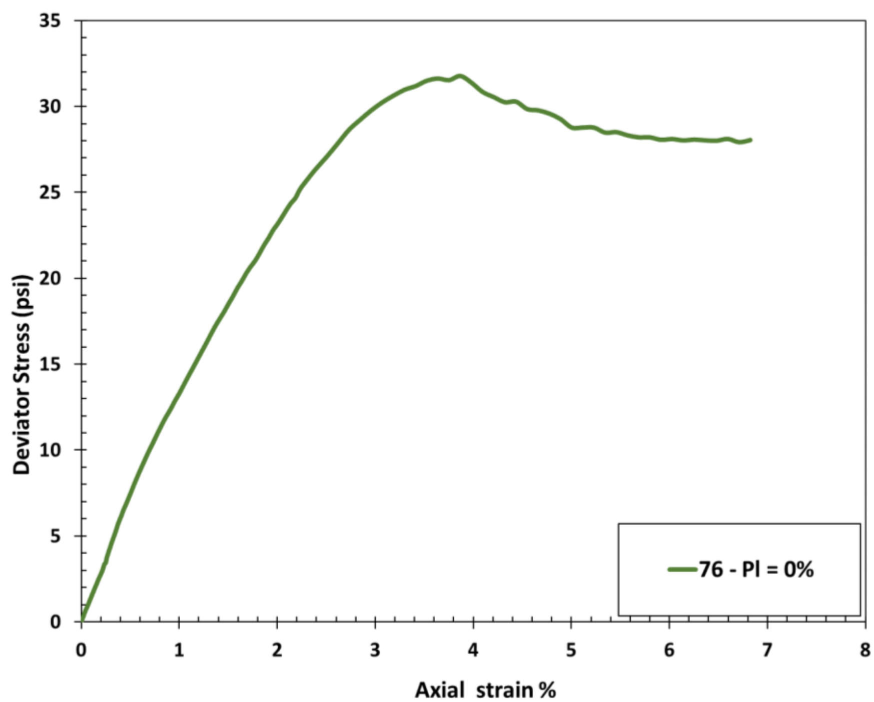



**Figure 41. Deviator Stress (psi) vs Axial Stress % for sample 76 with 0% plastic fines**

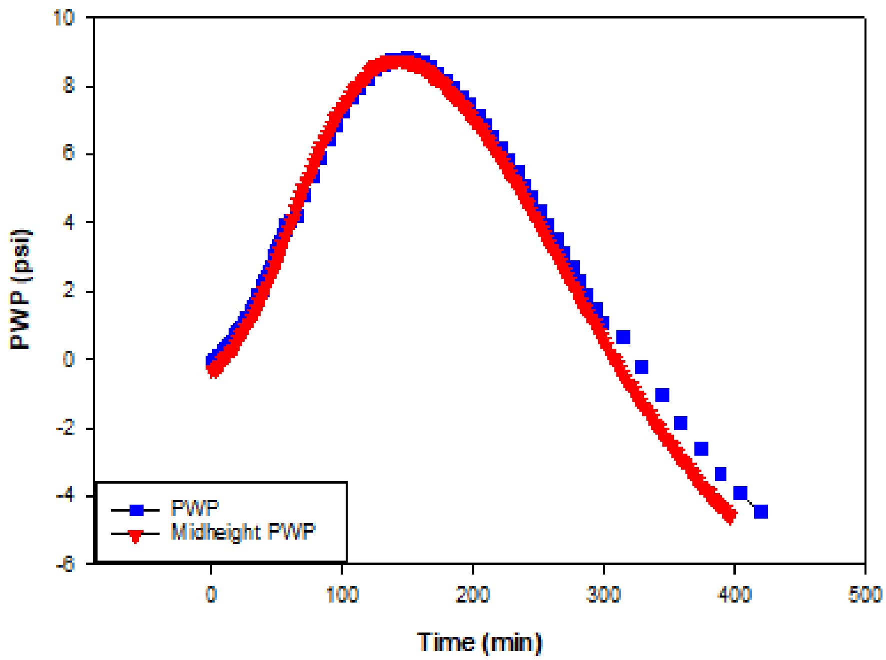



**Figure 42. Comparison between PWP (psi) at ends and mid-height vs time (min) for sample 76 with 0% plastic fines**

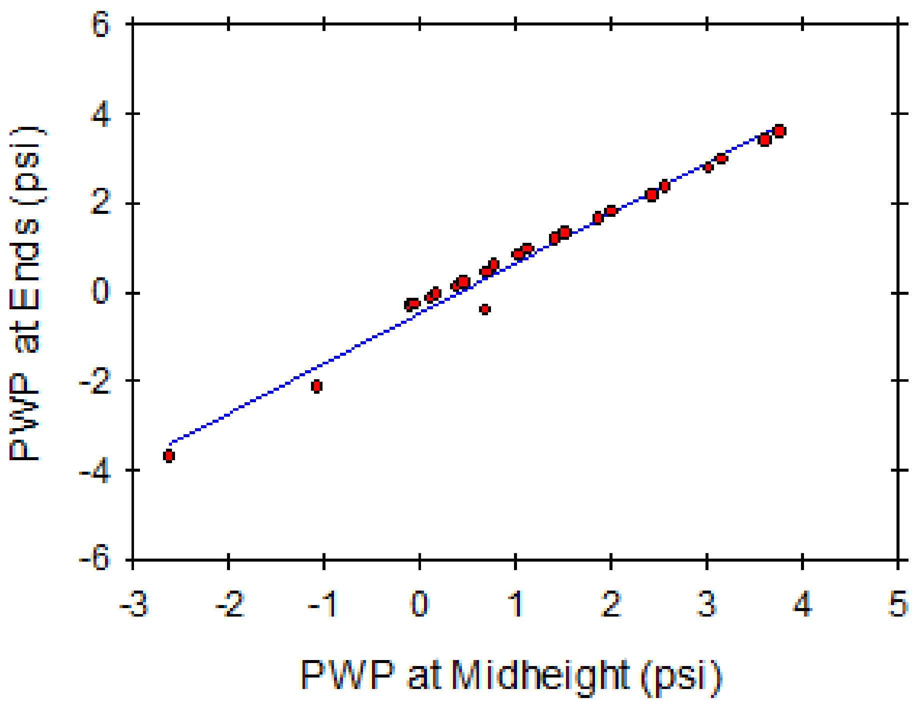



**Figure 43. Variation between PWP at ends and mid-height for sample 76 with 0% plastic fines**

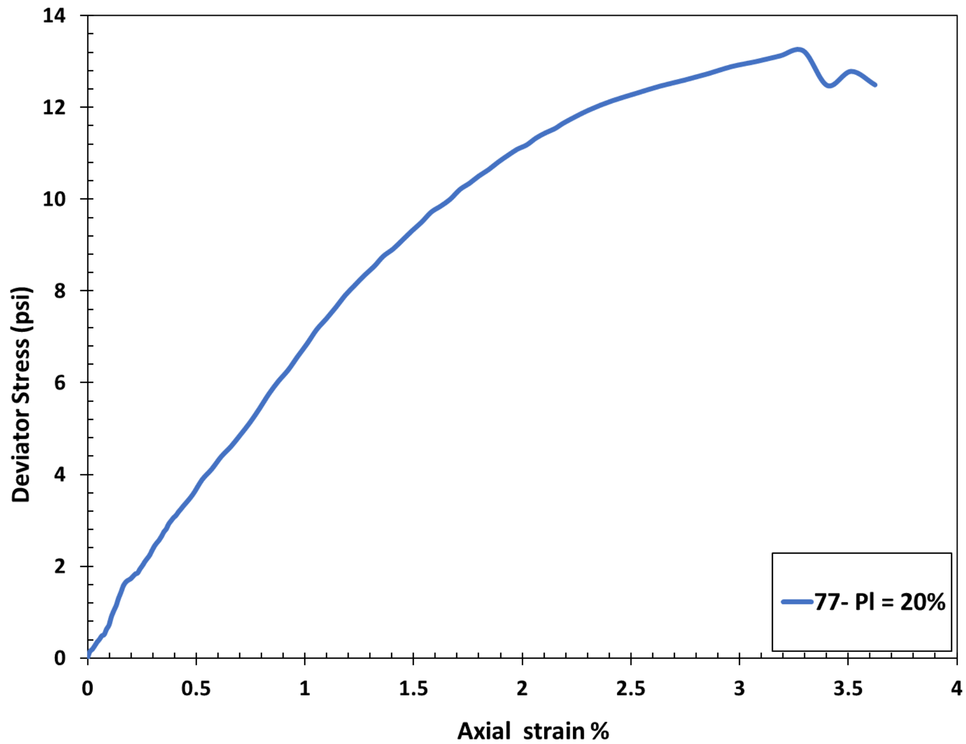



**Figure 44.  Deviator Stress (psi) vs Axial Stress % for sample 77 with 20% plastic fines**

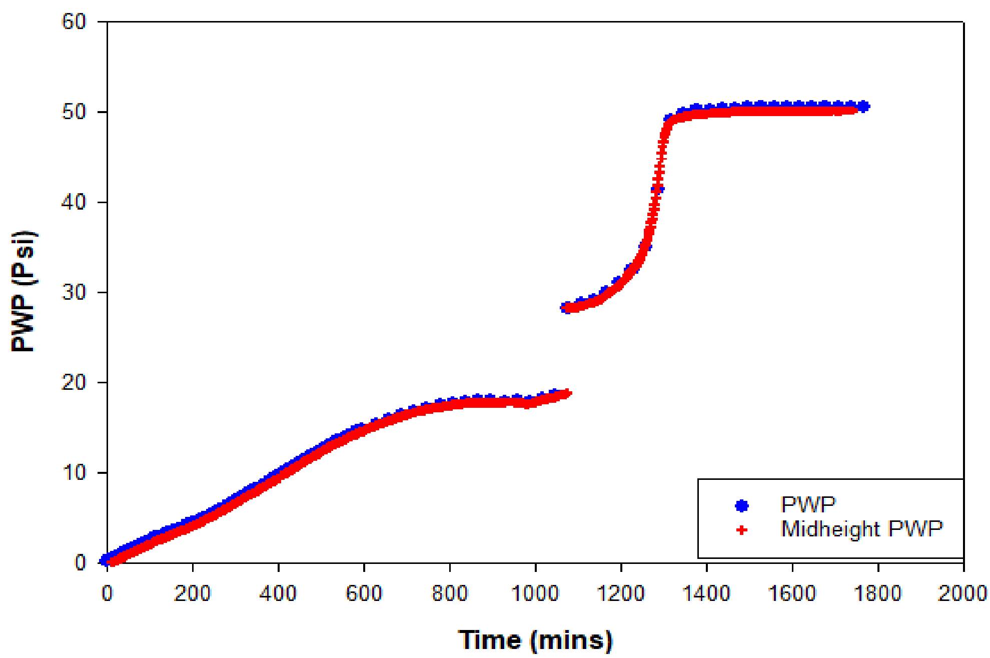



**Figure 45.  Comparison between PWP (psi) at ends and mid-height vs time (min) for sample 77 with 20% plastic fines**

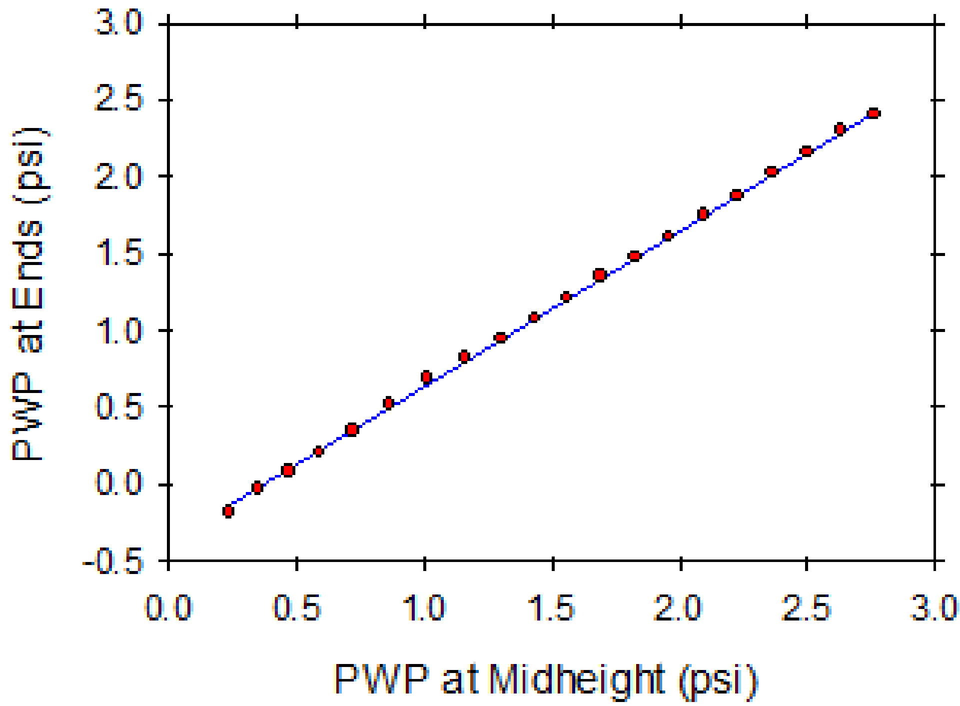



**Figure 50. Variation between PWP at ends and mid-height for sample 77 with 20% plastic fines**

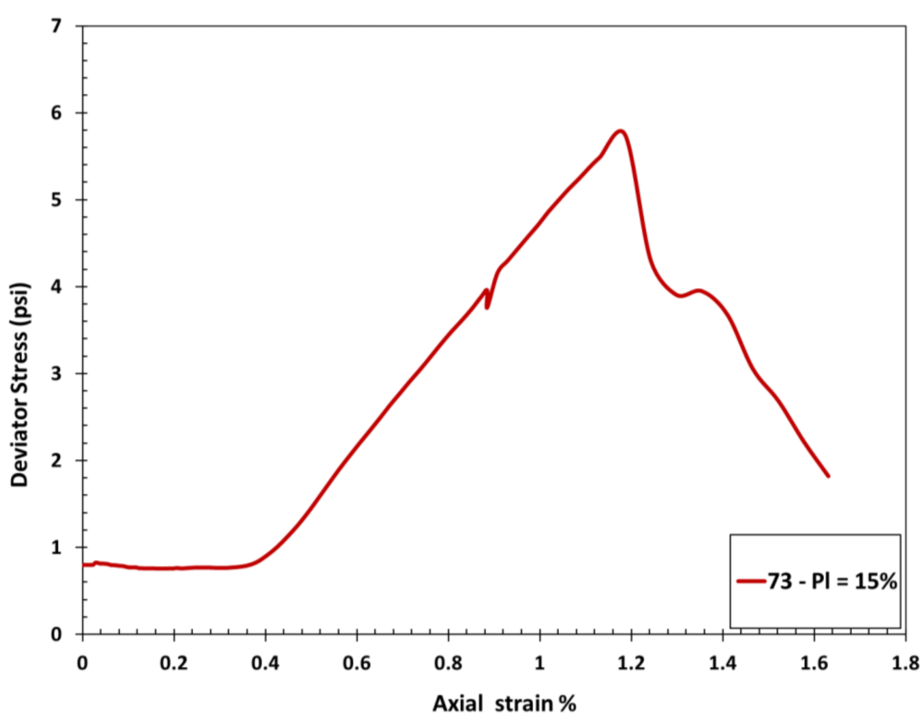



**Figure 51. Deviator Stress (psi) vs Axial Stress % for sample 73 with 15% plastic fines**

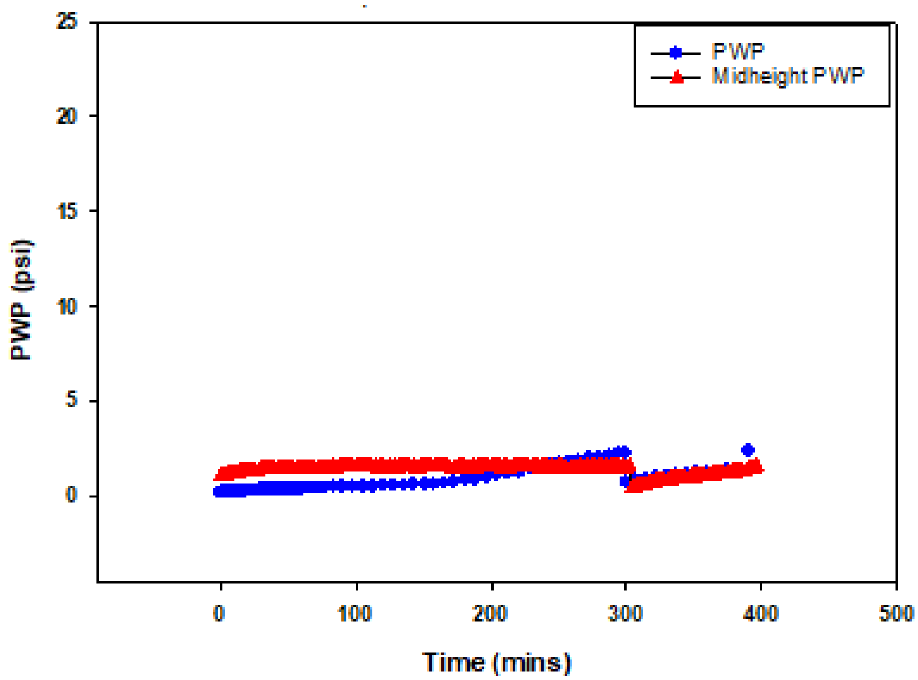



**Figure 46. Comparison between PWP (psi) at ends and mid-height vs time (min) for sample 73 with 15% plastic fines**

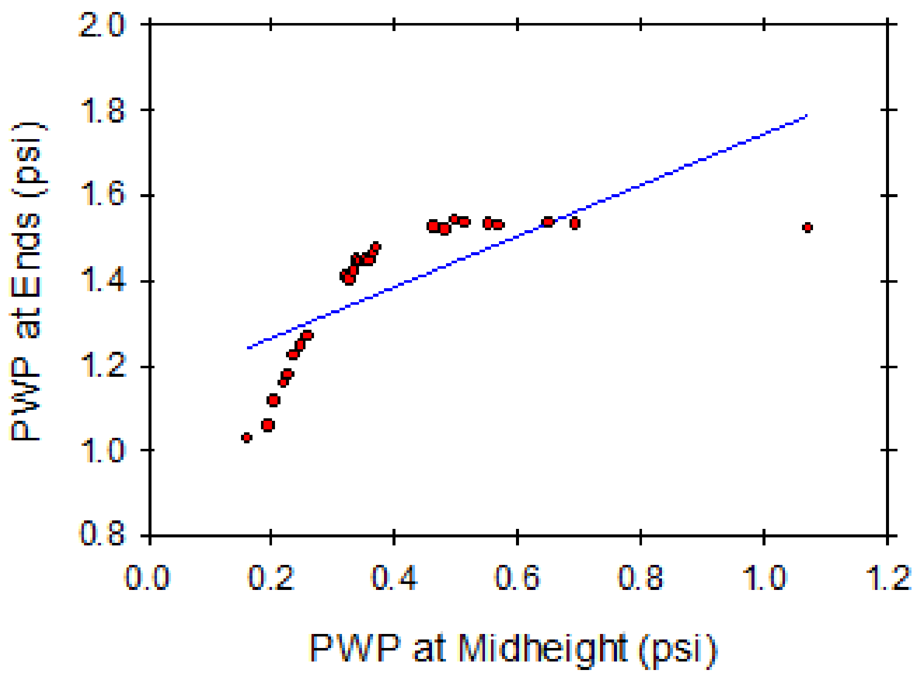



**Figure 47. Variation between PWP at ends and mid-height for sample 73 with 15% plastic fines**

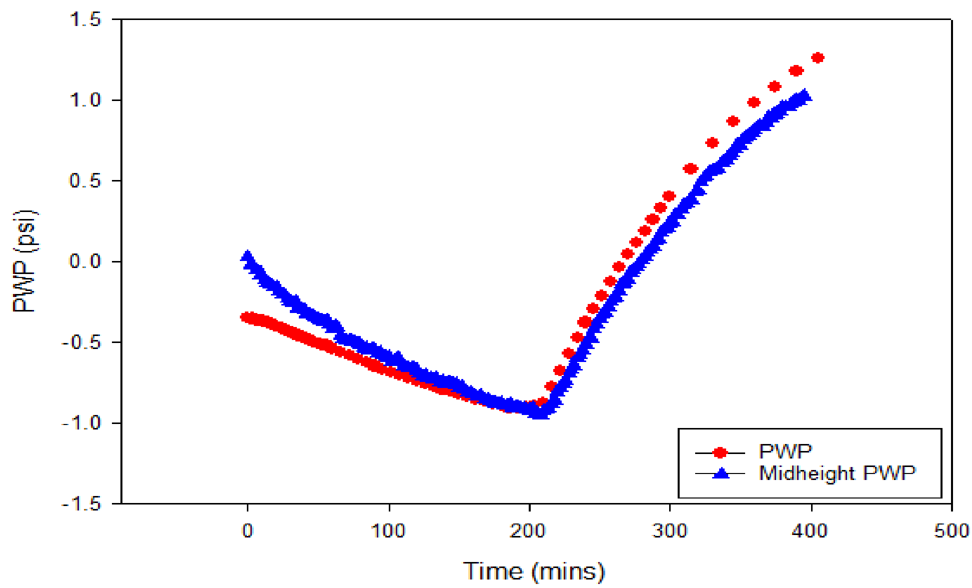



**Figure 54. Comparison between PWP (psi) at ends and mid-height vs time (min) for sample 73 with 15% plastic fines**

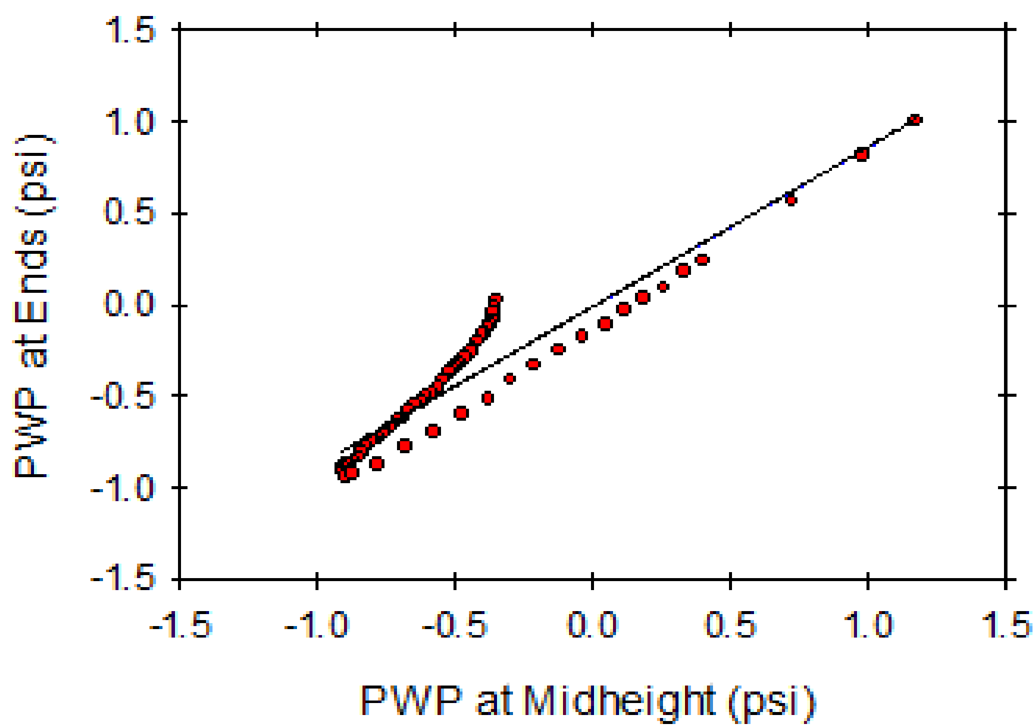



**Figure 48. Variation between PWP at ends and mid-height**

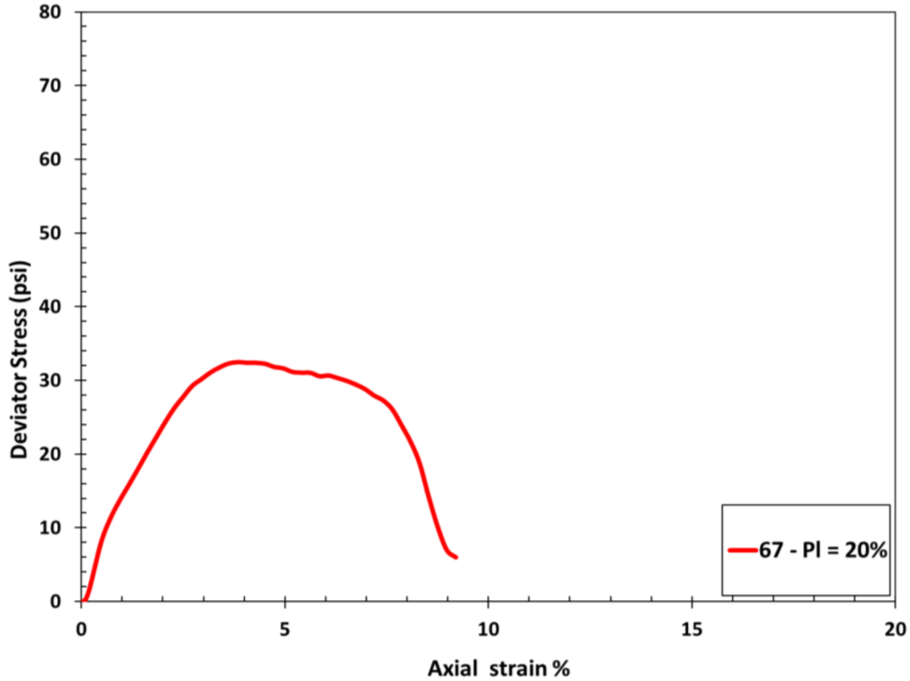



**Figure 49. Deviator Stress (psi) vs Axial Stress (%) for sample 67 with 20% plastic fines**

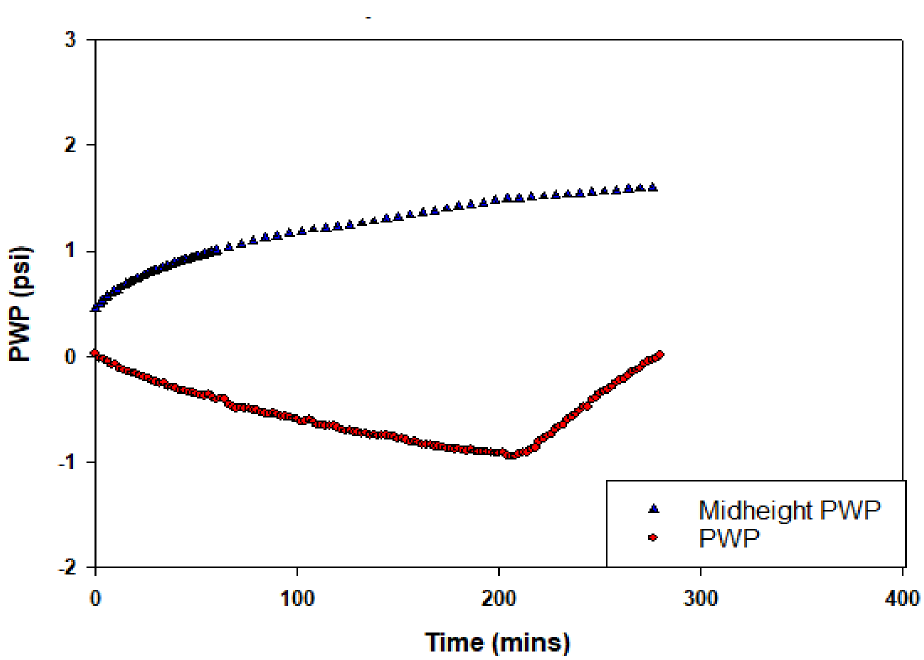



**Figure 50. Comparison between PWP (psi) at ends and mid-height vs time (min) for sample 67 with 20% plastic fines**

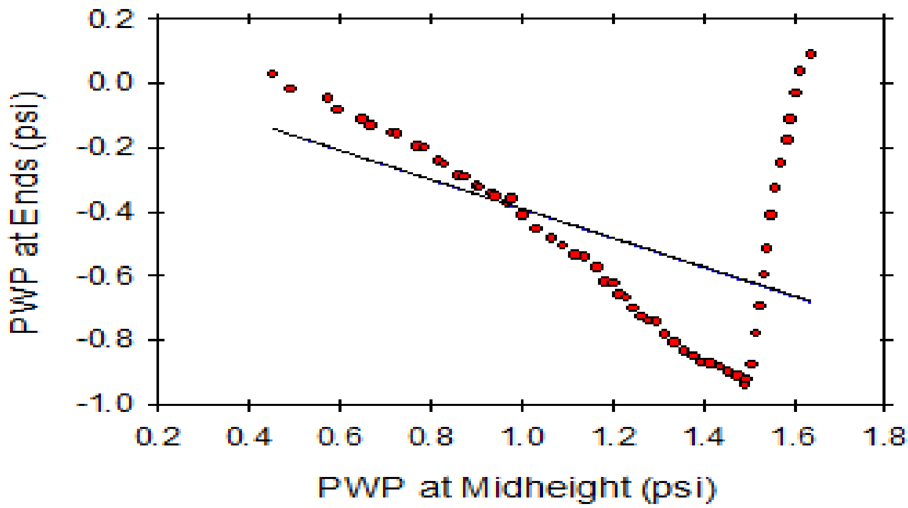



**Figure 51. Variation between PWP at ends and mid-height for sample 67 with 20% plastic fines**

# CHAPTER 7. CONCLUSION

In this study, the effects of non-plastic and plastic fines were observed on the sands of different relative densities. Thevanagayam et al. (2002) proposed three (3) different possible arrangements of fines within the sand matrix. As the amount of fines in the sand increases the limiting threshold fines content is reached where the fines no longer occupy the voids but participate in the loading during shear. The behavior of the sand with fines mixture changes upon reaching the threshold fines content was observed in the sand with silt mixture. It was also noted that this behavior is dependent on the initial relative density of the host sand and the threshold frequency is different for loose, medium dense and dense sands. The effect the fines have on the soil does not merely depend on the amount and type of fines but also on the initial relative density the sand is compacted within. Sands show a tendency of contraction till the limiting threshold fines content is reached and begin dilating as silts content increases further.

The use of skeletal void ratio was used to represent the fabric of the sand and fines mixture as opposed to relative density or traditional void ratio. Depending on the relative density during compaction, the sand undergoes five behaviors raging from liquefaction for very loose sands to stability for medium dense sands and dense sands. This behavior does not only depend upon the initial relative density but also on the confining stress during undrained shearing. All the tests in this study were carried out at a constant confining stress of 50 psi.

## 7.1. Discussion

Loose sands undergo dilative behavior during shear and showed a stress hardening response with an increase of silt content. These results were confirmed by previous research conducted by Pitman, Robertson, and Sego (1994a). As the silt content in the sand increased to 10%, as observed in Figure 24, the loose sand showed a contractive behavior. This indicates the fines that are present in the voids participated in the load distribution within the sands. With a further increase in the silt content, the sand showed a more rapid contraction and resistance towards liquefaction behavior. However, as the sands reached the threshold frequency at 30% in case of loose sands, the soil started to undergo dilation due to the fines now not just participating but filling the voids and becoming a part of the stress distribution in the grain to grain interaction of the sand particles.

This result was prominently observed in sands with a higher density such as the medium to dense sands in Figure 29. For medium to dense sands, the contractive behavior as observed for loose sands was also observed at lower fines content up to 12 and 18%, respectively. However, with an increase of silt content beyond this point, the sand and silt mixture showed dilation behavior. It was highly prominent for dense sands at 35% silt and medium dense sand at 40% silt content where the soils are extremely dilatant due to silt participation in the shearing stage. These soils also exhibited reversal behavior with respect to stability upon reaching the threshold content.

In the case of plastic fines, however, with the increase of fines resistance was observed toward liquefaction. The sands showed a steady decrease in strength but an increase in stability. This observation has been consistent for both loose as well as medium dense and dense sands. In the case of dense sands, the sand with fines mixture was stable at lower ends of the sand with kaoline mixture.

During the triaxial tests, the pore water pressure measurement was carried out in the middle of the specimen as compared to the traditional measurement. These tests were carried out on pure sand and sand and plastic fines mixture at various fines content and also on different relative densities. It was observed that as the fines content increases, the distribution of the pore water pressure at 2 different points measured at the same time reduces and it no longer is the same. This means the pore water pressure diffusion takes a longer time as compared to the just sands. It also changes on the increase of relative density of the sands at a constant plastic fines content. The diffusion of pore water pressure within the soil sample was the highest in the case of dense sands with a higher high amount of plastic fines.

**7.2. Implications**

The study provides a better understanding of the behavior of sands with fines mixture and helps better understanding of the soil using steady monotonic loading. The understanding that the increase in fine content depending upon the relative density and type of sand can be used as an effective means to prevent liquefaction in liquefaction prone areas and areas consisting of loose sands. It can be developed as a soil mixing technique for ground improvement.

**7.3. Limitations**

This study determined that the behavior of the sand with fines mixture depends upon the distribution of fines, the arrangement of particles and the relative density of the mixture. Hence, the behavior of the mixture changes with the change in the intrinsic properties of the material. Ottawa sand is medium dense sand with rounded particles. The behavior of fine sands or very coarse sands might be different since the distribution of fines within them change along with the nature of the sand skeleton. It also changes with the size and nature of fines, their plasticity index, their behavior when in contact with water.

In the case of PWP measurement at two (2) ends, the greatest limitations are the time and complexity of making the sample and performing the test. Due to the puncture made by the needle in the membrane, the soil is very susceptible to leakage which can lead to failure. The presence of the needle also provides resistance to the shear in the soil and does not provide an accurate picture for stress distribution within the system. Another limitation is the blocking caused in the needle because of the flow of the fines due to pore water pressure which might lead to incorrect measurements and require regular maintenance.

**7.4. Future Research**

The following recommendations are suggested for further study:

- Study the behavior of sand with fines mixture in static loading to gain a clearer understanding of the distribution of fines within the sand skeleton. Further research should be conducted to investigate different types of sand with finer or coarser grains to analyze the behavior in greater detail.
- Examine the properties of the fines by changing its PI, sizes and shapes, e.g. flaky or rotund fines.
- Perform cyclic triaxial tests to understand the behavior of sands with fines during earthquakes to more fully understand the nature of liquefaction.
- Conduct triaxial tests with PWP measurement at multiple points to acquire a better understanding of the generation and dissipation of PWP within samples, especially clays where the PWP distribution is slower upon loading, and observe a clearer delineation of PWP distribution within the soil samples.

**REFERENCES**

Baldi, G., D. W. Hight, and Thomas, G. E. (1988). A reevaluation of conventional triaxial test methods. *Advanced Triaxial Testing of Soil and Rock, ASTM STP 977*, 219–63. https://doi.org/10.1016/j.infsof.2008.09.005

Bayat, M., Bayat, E., Aminpour,H., and Salarpour, A. (2014). Shear strength and pore-water pressure characteristics of sandy soil mixed with plastic fine. *Arabian Journal of Geosciences, 7(*3):1,049–1,057. https://doi.org/10.1007/s12517-012-0753-9

Carraro, J., Antonio, H., Prezzi, M, and Salgado, R. 2009. Shear strength and stiffness of sands containing plastic or nonplastic fines. *Journal of Geotechnical and Geoenvironmental Engineering* 135 (9):1167–78. https://doi.org/10.1061/(ASCE)1,090–0241(2009)135:9(1,167).

Cherrill, H. E. (1990). The influence of loading rate on excess pore pressures in triaxial tests. (Doctoral dissertation), London, U.K.: City University.

Chu, J. Y., and Leong W. K. (2002). Effect of fines on instability behaviour of loose sand. *Géotechnique*, *52*(10):751–755.

Fourie, A. B., and Xiaobi, D. 1991. Advantages of midheight pore pressure measurements in undrained triaxial testing. *Geotechnical Testing Journal*, *14*(2):138–145. https://doi.org/10.1520/GTJ10555J

Georgiannou, V. N. (1988, November). *The behaviour of Clayey Sands under monotonic and cyclic loading.* (Doctoral dissertation), U.K.: University of London.

Georgiannou, V. N. (2006a). The undrained response of sands with additions of particles of various shapes and sizes. *Géotechnique, 56*(9):639–649. https://doi.org/10.1680/geot.2006.56.9.639

Georgiannou, V. N. (2006b). The undrained response of sands with additions of particles of various shapes and sizes. *Géotechnique, 56*(9):639–649. https://doi.org/10.1680/geot.2006.56.9.639

Georgiannou, V. N, Burland, J. B., and Hight, D. W. (1990). The undrained behaviour of clayey sands in triaxial compression and extension. *Géotechnique, 40*(3), 431-449.

Hight, D. W. (1982). A simple piezometer probe for the routine measurement of pore pressure in triaxial tests on saturated soils. *Géotechnique, 32(*4):396–401. https://doi.org/10.1680/geot.1982.32.4.396

Holtz, R. D., and Kovacs, W. D. 1981. *An introduction to geotechnical engineering*. Englewood Cliffs, NJ: Prentice-Hall.

Ishihara, K. (1993). Liquefaction and flow failure during earthquakes. *Géotechnique, 43*(3):351–451. https://doi.org/10.1680/geot.1993.43.3.351

Karim, M. E., and Jahangir Alam, M. (2017). Effect of nonplastic silt content on undrained shear strength of sand-silt mxtures. *International Journal of Geo-Engineering, 8*(1). Singapore: Springer, 1–26. https://doi.org/10.1186/s40703-017-0051-1

Kenny, T. C. (1976). Residual strength of mineral mixtures. In *9^< Th> International Conference on Soil Mechanics*, 155–60.

Lade, P. V., and Yamamuro, J. A. (1997). Effects of nonplastic fines on static lLiquefaction of sands. *Canadian Geotechnical Journal, 34*(6): 918–928. https://doi.org/10.1139/t97-052

Mahedi, M., Bora, C., and Dayioglu, A. (2019). Leaching behavior of aluminum, copper, iron and zinc from fly ash and slag stabilized soil. *Waste Management*, *95*, 334-355.

Maher, A., Rutherford, C., and Schaefer, V. (2018). Development of a split mold for internal consolidated undrained isotropic compression (CUIC) testing of low effective-stress soils from slurry by. (Master of Science thesis), Iowa City, University of Iowa.

Maleki, M., Ezzatkhah, A., Bayat, M., and Mousivand, M. (2011). Effect of physical parameters on static undrained resistance of sandy soil with low silt content. *Soil Dynamics and Earthquake Engineering*, *31*(10). Elsevier: 1,324–1,331. https://doi.org/10.1016/j.soildyn.2011.05.003

Mitchell, J. K., and Soga, K. (1976). *Fundamentals of soil behavior*. New York, NY: John Wiley & Sons.

Mohamad, Ramli, B., and Dobry, R. (1986). Undrained monotonic and cyclic triaxial strength of sand. *Journal of Geotechnical Engineering, 112*(10):941–958. https://doi.org/10.1061/(ASCE)0733-9410(1986)112:10(941)

Murthy, T. G., Loukidis, D., Carraro, J. A. H., Prezzi, M., and Salgado, R. (2007). Undrained monotonic response of clean and silty sands. *Géotechnique, 57*(3):273–288. https://doi.org/10.1680/geot.2007.57.3.273

Ni, Q., Tan, T. S., Dasari, G. R., and Hight, D. W. (2004). Contribution of fines to the compressive strength of mixed soils. *Géotechnique, 54*(9):561–569. https://doi.org/10.1680/geot.2004.54.9.561.

Ölmez, M. S. (2008). Shear strength behaviour of sand-clay mixtures. (Master of Science thesis). http://etd.lib.metu.edu.tr/upload/12609558/index.pdf

Ovando-Shelley, E., and Pérez, B. E. (2009). Undrained behaviour of Clayey Sands in load controlled triaxial tests. *Géotechnique, 47*(1):97–111. https://doi.org/10.1680/geot.1997.47.1.97

Pitman, T. D., Robertson, P. K., and Sego, D. C. (1994a). Influence of fines on the Collapse of loose sands. *Canadian Geotechnical Journal, 31*(5): 728–739. https://doi.org/10.1139/t94-084

Pitman, T. D. (1994b). Influence of fines on the collapse of loose sands. *Canadian Geotechnical Journal*. https://doi.org/10.1139/t94-084

Salgado, R., Bandini, P., and Karim, A. (2001, April). Shear strength and stiffness of silty sand *126*, 367–373.

Thevanayagam, S. (1998). Effect of fines and confining stress on undrained shear strength of silty sands. *Journal of Geotechnical and Geoenvironmental Engineering, 124*(6):479–491. https://doi.org/10.1061/(ASCE)1090-0241(1998)124:6(479)

Thevanayagam, S. (2007). Closure to “Effect of Fines and Confining Stress on Undrained Shear Strength of Silty Sands.” *Journal of Geotechnical and Geoenvironmental Engineering* 125(11):1,024–1,027. https://doi.org/10.1061/(asce)1090-0241(1999)125:11(1024.2)

Thevanayagam, S., Ravishankar, K, and Mohan, S. (1997). Effects of fines on monotonic undrained shear strength of sandy soils. *Geotechnical Testing Journal, 20*(4):394–406.

Thevanayagam, S., Shenthan, T., Mohan, S.,and Liang, J. (2002). Undrained fragility of clean sands, silty sands, and sandy silts. *Journal of Geotechnical and Geoenvironmental Engineering, 128*(10):849–859. https://doi.org/10.1061/(ASCE)1090-0241(2002)128:10(849)

Vaid, Y. P, and Negussey, D. 1988. Preparation of reconstituted sand specimens. In *Advanced triaxial testing of soil and rock*. https://doi.org/10.1520/STP29090S

Vaid, Y. P., Sivathayalan, S., and Stedman, D. (1999). Influence of specimen-reconstituting method on the undrained response of sand. *Geotechnical Testing Journal, 22*(3):187–195. https://doi.org/10.1520/GTJ11110J

Wood, F., Jerry, M., Yamamuro, A., and Lade, P. V. (2008). Effect of depositional method on the undrained response of silty sand. *Canadian Geotechnical Journal, 45*(11):1,525–1,537. https://doi.org/10.1139/t08-079

Yamamuro, J. A., and Fletcher M. Wood. (2004). Effect of depositional method on the undrained behavior and microstructure of sand with silt. *Soil Dynamics and Earthquake Engineering, 24*(9–10):751–760. https://doi.org/10.1016/j.soildyn.2004.06.004

Yamamuro, J. A, and Lade, P. V. (1998). Steady-state concepts and static liquefaction of silty sands. *Journal of Geotechnical and Geoenvironmental Engineering, 124*(9):868–877. https://doi.org/10.1061/(ASCE)1090-0241(1998)124:9(868)

Zlatovic, S., and Ishihara, K. (2013). Normalized behavior of very loose non plastic soils: Effects of fabric. *Journal of Geotechnical and Geoenvironmental Engineering, 1*(3):1–11. https://doi.org/10.1061/ASCE1090-02412003129:112

# APPENDIX. ADDITIONAL DATA

| Sample | Relative density % | Type of fine | % of Fines (fc) | $e_{sk}$ | Initial Confining Pressure |
|---|---|---|---|---|---|
| 20 | 90 | Plastic | 15 | 0.7 | 50 |
| 25 | 45 | Plastic | 8 | 0.82 | 50 |
| 30 | 90 | Plastic | 10 | 1.02 | 50 |
| 33 | 75 | None | 0 | 0.56 | 50 |
| 36 | 90 | Plastic | 10 | 0.68 | 50 |
| 39 | 50 | None | 0 | 0.64 | 50 |
| 42 | 25 | Plastic | 5 | 0.86 | 50 |
| 44 | 45 | Plastic | 17 | 0.7 | 50 |
| 45 | 5 | Plastic | 35 | 1.76 | 50 |
| 47 | 5 | Plastic | 35 | 1.76 | 90 |
| 48 | 65 | None | 0 | 0.74 | 50 |
| 49 | 3 | Plastic | 40 | 2.03 | 103 |
| 50 | 50 | Plastic | 25 | 1.22 | 50 |
| 51 | 4 | Plastic | 20 | 0.88 | 50 |
| 52 | 35 | Loess | 15 | 1.07 | 50 |
| 53 | 14 | Loess | 30 | 1.21 | 50 |
| 54 | 25 | Loess | 20 | 1.31 | 50 |
| 57 | 15 | Loess | 10 | 0.94 | 50 |
| 58 | 18 | Loess | 23 | 1.5 | 50 |
| 59 | 40 | Plastic | 30 | 1.25 | 50 |
| 61 | 5 | Loess | 32 | 1.65 | 50 |
| 63 | 50 | Plastic | 0 | 0.64 | 50 |
| 65 | 15 | None | 0 | 0.78 | 50 |
| 66 | 12 | Plastic | 13 | 1.09 | 50 |
| 67 | 75 | Plastic | 20 | 0.56 | 50 |
| 68 | 40 | Plastic | 20 | 1.1 | 50 |
| 69 | 10 | Plastic | 20 | 1.04 | 50 |
| 74 | 12 | Plastic | 24 | 1.3 | 50 |
| 78 | 12 | Plastic | 31 | 1.58 | 50 |
| 81 | 12 | Plastic | 10 | 0.99 | 50 |
| 80 | 12 | Loess | 0 | 0.78 | 30 |
| 84 | 30 | Loess | 16 | 1.07 | 50 |
| 86 | 30 | Loess | 25 | 1.3 | 50 |
| 88 | 30 | Loess | 30 | 1.48 | 50 |
| 90 | 30 | Loess | 10 | 0.91 | 50 |

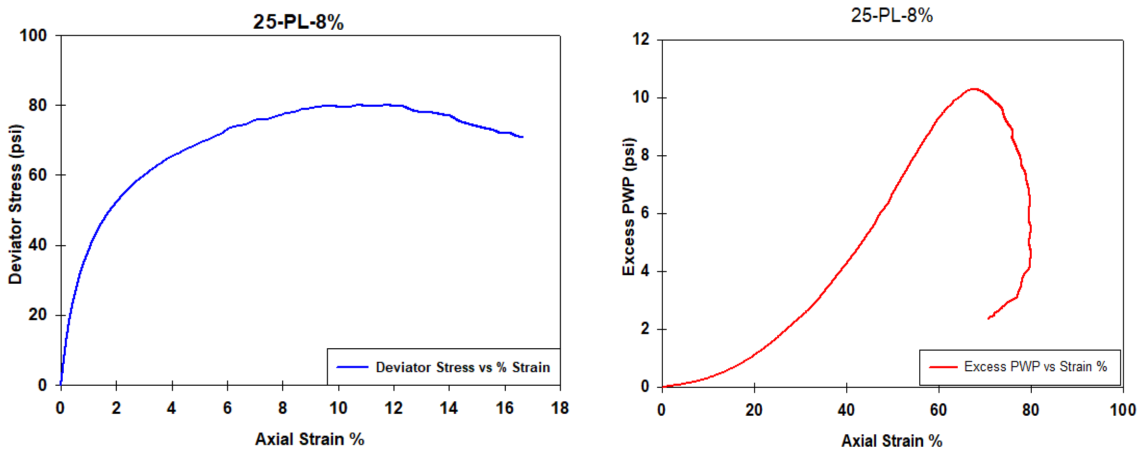



**Figure A-1. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 25**

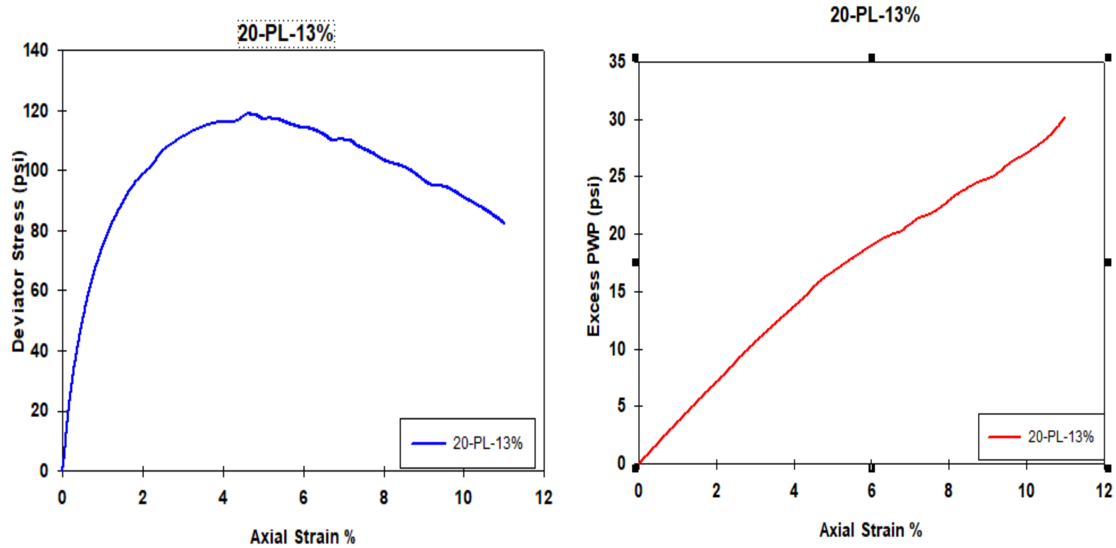



**Figure A-2. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 20**

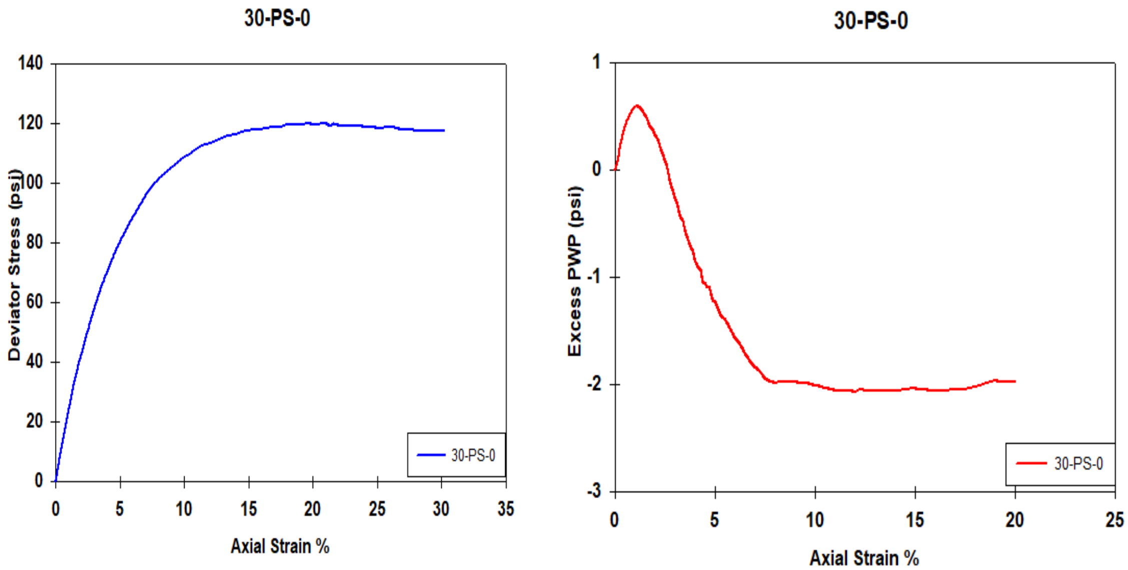



**Figure A-3. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 30**

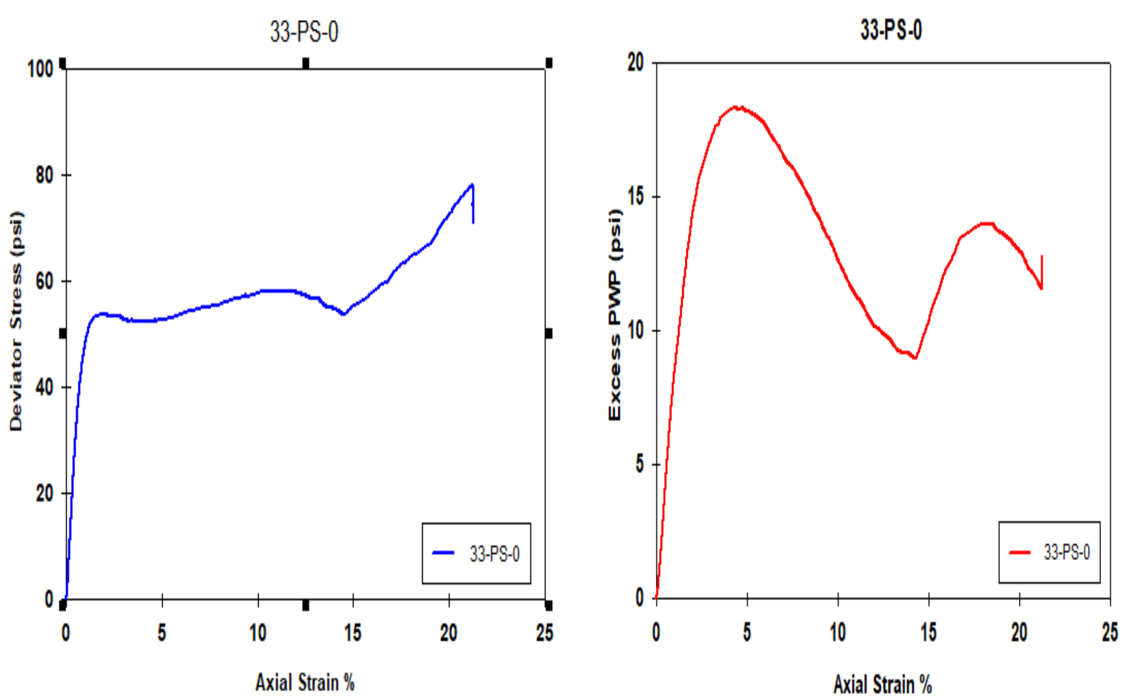



**Figure A-4. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 33**

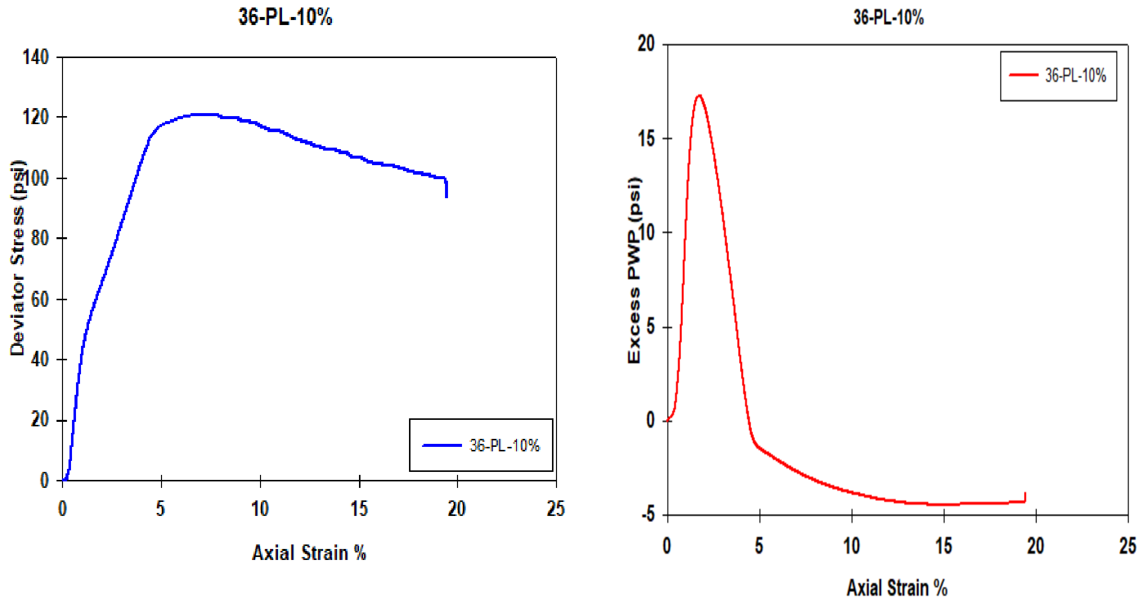



**Figure A-5. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 36**

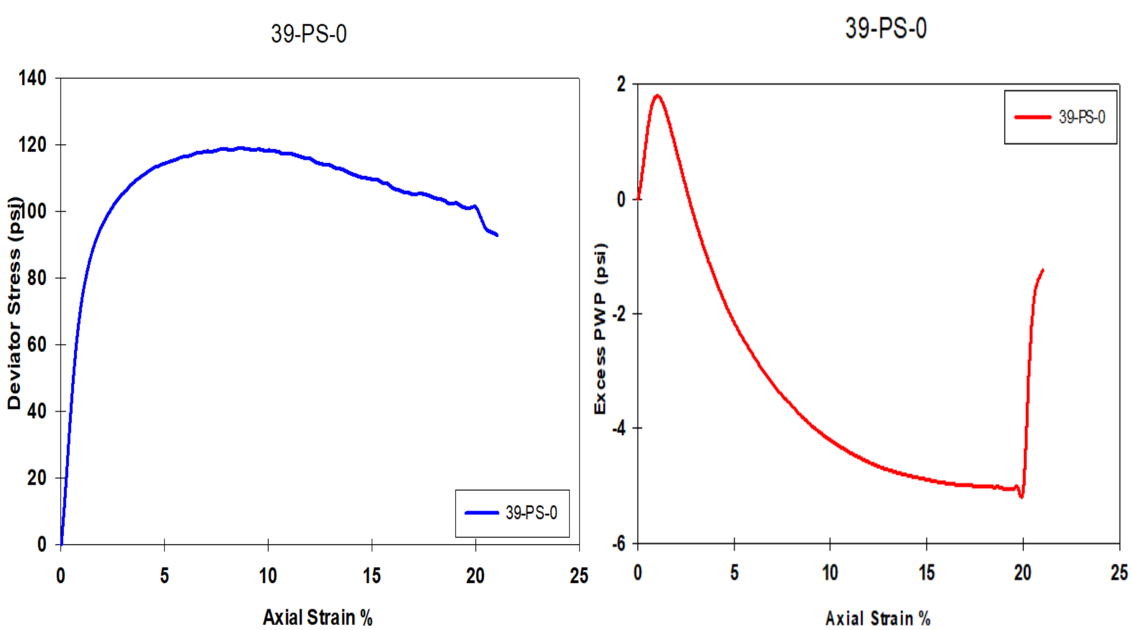



**Figure A-6. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 39**

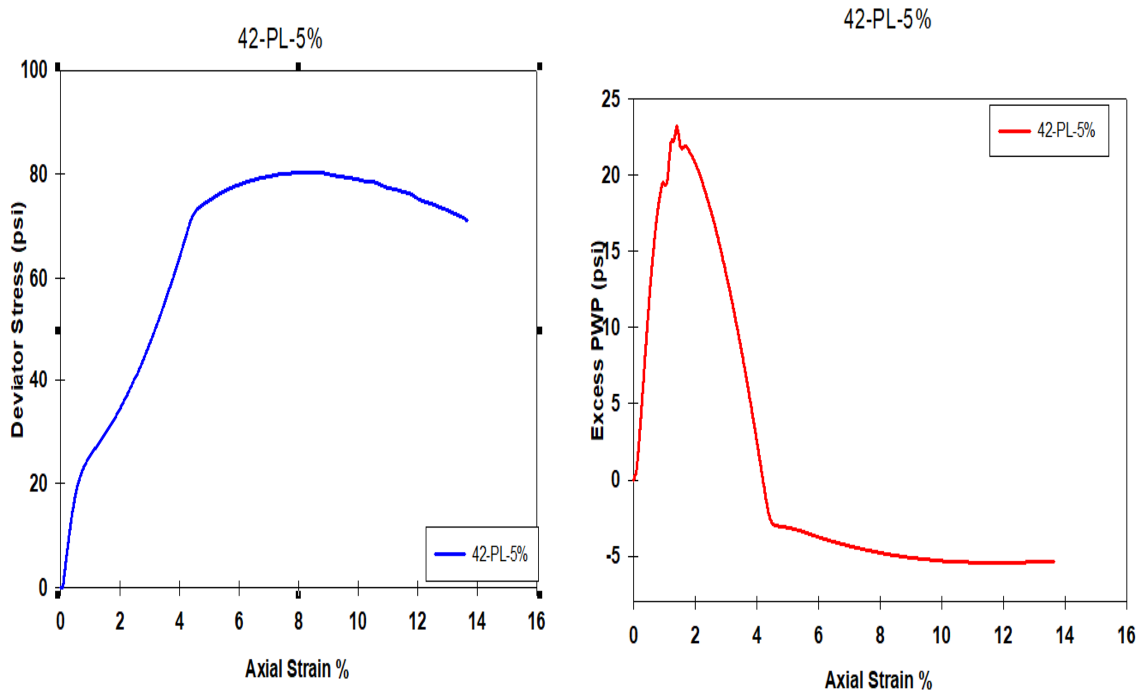



**Figure A-7. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 25**

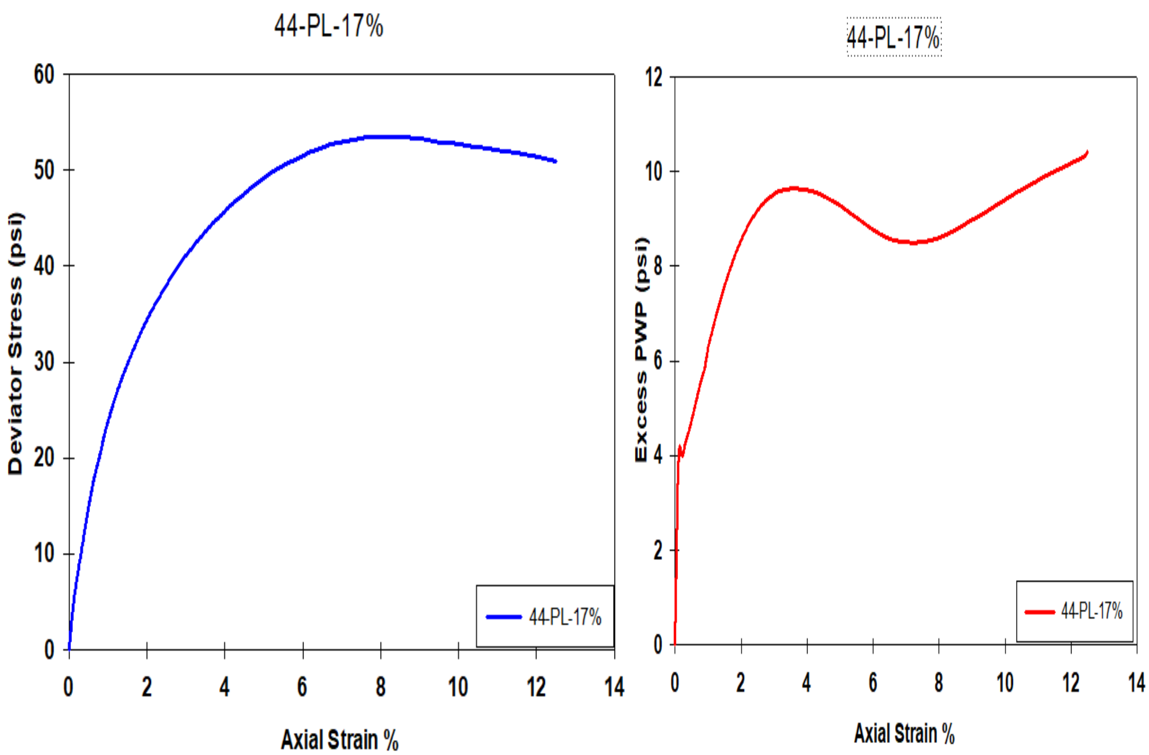



**Figure A-8. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 44**

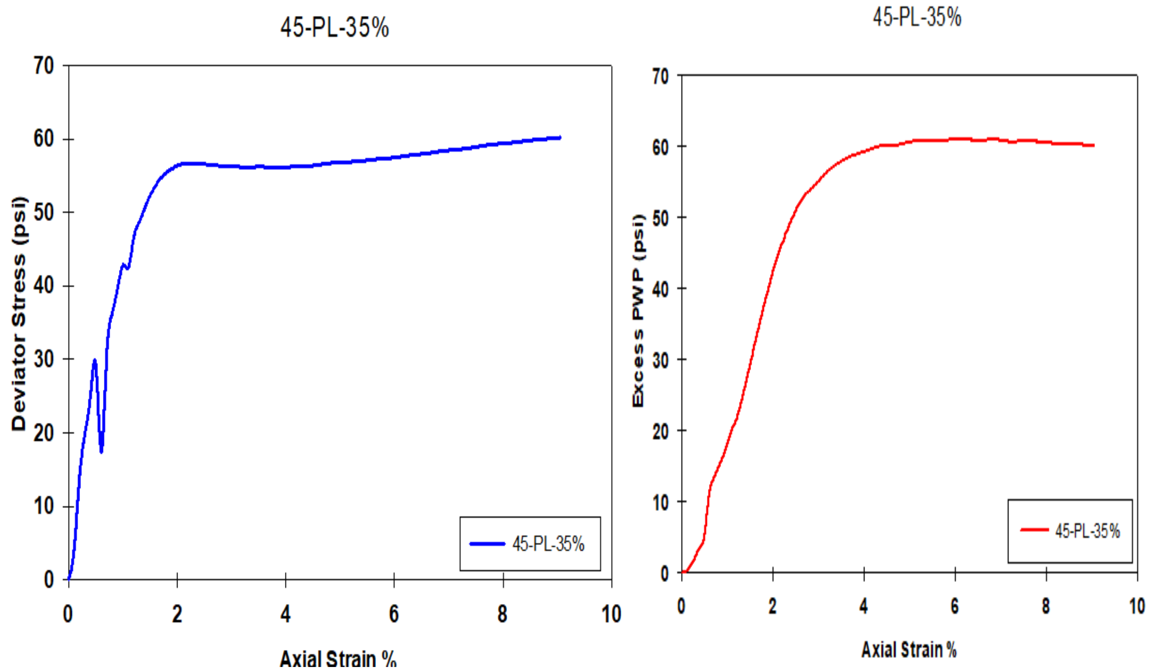



**Figure A-9. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 45**

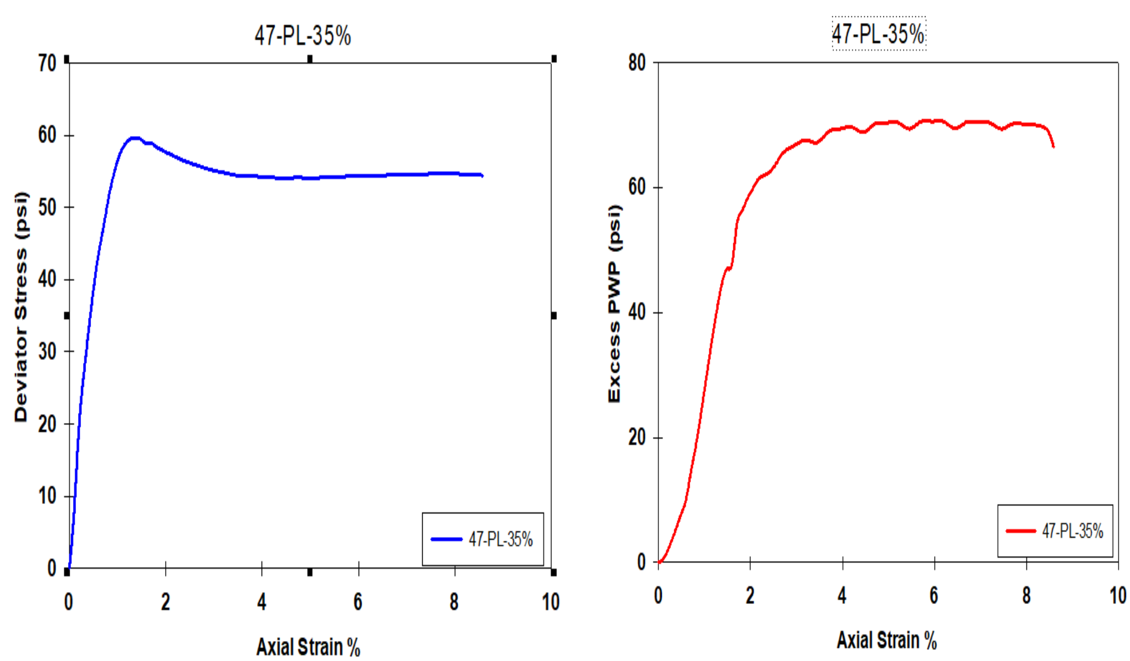



**Figure A-10. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 47**

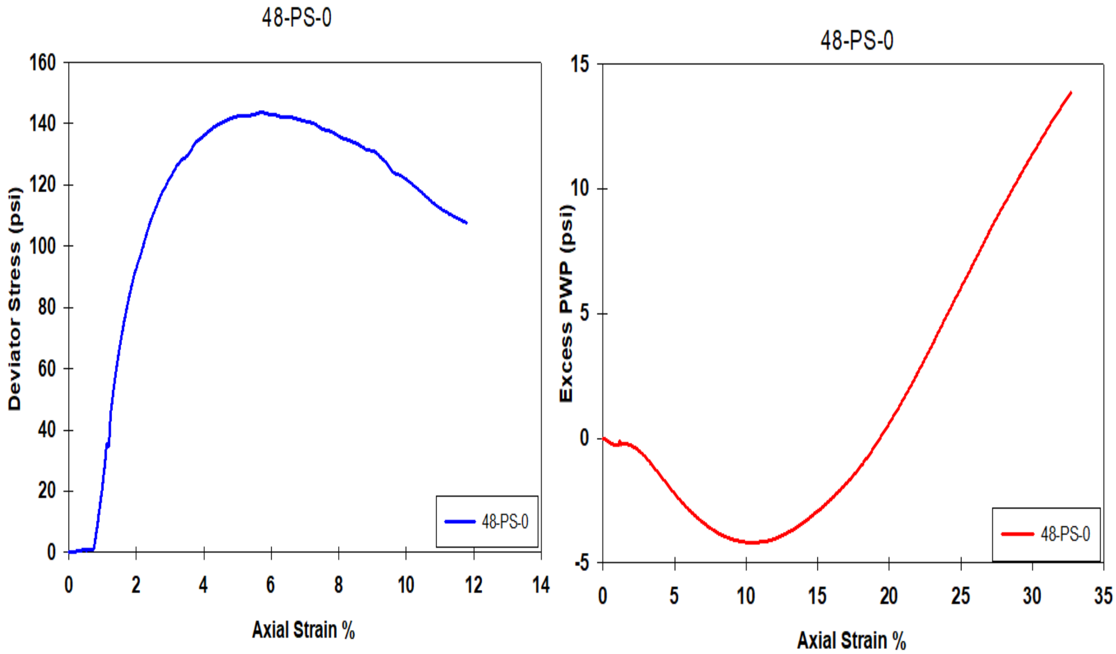



**Figure A-11. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 48**

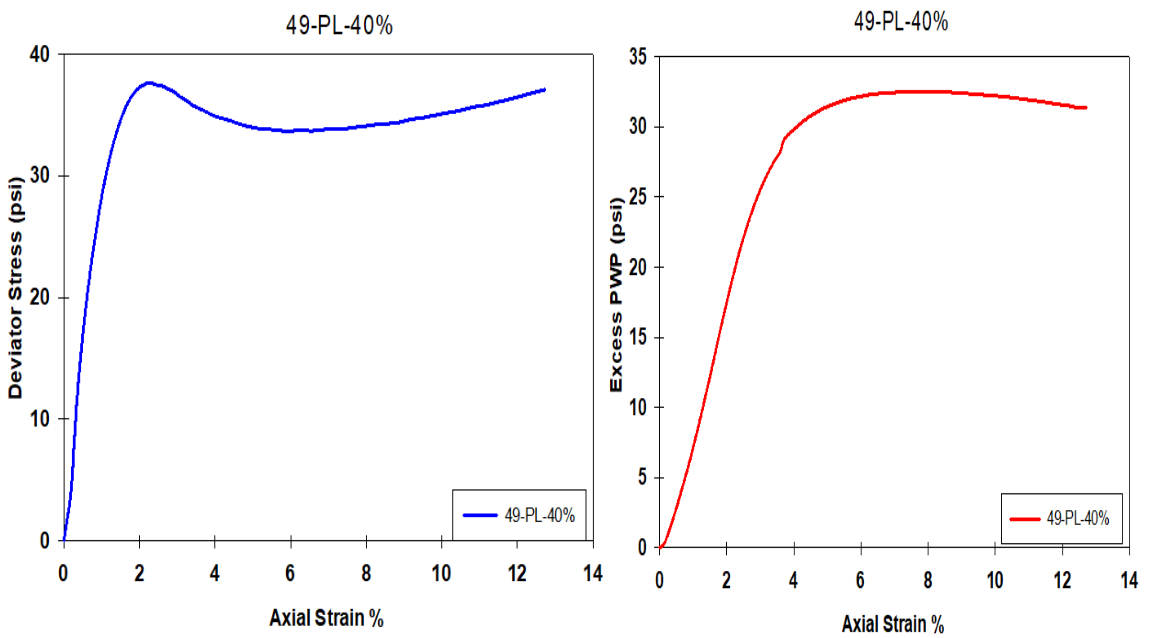



**Figure A-12. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 49**

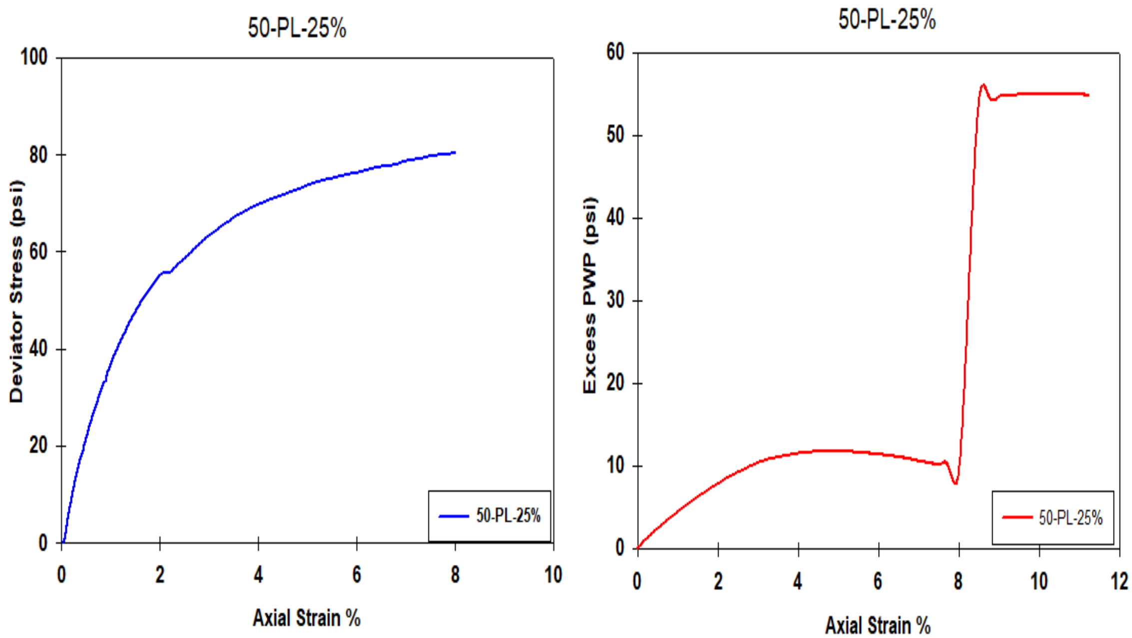



**Figure A-13. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 50**

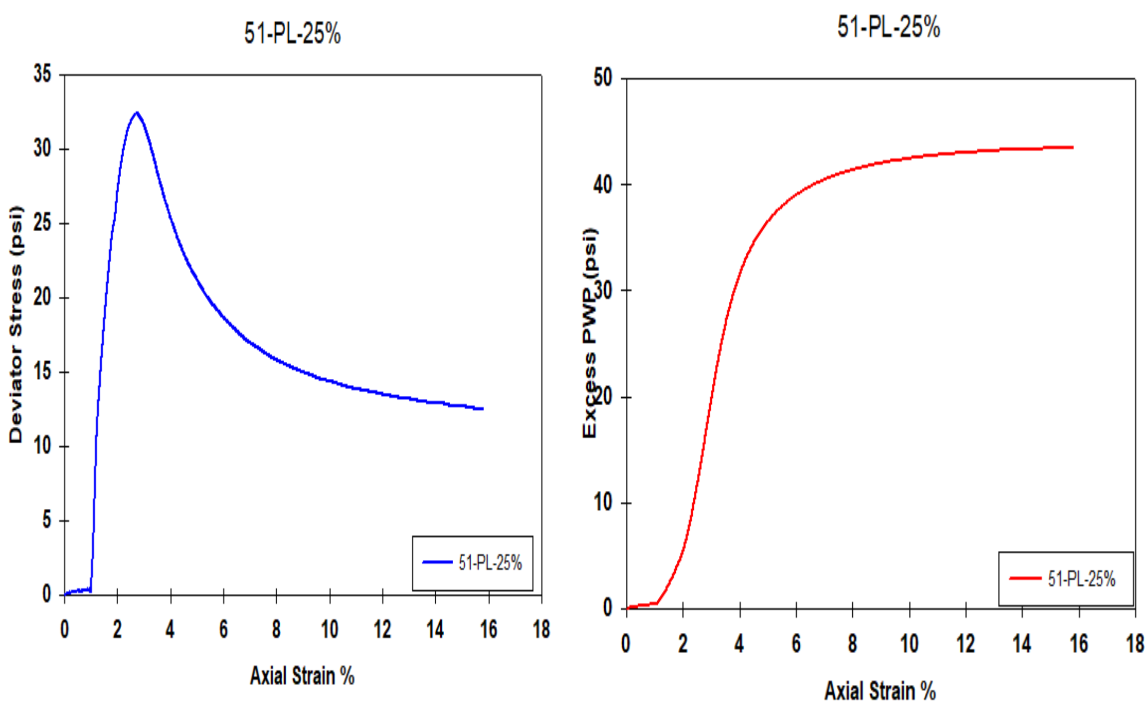



**Figure A-14. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 51**

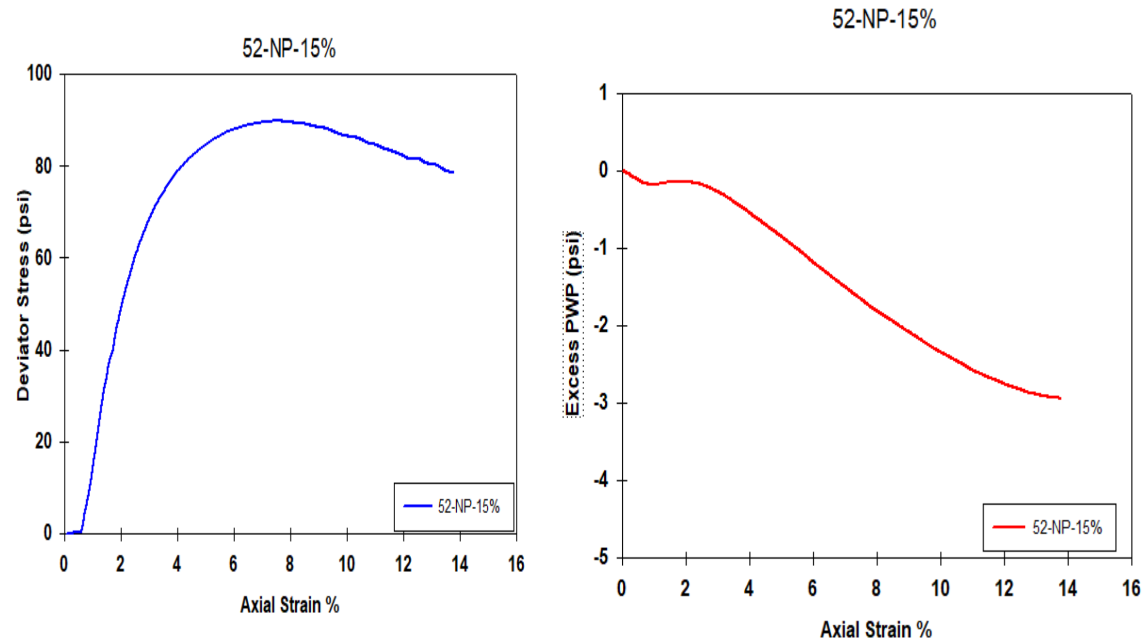



**Figure A-15. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 52**

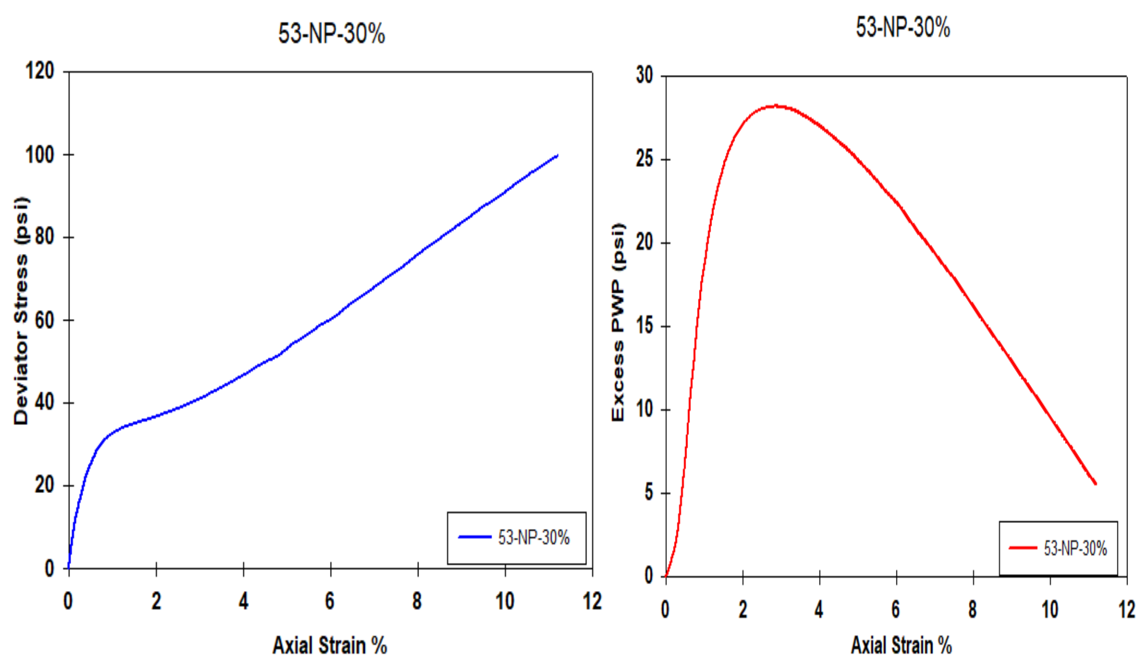



**Figure A-16. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 53**

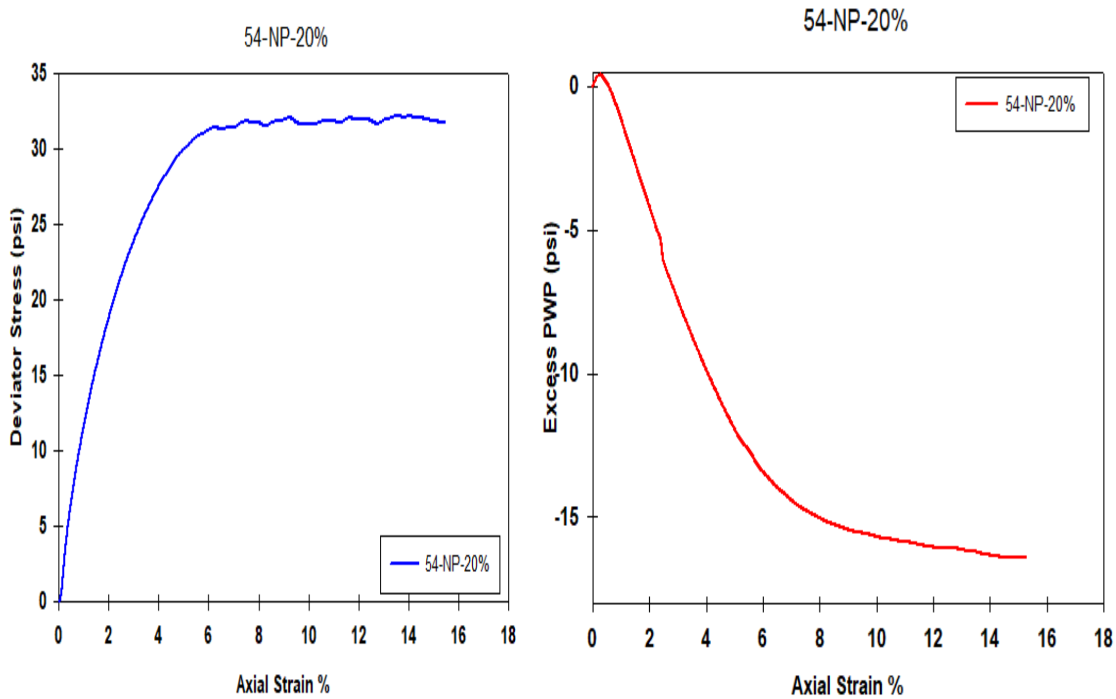



**Figure A-17. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 54**

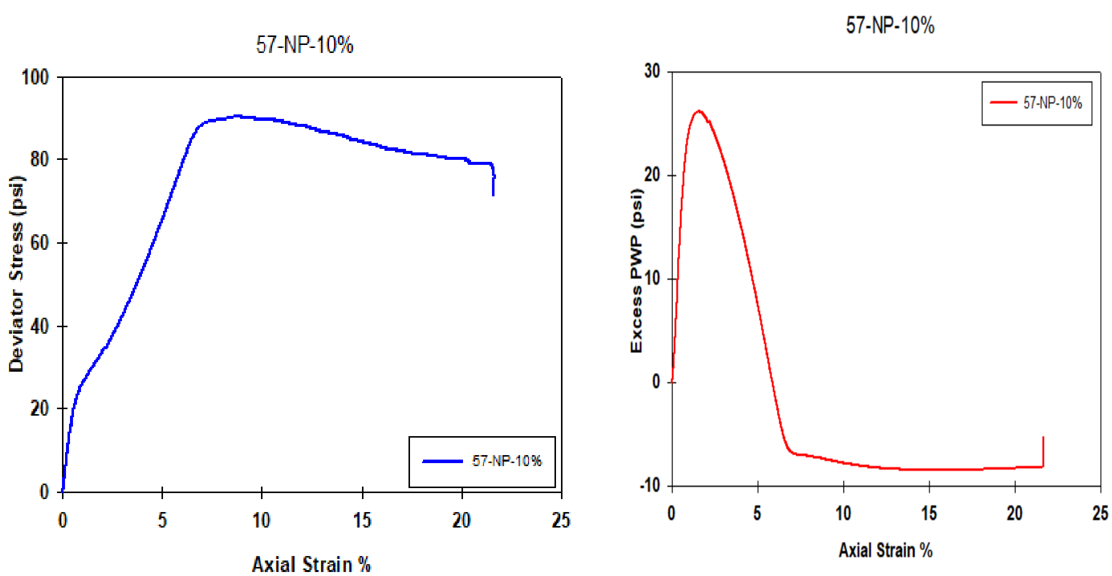



**Figure A-18. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 57**

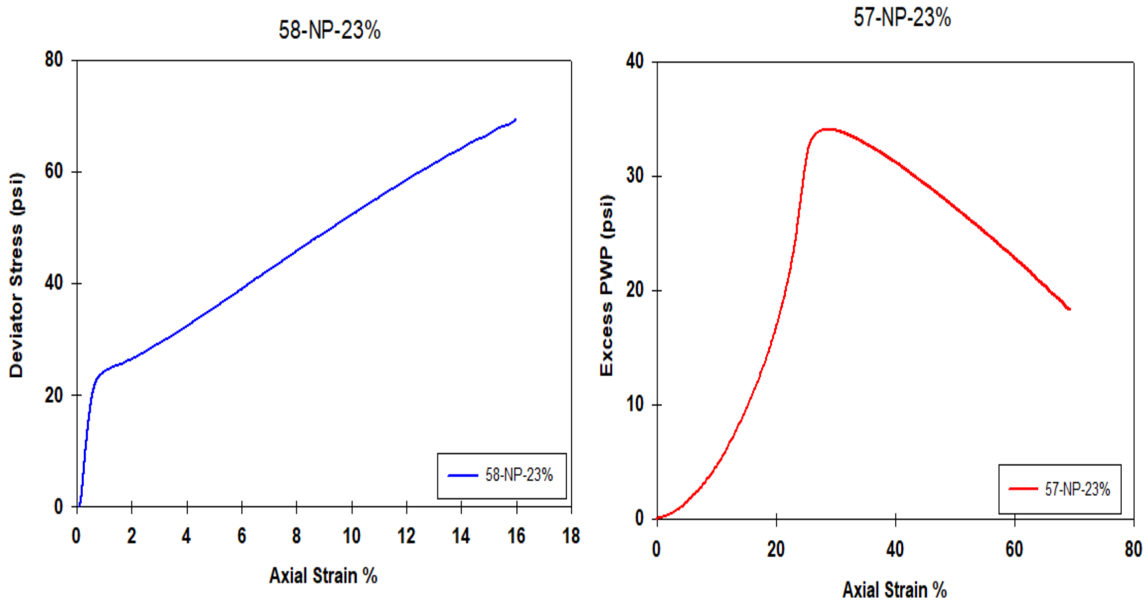



**Figure A-19. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 57**

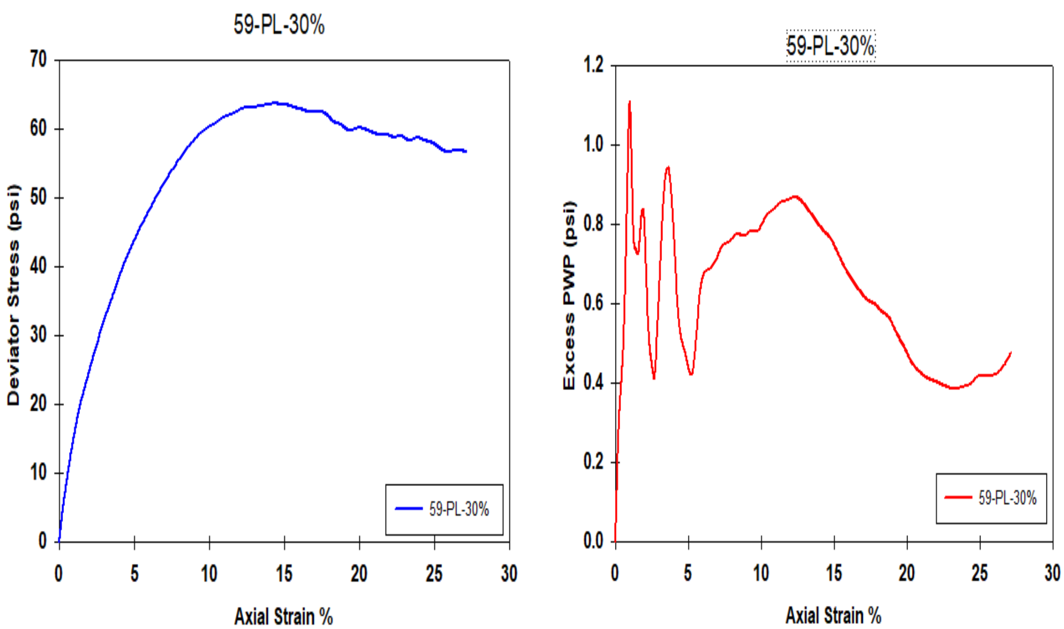



**Figure A-20. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 59**

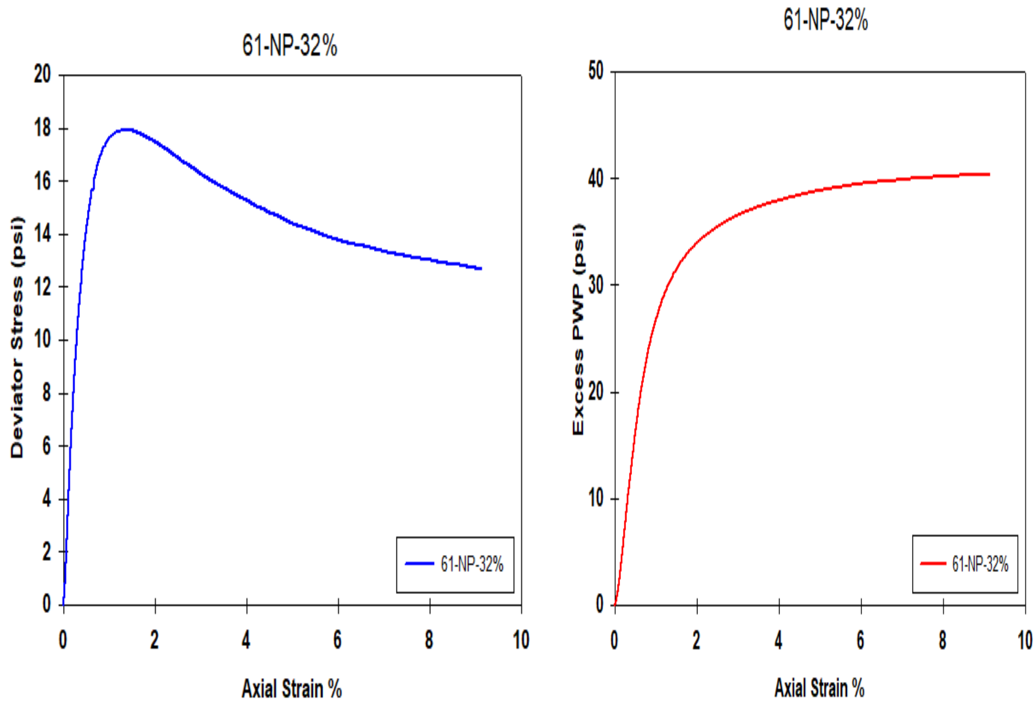



**Figure A-21. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 61**

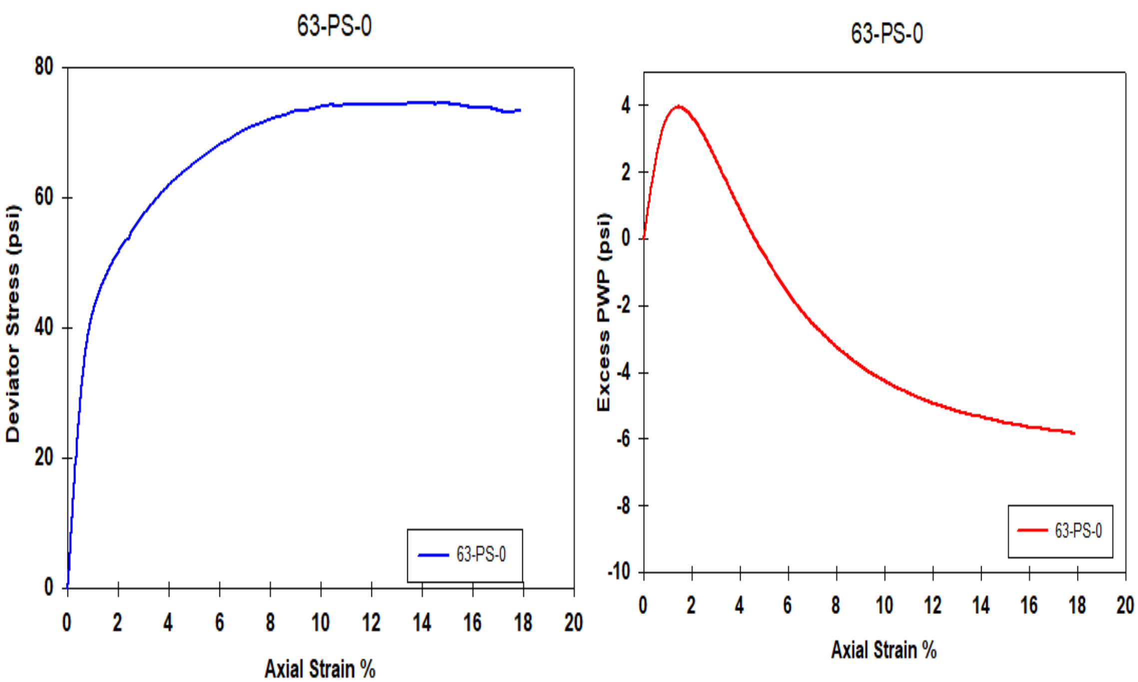



**Figure A-22. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 63**

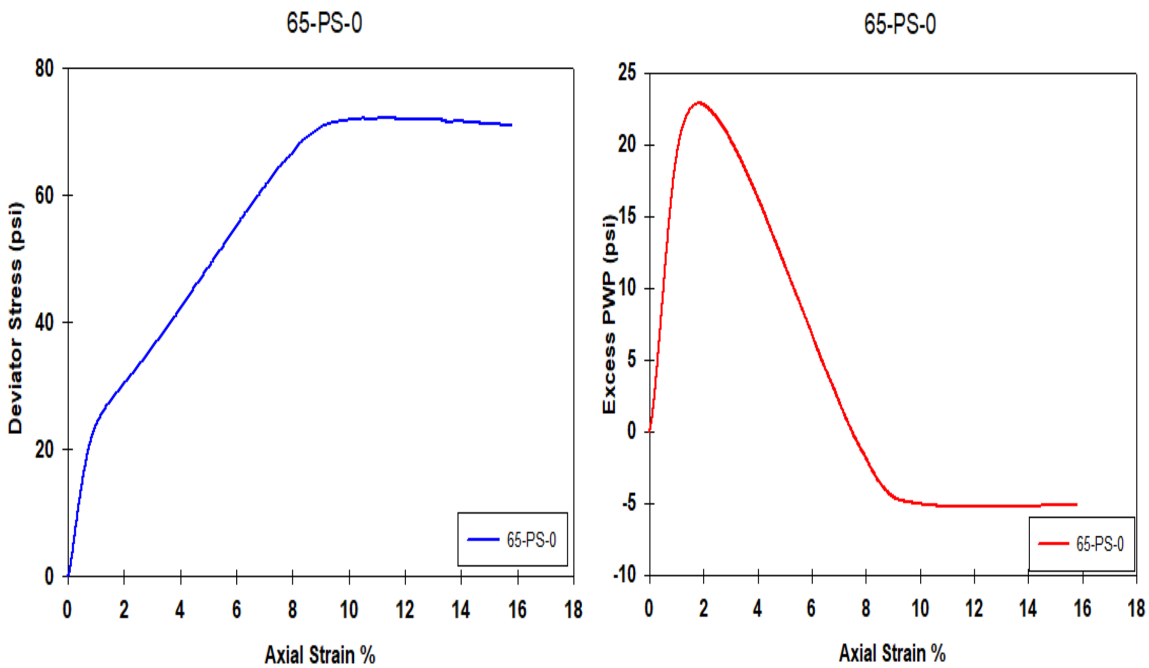



**Figure A-23. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 65**

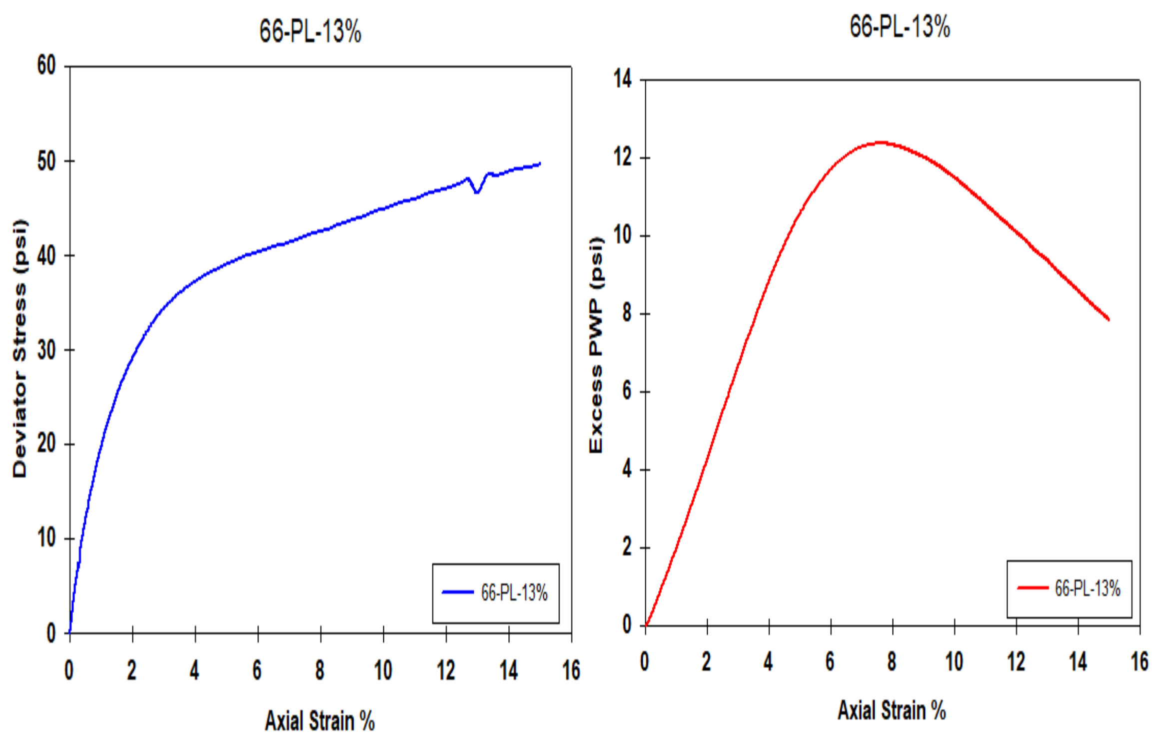



**Figure A-24. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 66**

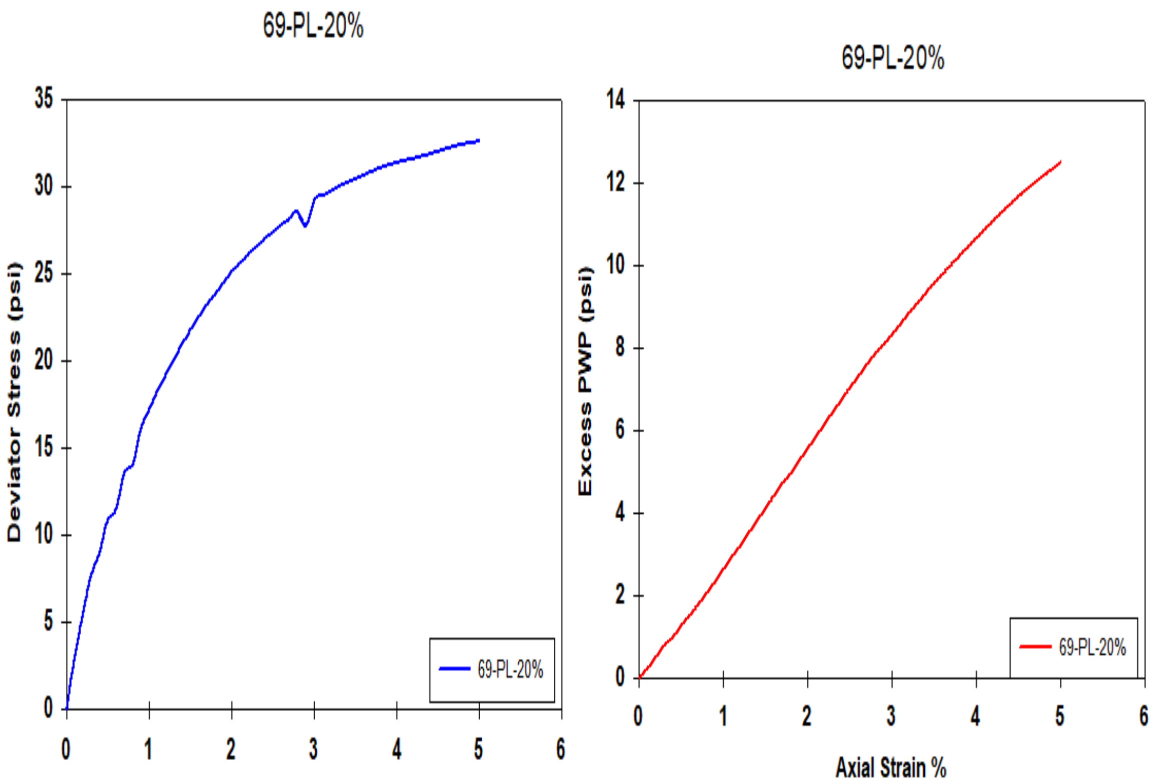



**Figure A-25. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 69**

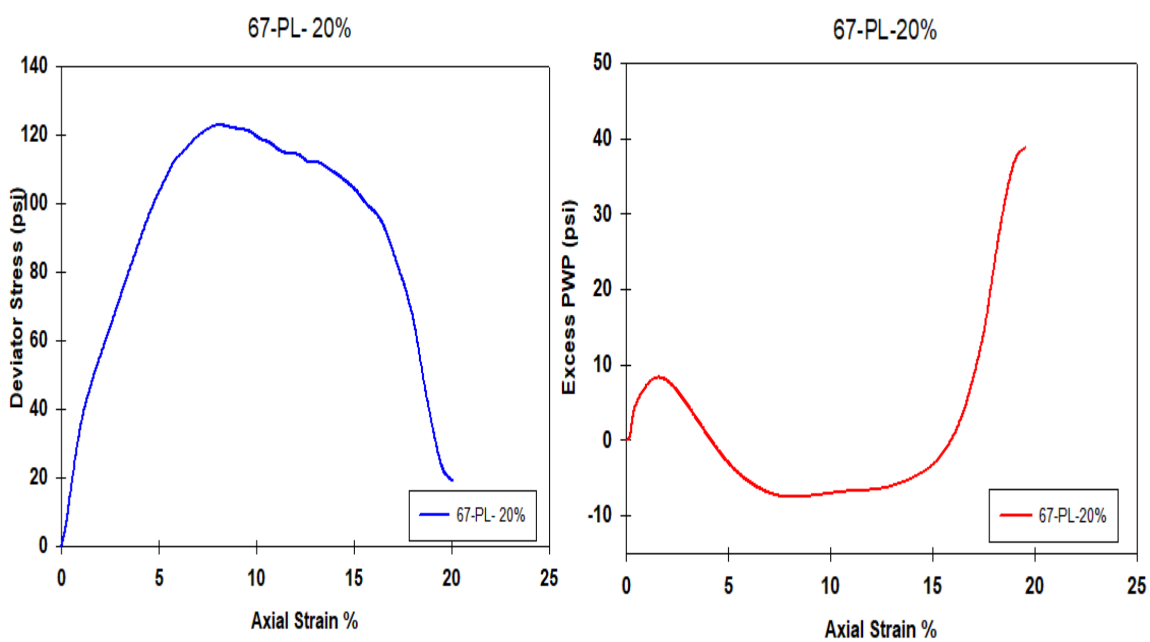



**Figure A-26. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 67**

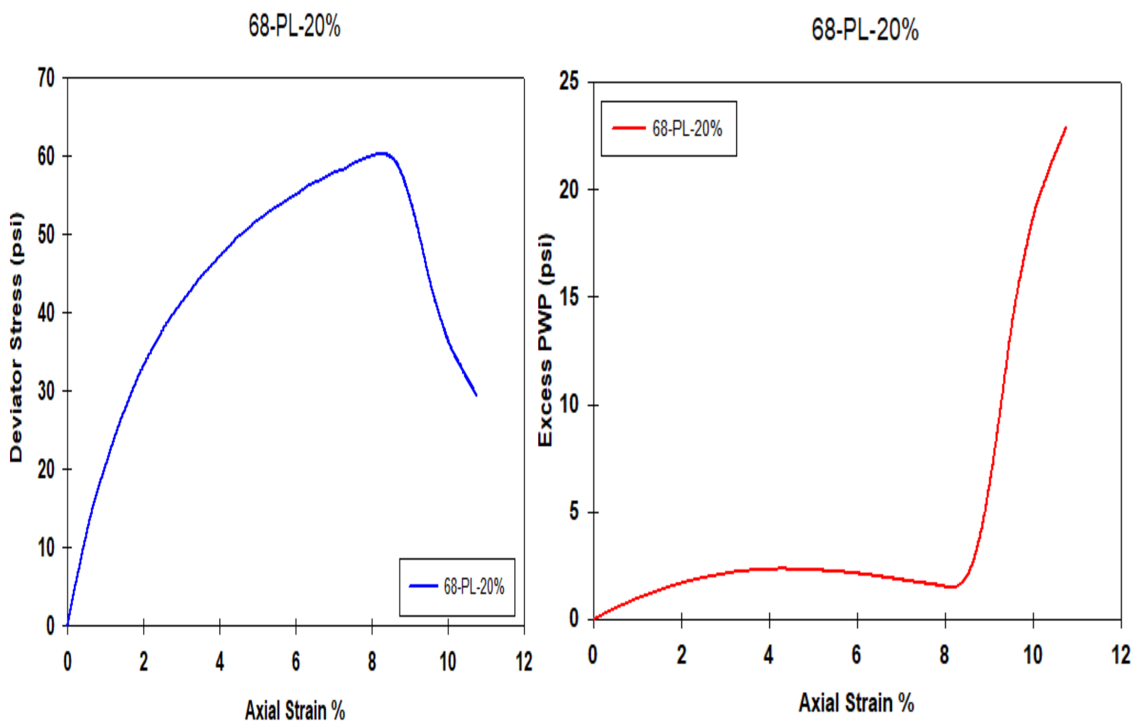



**Figure A-27. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 68**

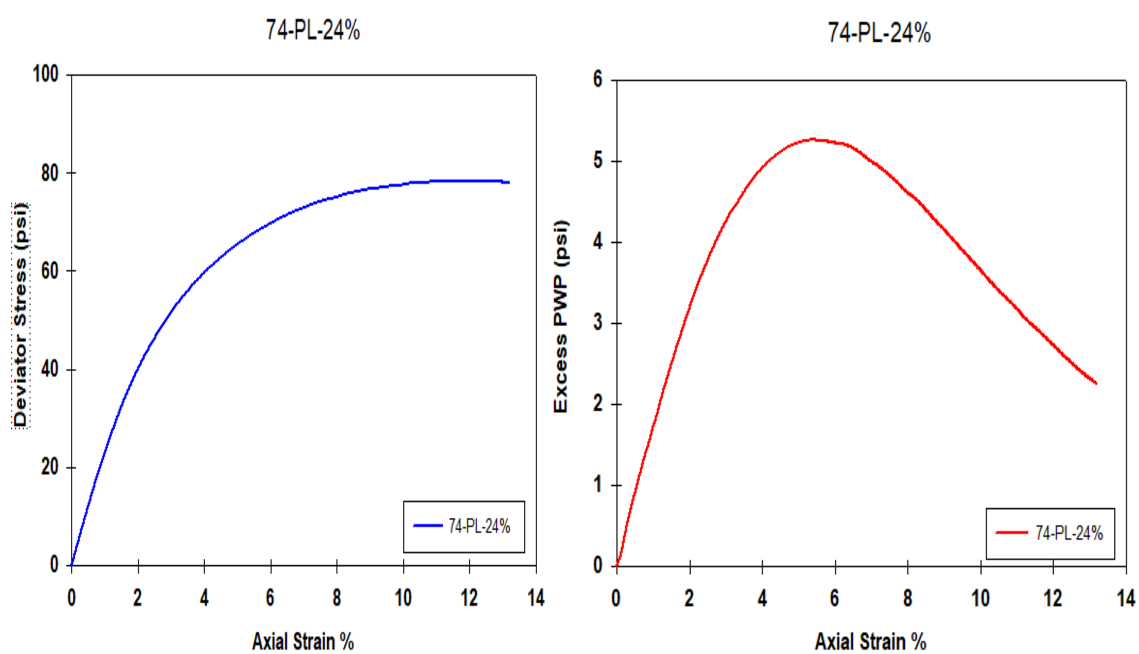



**Figure A-28. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 74**

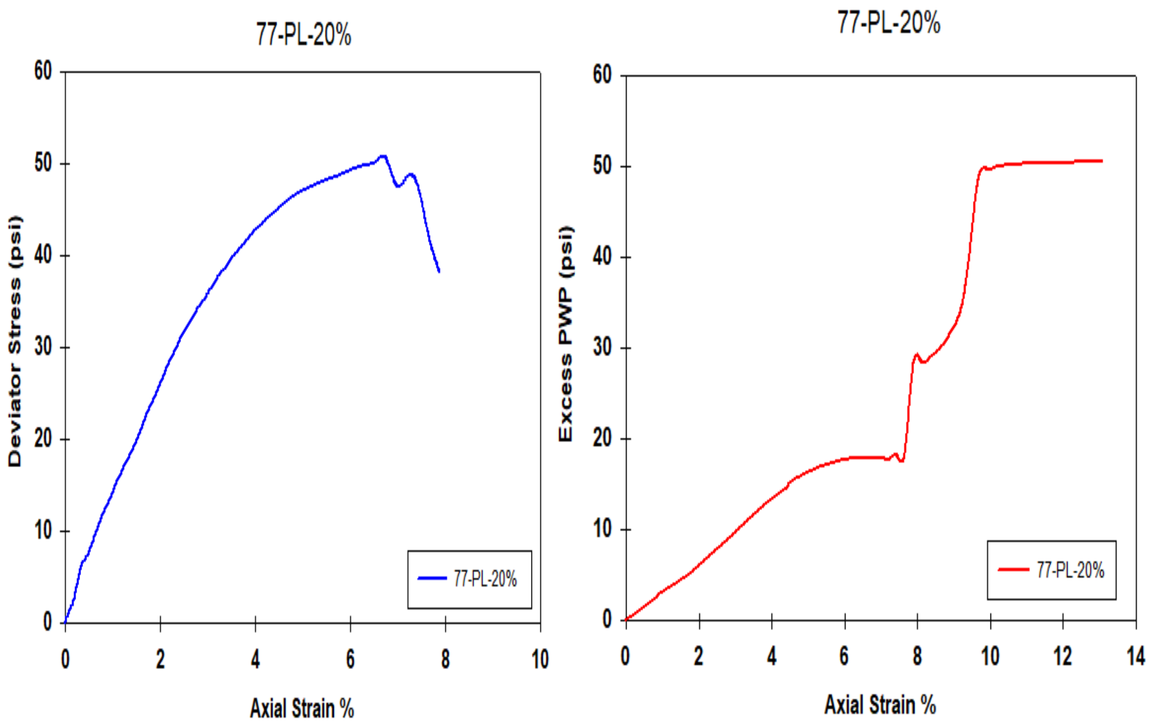



**Figure A-29. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 77**

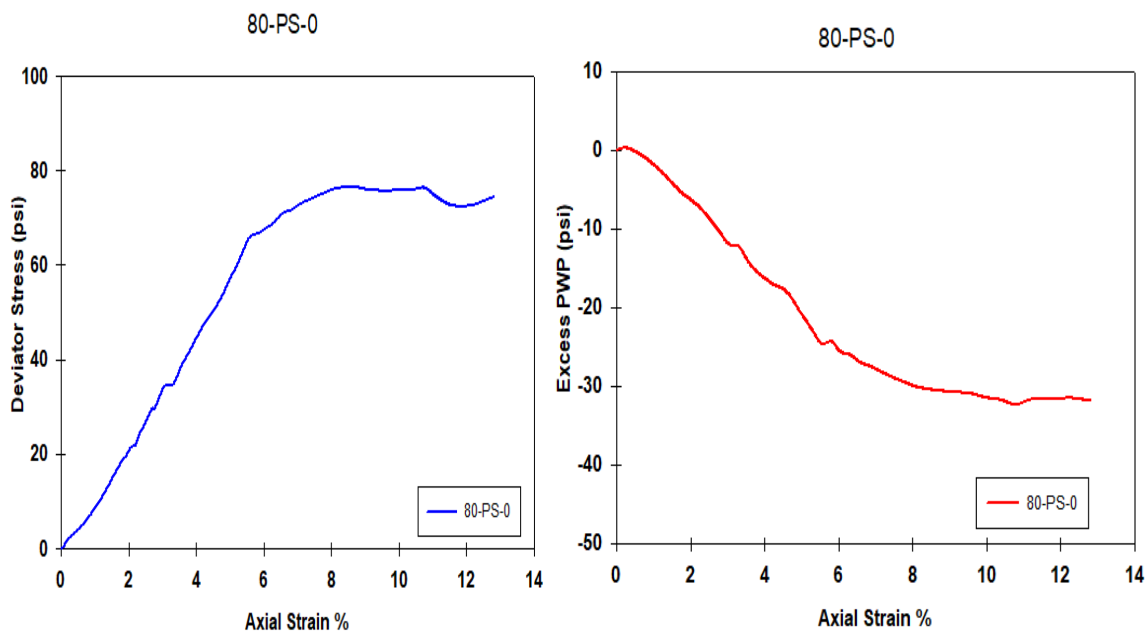



**Figure A-30. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 80**

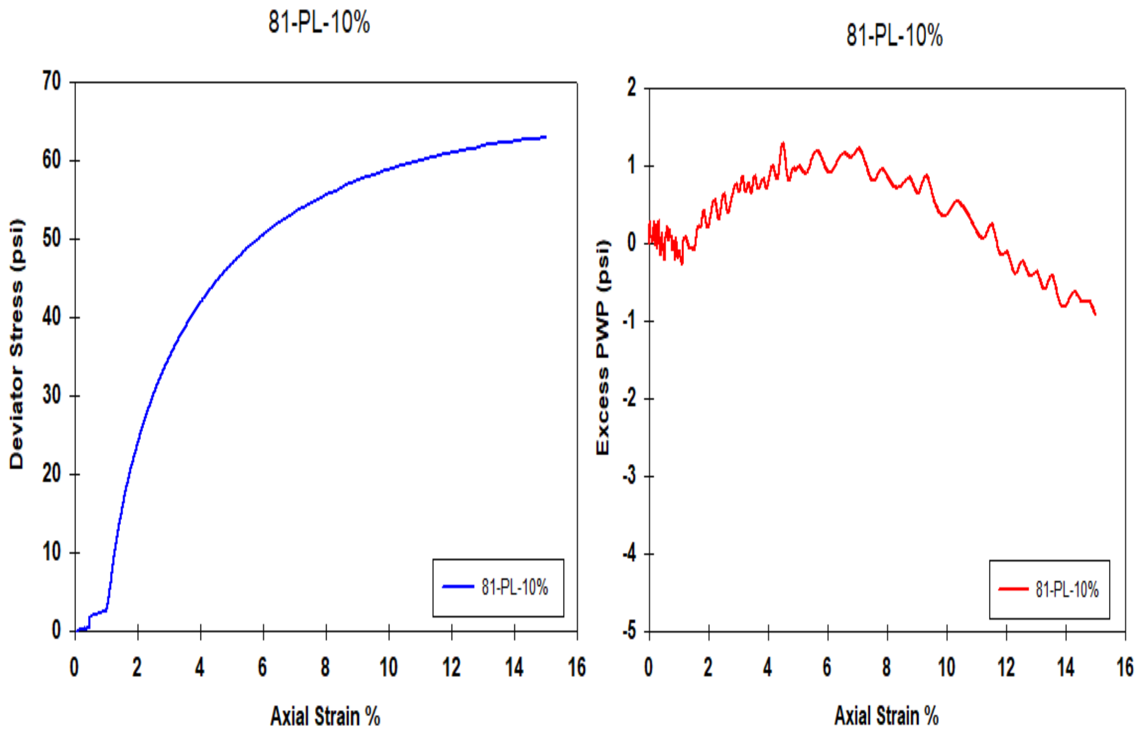



**Figure A-31. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 81**

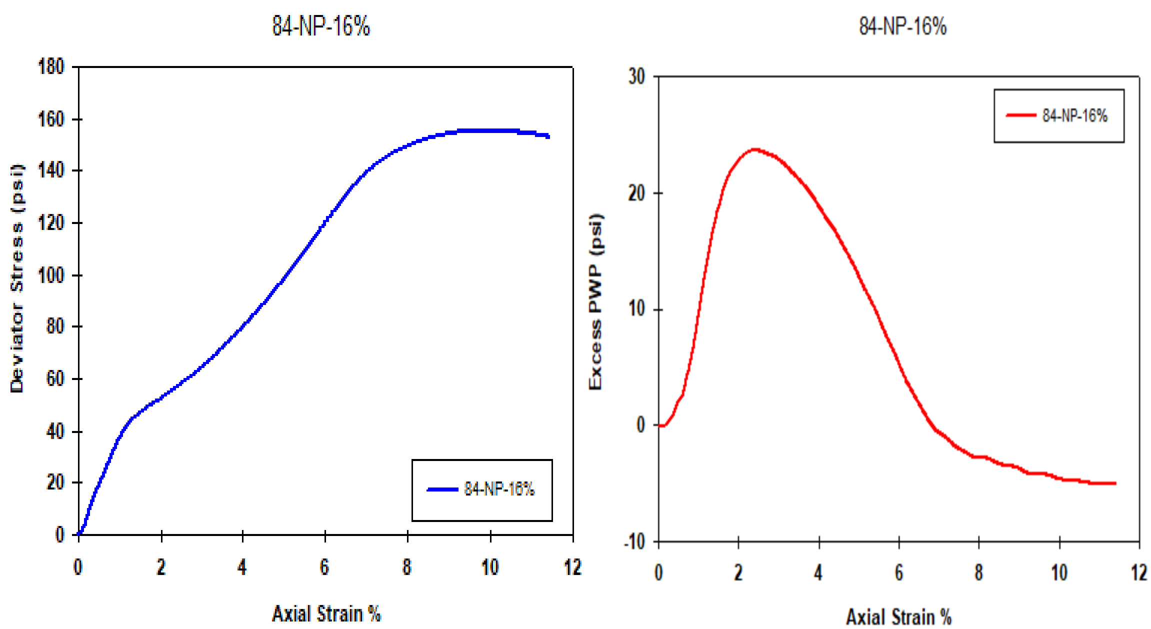



**Figure A-32. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 84**

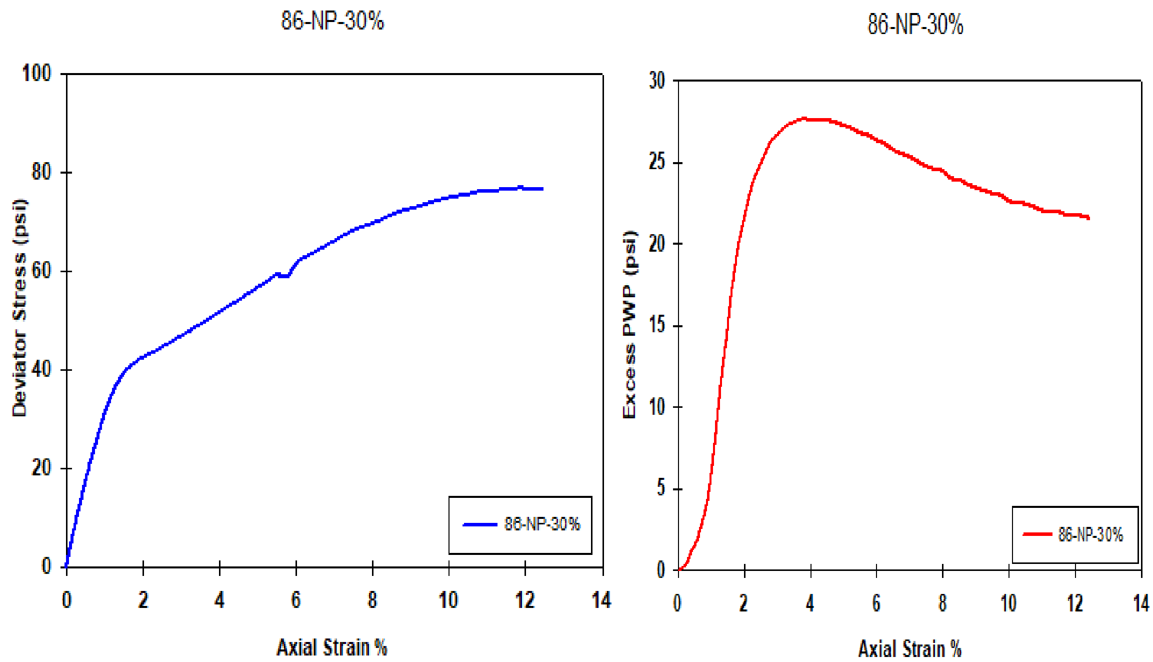



**Figure A-33. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 86**

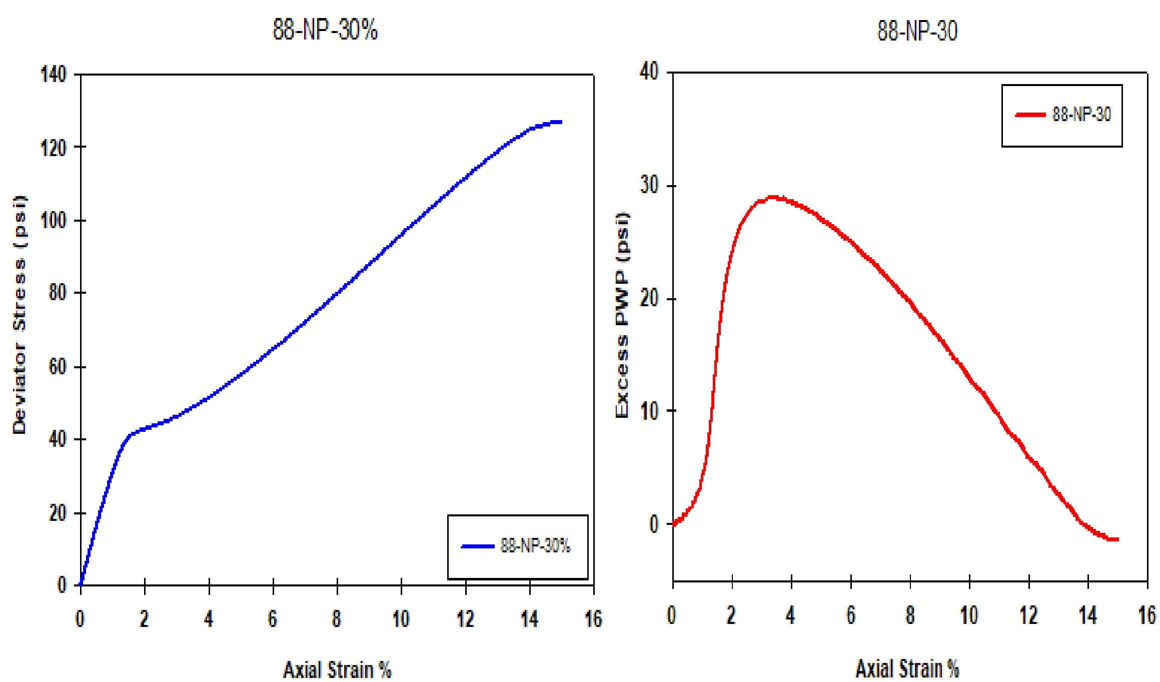
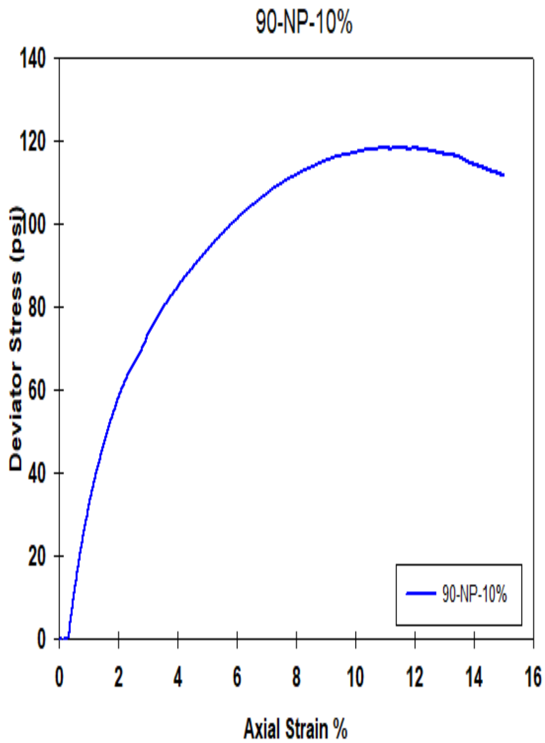



**Figure A-34. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 88**



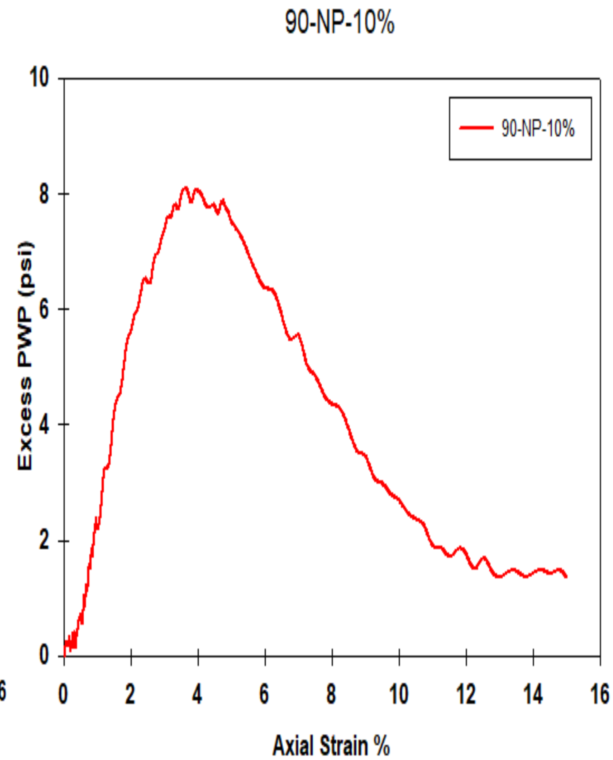




**Figure A-35. Deviator Stress (psi) and Excess PWP (psi) vs Axial Strain % for sample 90**