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Study of the Effect Of Heating - Cooling Cycles On the Clay - Concrete Pile Interface Characteristics

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STUDY OF THE EFFECT OF HEATING - COOLING CYCLES ON THE CLAY - CONCRETE PILE INTERFACE CHARACTERISTICS

A Thesis

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

in

The Department of Civil and Environmental Engineering

by

Abedalqader Ahmad Idries
B.Sc., Jordan University of Engineering and Technology, 2016
August 2020
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Abstract
This study aimed at investigating the effect of heating-cooling cycles on the interface strength parameters of clay soil-concrete for potential applications to improve the side capacity of piles driven in clayey soil by the heating process, and also to assess the behavior of geothermal piles regarding interface shear strength parameters. A modified large-size direct shear test device with the dimensions 12” x 12” x 8”, was used in this study. A concrete block (12” x 12” x 4”) of similar texture and smoothness to the concrete piles was used to simulate the pile interface. The tested clays are low plasticity index, P.I., clay (PI=12), medium plasticity index, P.I., clay (PI=30), and high plasticity index, P.I., clay (PI=60). All three types of clay soils were tested under one heating-cooling cycle of (20° - 70° - 20°) C temperatures. Only the low P.I. clay soil was also tested under two different heating-cooling cycles of (20° - 55° - 20°) C temperatures and (20° - 40° - 20°) C temperatures. In addition, it was tested under 16psi and four number of heating-cooling cycles of (20° - 70° - 20°) C, (20° - 55° - 20°) C, and (20° - 40° - 20°) C temperatures. Furthermore, the low P.I. clay soil was also tested under nine number of heating-cooling cycles of (20° - 55° - 20°) C temperatures under 16psi. A heating system was designed and used to apply up to 70°C temperature on the soil-concrete inside the direct shear test device. The experimental program includes shearing the three different types of clay soils for both conditions: without applying heating-cooling, and with applying heating-cooling cycles under three different normal stresses (10psi, 16psi, and 21.8psi), and also under 4.35psi normal stress for the low P.I. clay. After consolidation, the temperature for the heated specimens was increased gradually during the heating process from room temperature (22°C ±1°C) up to the target temperature, and then, during the cooling process, the specimens were cooled back to the room temperature. The test results of this study showed a significant increase of interface peak shear strength, cohesion, and the peak friction
angle of the heated high P.I. soil specimens, while insignificant change was observed in residual shear strength parameters. However, for low and medium P.I. clay soils, the results showed an increase in both peak and residual interface shear strength parameters. In addition, the increase in shear strength parameters of the low P.I. clay was found to be proportional with number of cycles and the target temperature.
CHAPTER 1.
INTRODUCTION

Studying the pile-clay interface is very important for evaluating the behavior of traditional piles and/or geothermal piles applications (geothermal piles, diaphragms, tunnels, etc) with regards to both shear strength and deformation. The equation describing the shear strength, normal stress, and friction angle at the of pile-soil interface is as follows:

\[ \tau = c_a + \sigma' \tan(\delta) \]  

where \( \tau \) is the shear strength at the pile-soil interface.

\( c_a \) is the adhesion at the pile-soil interface.

\( \sigma' \) is the effective normal stress over the pile-soil interface.

\( \delta \) is the friction angle at the pile-soil interface.

As shown in the above equation, the relationship between the shear strength and the applied effective normal stress and between the shear strength and the friction angle (\( \delta \)) is directly proportional. In the case of geothermal energy structures, the effective normal stress on the interface is the lateral load on the foundation. The lateral load depends mainly on the depth of the foundation, the density or unit weight of the surrounding soil, the saturation condition, the existence of a surcharge load, and the lateral condition (passive, active or at rest). The interaction between the pile and the adjacent soil is very complex and depends on many factors such as the type of soil and its engineering characteristics (i.e. shear strength, water content, particle size, permeability…, etc.), the type, material, surface roughness, and geometry of the pile. Moreover, unique phenomena like pile setup, downdrag, and drag load play an important role in pile-soil strength.
According to Yazdani et al. (2019-a), the two most significant factors affecting the behavior and failure mechanism of the interface are the soil average grain size and roughness of the surface. According to Di Donna (2014) and Yazdani et al. (2019-a), the structure roughness is commonly described as a normalized roughness $R_n$, defined as:

$$R_n = \frac{R_{\text{max}}}{D_{50}}$$

where $R_{\text{max}}$ is the max vertical distance between the lowest and highest peaks of the surface asperities over a horizontal distance $L = D_{50}$.

$D_{50}$ is the mean grain size of soil.

According to Di Donna (2014), the critical roughness $R_{cr}$ is the value of roughness at which any value of roughness less than $R_{cr}$ represent a smooth surface, while any value higher than $R_{cr}$ represent a rough surface. According to Yazdani et al. (2019-a), for the mechanism of interface failure, there are three classified zones with regards to normal roughness. These are the smooth zone, the intermediate zone, and the rough zone. For the smooth zone, the governing failure mechanism is the sliding of soil particles at the interface. The shear strength of the interface will reach an almost constant value with decreasing normalized roughness. Whereupon, at very small values of normalized roughness, the shear strength of the interface will be independent of normalized roughness. For the intermediate zone, the failure mechanism is related to sliding at the surface and the shear deformation of the soil. In addition, in this zone, increasing $R_n$ will induce a gradual increase in shear strength of interface until reaching a critical value of $R_n$ where the associated strength of this value reaches the shear strength of the soil itself. The rough zone will take place where $R_n$ is greater than the critical $R_n$ by 0.06 to 0.03. In this case, within the soil, a shear band will be formed, and failure will occur in a rough zone. It was also reported from previous experimental results that the shear strength of the interface is proportional to the
roughness of the surface (Di Donna, 2014). However, it is always less than the shear strength of the soil. According to Schofield and Wroth (1968), under shearing, the response of a smooth surface is elastic-perfectly plastic, while the response of a rough surface is close to the response of the soil and is analyzed based on critical state theory.

The interface shearing behavior of sand-solid is different from clay-solid. Shearing in a sand-solid interface is related to the turbulent shearing where translating and rolling of particles are the main factors for the shearing behavior of sand. However, within the shear zone, particle orientation is the main factor in clays due to its platy shape (Yazdani et al., 2019-a).

Evaluating the shear strength could be done by several methods, starting with correlations between shear strength and pile capacity with cone penetration tests (CPT) and standard penetrations (SPT) tests data, small scale laboratory tests, large scale laboratory tests, up to a full-scale testing of actual piles in the field.

Nevertheless, in the past few decades, geothermal piles have been a hot topic because of its cost effective and gas emission reduction strategies. The only difference between conventional piles and geothermal piles is that a certain type of looping system consisting of pipes is inserted along the pile length of geothermal piles and connected to a ground source heat pump. Usually antifreeze liquid is being circulated through the pipes, performing heat exchange within the piles which in return exchanges heat with the adjacent soil. Therefore, temperature change is induced within the pile and the surrounding soil, in which case the need arises for studying the effect of heat variation of the pile-soil interface and its impact on pile capacity (i.e. interface shear strength) due to geothermal applications. Another objective of this study is to evaluate the use of heating-cooling cycles as a pile-soil improvement technique of the deep foundation.
1.1. Problem Statement

There is a lack of research about the effect of heating-cooling cycles under different temperatures, including high temperatures (i.e. higher than temperatures tested to simulate geothermal piles), on the improvement of pile capacity. Intentionally heating a pile and cooling it down to improve its capacity or as an act of treatment in cases of a pile driven in weak soils is, to the best of the author’s knowledge, not mentioned in literature. In addition, almost all the studies that are related to the same context of this study were performed using small-size direct shear devices, which are not as reliable as the large-size direct shear device when it comes to simulating pile capacity. Furthermore, some researchers did try to relate the performance of clay due to thermal loading with the plasticity index by analyzing separate data from the literature. However, the number of researchers who did perform tests with different plasticity indices is low.

Evaluating and assessing the performance of geothermal piles due to heating and cooling is of a great importance. However, there is a scarcity of studies performed in Louisiana or using Louisianan soil. This research study is intended to use three different types of Louisianan clays with different plasticity indices (low PI, medium PI, and high PI).

1.2. Research Objectives and Approach

The ultimate goal of this study is to evaluate the effect of heating-cooling cycles over the shear strength parameters of the clay-concrete interface. Furthermore, to assess the improvement of pile capacity after heating-cooling cycles and the performance of geothermal piles. The approach consists of performing direct shear tests in a large-size direct shear device (12’’ x 12’’ x 8’’) on three different types of clays, each with a different plasticity index. The tested clays are low P.I. clay (P.I.=12), medium P.I. clay (P.I.=30), and high P.I. clay (P.I.=60). To accelerate testing, only clay-clay shear strength parameters were evaluated using the small-size direct shear device.
The experimental program was executed in two phases. In Phase 1 of this study, all three types of clay were tested under three different normal stresses of 10psi, 16psi, and 21.8psi, except for the low P.I. clay, which was also tested under a fourth normal stress of 4.35psi. All tests were performed with and without one heating-cooling cycle (i.e. temperature was raised from around 20°C to 70°C and cooled down to 20°C). Table 1.1 shows the factorial table of phase 1 testing.

<table>
<thead>
<tr>
<th>Clay Type</th>
<th>Plasticity Index</th>
<th>Normal Stress (psi)</th>
<th>Number of cycles</th>
<th>Temperature, °C.</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low P.I.</td>
<td>12</td>
<td>4.35</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.35</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.8</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.8</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td>Mediu m P.I.</td>
<td>30</td>
<td>10</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.8</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.8</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td>High P.I.</td>
<td>60</td>
<td>10</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.8</td>
<td>1</td>
<td>20°C -70°C -20°C</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21.8</td>
<td>0</td>
<td>20°C</td>
<td>1</td>
</tr>
<tr>
<td>Total tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20</td>
</tr>
</tbody>
</table>

Phase 2 consists of evaluating the effect of more than one heating-cooling cycle and using different ultimate temperatures (40°, 55°, and 70°) Con only low P.I. clay soil. In phase 2, the low P.I. clay was tested under normal stresses of 10psi, 16psi, and 21.8psi and under two different heating-
loading cycles of (20° - 55° - 20°) C temperatures and (20° - 40° - 20°) C temperatures. In addition, it was tested under 16psi and four number of heating-cooling cycles of (20° - 70° - 20°) C, (20° - 55° - 20°) C, and (20° - 40° - 20°) C temperatures. Lastly, it was tested under nine number of heating-cooling cycles of (20° - 55° - 20°) C temperatures under 16psi. Table 1.2 shows the factorial table of phase 2 testing.

Table 1.2. Phase 2 factorial table.

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Normal stress (psi)</th>
<th>No. of Cycles</th>
<th>No. of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>40°C</td>
<td>10</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>21.8</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>55°C</td>
<td>10</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>21.8</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>70°C</td>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>21.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>10</strong></td>
<td></td>
</tr>
</tbody>
</table>

1.3. Outline

The outline for this Thesis is as following: Chapter 2 presents literature review for the previous work on the behavior of clay soils under thermal loading and on the effect of heating-cooling cycles on the soil-concrete interface shear strength parameters. Chapter 3 presents the testing and heating methodology followed in this study, and the direct shear results for low P.I. clay soil-concrete interface under one heating-cooling cycle at 70°C. Chapter 4 presents the direct shear
results for medium P.I. clay soil-concrete interface under one heating-cooling cycle at 70°C. Chapter 5 presents the direct shear results for high P.I. clay soil-concrete interface under one heating-cooling cycle at 70°C. Chapter 6 presents the direct shear results for low P.I. clay soil-concrete interface under one heating-cooling cycle at 40°C and at 55°C. Chapter 7 presents the direct shear results for low P.I. clay soil-concrete interface under multiple heating-cooling cycles under 16psi at 40°C, 55°C and at 70°C. Chapter 8 presents the conclusions and recommendations.
Chapter 2.
LITERATURE REVIEW

2.1. Introduction

The number of experiments dealing with thermo-mechanical behavior on sand is low. This is due to the limited temperature variation effect over sand’s deformation and shear strength and due to its high permeability (Di Donna, 2014). The behavior of granular soil under thermal load is thermo-elastic and the deformation is reversible. It contracts and expands elastically under cooling and heating, respectively. When fully saturation condition presents, water and granular material dilate (Di Donna, 2014).

Many experiments were performed to study the effect of heat variation on clay's properties, in which most of the conducted tests showed the same behavioral observation. The observation is that for a normally consolidated (NC) clay material subjected to heating, it experiences contraction, while a slightly overconsolidated (OC) material shows initial dilation followed by a contraction in heating and contraction in cooling. Furthermore, highly OC material experiences an expansion during heating, as shown in Figure 2.1. For NC and slightly OC clay, the material shows irrevocable deformation under thermal loading at constant mechanical load (Laloui and Di Donna, 2013; Laloui, 2001). Under cooling following the heating for NC clay, some researchers observed an elastic contraction (Abuel-Naga et al., 2006: Uchaipichat and Khalili, 2009), while others observed partial expansion recover (Coccia and McCartney, 2011). A recoverable deformation was noticed for a highly OC material (i.e. OC more than 1.5 to 3) (Baldi et al., 1988; Hueckel and Baldi, 1990; Towhata et al., 1993).

According to Laloui and Di Donna (2013), Laloui (2001), Abu-elnaga (2006-a, 2006-b, 2007), Coccia and McCartney (2011), Burghignoli et al. (2000) and others, the suggested explanation is
that the apparent preconsolidation pressure decreases with increasing temperature at a constant void ratio. In contrast, the applied mechanical load or the maximum historical load is constant. The theoretical schematized framework behind that could be demonstrated through plotting the isotropic preconsolidation pressure (mean effective stress - $\rho'$) versus temperature (T) to demonstrate the evolution of apparent preconsolidation pressure, as shown in Figure 2.2.

where; $\rho' = \frac{\sigma'' + 2\sigma''}{3}$

Figure 2.1. Volumetric strain under drained heating/cooling cycle of soft Bangkok clay (Abuel-Naga et al., 2006-a).

Figure 2.2. Effect of OCR and temperature on an induced volumetric strain of soft Bangkok clay (Abuel-Naga et al., 2006-a).

The reduction in the apparent preconsolidation pressure with increasing temperature is known as thermal softening. However, the thermal path that induces plasticity will also induce an increase in elastic domain (strain hardening). In addition, if a NC clay experienced an increase in mechanical loading that will also increase the elastic domain (Laloui, 2001).

Abuel-Naga et al. (2007) observed the same volume change behavior for NC, OC, and slightly OC clayey soils (based on stress history). They suggest that this behavior under thermal loading is due to the viscous shear resistance and inter-particle forces of the adsorbed water, which affect the resistance of the clay particles to fabric changes. However, in their findings, they concluded that Roscoe surface geometry, elastic zone with a constant plastic strain, and the flow rule are all
temperature dependent, and that drained, undrained shear strength, stiffness, are also temperature dependent with a proportional relationship. However, critical state line, compression line ($\lambda$), and swelling line ($\kappa$) in q-p space are not dependent on temperature. In addition, they observed volume change depends on stress history and that heating increased hydraulic permeability, stiffening, increased the apparent overconsolidation state after thermal cycles, and decreasing the conventional elastic zone.

### 2.2. Impact of Temperature on Preconsolidation Pressure

Tidfors and Sallfors (1989) conducted several conventional oedometer tests and constant-rate-of-strain (CRT) tests on five different clays with different composition, stress history (from 1 to over 5 OCRs), and geological history. They observed a clear reduction of preconsolidation pressure with increasing temperature, this reduction increases with increasing clay contents. Figure 2.3 shows the result of changing temperature during a CRT test and Figure 2.4 shows the reduction in preconsolidation pressure with time in a best fit straight line by the least square method.

![Figure 2.3. CRS test with varying temperature. Clay from Backebol (Tidfors and Sallfors, 1989).](image1)

![Figure 2.4. Preconsolidation pressure as a function of test temperature for specimens taken at 7 m depth at Biickebol. Full line shows results of linear regression analysis (Tidfors and Sallfors, 1989).](image2)
Experiments performed on saturated Illite showed that when the sample is heated before loading, densification occurs for the soil under the same isotropic pressure. Furthermore, by testing samples in a consolidation test under different temperatures, it was found that preconsolidation non-linearly decreases with increasing temperatures, and this phenomenon is not related to viscous effect (Laloui, 2001).

Abuel-Naga et al. (2007) related the reduction in the elastic zone size with temperature to the preconsolidation pressure evolution at constant plastic strain condition as a result of heating, as shown in Figure 2.5. Figure 2.5 shows a decrease in elastic zone size with increasing constant stress ratio $\eta$. A decrease in elastic zone size with increasing constant stress ratio $\eta$ (the ratio between the deviatoric stress to the mean effective stress) was found (Laloui, 2001).

Elasticity domain expands when the soil temperature is lowered and shrinks when it raised. Another important factor in this process or phenomenon is the thermal evolution of the critical state locus (i.e., variable $M$).

![Figure 2.5. Temperature effect on the size of the elastic zone at constant plastic strain condition as a function of constant stress ratio $\eta$ (Abuel-Naga et al., 2006-b).](image-url)
In the isotropic mean effective stress $P'$ vs. deviatoric stress invariant $q$ space, the yield locus in the thermal cam-clay model is considered either as an ellipse (modified Cam-clay locus) as shown in equation 2 or logarithmic function (original Cam-clay locus) as shown in equation 3 (Hueckel et al., 2009).

$$f = P'^2 - P'P'_c + \left(\frac{q}{M}\right)^2 = 0$$  \hspace{1cm} (2)

$$f = \frac{q}{MP'} + \ln \left(\frac{2.718P'}{P'_c}\right) - 1 = 0$$  \hspace{1cm} (3)

where $P'_c$ is the apparent preconsolidation pressure and it denotes the size of the locus and it is a temperature dependent during cooling and heating. Regarding temperature, the considered factor is $\Delta T$, which is the difference between the material temperature and $T_0$, which is the temperature at $P' (\Delta T=0) = P'_c o$ and the measurement of all parameters is at $T_0$. Moreover, there is experimental evidence that shear strength is thermally dependent by a change of critical state with temperature. For example, experiments showed that boomed clay and kaolinite are thermal dependents, but Pontida clay is not, by variation of $M(\Delta T)$. This dependence of on the slope $M$ is material specific (Hueckel et al., 2009).

2.3. Volume of Drained Water Under Fully Saturation Condition

According to Campanella and Mitchell (1968), the volume of drained water due to changing temperature under constant effective stress and fully saturation conditions is given as follows:

$$(\Delta V_{DR})_{\Delta T} = (\Delta V_w)_{\Delta T} + (\Delta V_s)_{\Delta T} - (\Delta V_m)_{\Delta T}$$  \hspace{1cm} (4)

where:

$$(\Delta V_w)_{\Delta T}:$$ the pore water volume change, which is expressed by:

$$(\Delta V_w)_{\Delta T} = \alpha_w V_w \Delta T$$  \hspace{1cm} (5)

$\alpha_w$: the pore water coefficient of thermal expansion.
$V_w$: the pore water volume.

$\Delta T$: the change in temperature.

$(\Delta V_s)_{\Delta T}$: mineral solids volume change, which is expressed by:

$$(\Delta V_s)_{\Delta T} = \alpha_s V_s \Delta T \tag{6}$$

$\alpha_s$: the mineral solids coefficient of cubical thermal expansion.

$V_s$: the mineral solids volume.

$(\Delta V_m)_{\Delta T}$: the soil specimen volume change, which is expressed by

$V_m$: the soil specimen volume.

Campanella and Mitchell (1968) presented the following relationship of soil specimen volume change based on their assumption that volumetric strain and $\alpha_s$ (the thermal expansion coefficient) is the same for all solid minerals and soil specimens. In addition, they accounted for another kind of volume change due to reorientations of interparticle forces as a result of changing temperature, which is denoted by $(\Delta V_{st})_{\Delta T}$.

Therefore, $(\Delta V_m)_{\Delta T} = \alpha_s V_m \Delta T + (\Delta V_{st})_{\Delta T} \tag{7}$

Combining Eqs. 3, 4, 5, and 6 gives

$$(\Delta V_{DR})_{\Delta T} = \alpha_w V_w \Delta T + \alpha_s V_s \Delta T - \alpha_s V_m \Delta T - (\Delta V_{st})_{\Delta T} \tag{8}$$

However, according to Baldi, et al. (1987), adsorbed water expansion is much lower than the thermal expansion of free water. They concluded that for low permeability soils, the change of pore water volume due to thermal loading is also dependent on electrical microstructural or electrochemical interaction. In contrast to Campanella and Mitchell (1968) concept of total dependence on the thermal expansion of free water.
2.4. Developed Concept Under Undrained Condition

The reason behind the generation of excess pore water pressure under heating is the difference between the coefficient of thermal expansion of water and soil particles. Therefore, resulting in generated excess pore water pressure under undrained conditions. It also results in an excess of pore water pressure dissipation under drained condition leading to an irrecoverable volumetric deformation (Coccia and McCartney, 2011; Di Donna, 2014). Campanella and Mitchell (1968) concluded that under an undrained condition with varying pressure and temperature, the soil specimen volume change must equal the soil constituent’s volume change (i.e. water and mineral solids) as shown in the following relationship:

\[
(\Delta V_w)_{\Delta T} + (\Delta V_s)_{\Delta T} + (\Delta V_w)_{\Delta P} + (\Delta V_s)_{\Delta P} = (\Delta V_m)_{\Delta T} + (\Delta V_m)_{\Delta P}
\]  

(9)

where:
\( \Delta P \): the change in pressure.

\[
(\Delta V_w)_{\Delta P} = m_w V_w \Delta u
\]

(10)

\( m_w \): the water compressibility.
\( \Delta u \): water pressure change.

Campanella and Mitchell (1968) divided the change in the volume of solid minerals into two parts. The first part is due to the pore water pressure and the second part is due to intergranular stress changes. These two parts are described in the following equation:

\[
(\Delta V_s)_{\Delta P} = m_s V_s \Delta u + m_s' V_s \Delta \sigma'
\]

(11)

where, \( m_s \): the mineral solids compressibility under uniform or equal pressure from all sides.

\( m_s' \): the mineral solids compressibility under concentrated loading.
\( \Delta \sigma' \): the change in effective stress (intergranular stress).

Also, \((\Delta V_m)_{\Delta P} = m_v V_m \Delta \sigma'\)  

(12)
\(m_v\): the soil structure or soil mass compressibility.

Substituting equations 5, 6, 10, 11, 12 with Eq. 9 gives:

\[
\alpha_w V_w \Delta T + \alpha_s V_s \Delta T - (\Delta V_m)_{\Delta T} = m_v V_m \Delta \sigma' - m_w V_w \Delta u - V_s (m_s \Delta u + m'_s \Delta \sigma') \quad (13)
\]

Taking into consideration that \(\Delta \sigma' = -\Delta u\) in the case of constant total stress leads to:

\[
\alpha_w V_w \Delta T + \alpha_s V_s \Delta T - (\Delta V_m)_{\Delta T} = -m_v V_m \Delta u - m_w V_w \Delta u - \Delta u V_s (m_s - m'_s) \quad (14)
\]

Since \(m_s - m'_s\) is very small and that \(m_v\) and \(m_w\) are much larger than both \(m_s\) and \(m'_s\), it is reasonable to assume \(m_s - m'_s = 0\). In this way Eq. 14 becomes as follows:

\[
\alpha_w V_w \Delta T + \alpha_s V_s \Delta T - (\Delta V_m)_{\Delta T} = -m_v V_m \Delta u - m_w V_w \Delta u \quad (15)
\]

Combining Eqs. 7 and 15 and since \(V_m = V_w + V_s\) leads to:

\[
\alpha_w V_w \Delta T - \alpha_s V_w \Delta T - (\Delta V_{st})_{\Delta T} = -m_v V_m \Delta u - m_w V_w \Delta u \quad (16)
\]

The porosity, \(n = \frac{V_o}{V_m}\), and in the case of saturated soil, porosity becomes \(n = \frac{V_w}{V_m}\). With this fact and rearranging Eq. 16 to find pore water pressure as a result of changing temperature leading to the following equation:

\[
\Delta u = \frac{n \Delta T (\alpha_s - \alpha_w) + (\Delta V_{st})_{\Delta T}}{m_v - n m_w} = \frac{n \Delta T (\alpha_s - \alpha_w) + \alpha_{st} \Delta T}{m_v - n m_w} \quad (17)
\]

where, \(\alpha_{st}\): the physico-chemical structural volume change coefficient.

Since \(m_v\) is much larger than \(n m_w\) for most soils, then Eq. 17 can be simplified into Eq. 18

\[
\Delta u = \frac{n \Delta T (\alpha_s - \alpha_w) + \alpha_{st} \Delta T}{m_v} \quad (18)
\]

In Eq. 20, the signs of \(\alpha_s\) and \(\alpha_w\) are positive if the sign of \(\Delta T\) is positive. The signs of \(m_v\), \(m_w\), and \(\alpha_{st}\) are also positive if the sign of \(\Delta T\) is positive.
2.5. Microscopic

According to Tidfors and Sallfors (1989), there is no strongly bound water surrounding the clay particles, and no particle-to-particle contact exists between clay particles. With increasing temperature, the strength of strongly bound water reduces, and the double layer becomes thinner leading to a deformation and vice versa. However, after more cycles, the strength increases and the recoverable thickness of the double layer becomes partial. Creep rate increases with increasing temperature and vice versa with reducing temperature.

Di Donna (2014) reported that the double layer thickness should not be affected by temperature based on Gou-Chapman equations. Therefore, at particle scale, either the prediction of interaction forces is failed by the theory of the double layer, or that other phenomenon is responsible for the thermal effect. According to Morin and Silva (1984), at high temperature, the double layer thickness is reduced, but Mitchell and Soga (2005) concluded that temperature does not affect the diffused double layer. Di Donna (2014) also reported that the generation of irreversible strains (or clay thermal contraction) is due to organized water disruption, which is the reason for creating mineral to mineral connection over water to mineral connections. Pusch (1986) observed a permanent contraction of some of the stacks at high temperatures. Pusch (1986) also associated the contraction of clays, while heating, to the closer tendency of layers to group. Di Donna (2014) reported that most studies regarding thermo-mechanical behavior of clays were performed on smectite clays, which shows a higher macroscopic effect than for kaolinite or illite clays. Besides, for illite clays, potassium cations are filling the space between layers. Therefore, it is not easy to study the effect of thermo-mechanical behavior of illite clays at the microscopic scale. This is why Abuel-Naga et al. (2006) linked the thermal deformation of clays to the plasticity index.
At the contacts of inter-particle, the frictional strength decreases when energy is provided to the material (increasing temperature), which results in a partial collapse of soil structure and a decrease in void ratio. After this, additional contacts are created between particles to carry the imposed higher temperature. When the number of the created contacts are sufficient, an equilibrium state is reached again for the inter-particle.

For NC clays, the condition of the contacts is closer to failure and the predominant effect regarding the thermo-elastic expansion of soil constituents is the rearrangement of particles. This is in contrast to the case of OC clays, where the contacts are in a more stable condition and, the predominant effect is the thermo-elastic expansion of soil constituents and a limited rearrangement of particles effect. The slippage potential of bonds is higher for the contacts closer to failure. Therefore, slippage potential of bonds for NC clays in higher than that of OC clays. (Di Donna, 2014).

To explain the induced volume change by thermal effect under fully saturation concept, Abuel-Naga et al. (2006-b) adopted the series effective stress model by which an effective stress change \( \Delta \sigma' \) is described by net electrochemical forces as shown in the following equation:

\[
\Delta \sigma' = \Delta \sigma - \Delta u = \Delta (R_{DL} - A_{tt})
\]  

(21)

where, \( \Delta (R_{DL} - A_{tt}) \) is the net electrochemical forces, \( R_{DL} \) is the repulsive interparticle forces, and \( A_{tt} \) is the attraction interparticle forces.

According to the same model, the change in electrochemical force is the difference between total stress and pore water pressure. In addition, the induced strain from electrochemical forces is perfectly reversible, but strain can be irreversible, this is due to strain additivity.
Following the same approach by Abuel-Naga et al. (2006-b), the reversible expansion consists of two components. The first one is due to clay minerals expansion by thermal effect and the second is because of temperature on physico-chemical interparticle forces.

2.6. Plasticity Index, PI, Liquid Limit, LL, and Initial Void Ratio

Abuel-Naga et al. (2006-b) studied the relationship between the plasticity index and the thermal deformation of clays. They compared their results and others as well with NC clay regarding induced volume change under temperature change of $\Delta T= (65-70)\, ^{\circ}C$ with plasticity index. Figure 2.6 shows a trend between plasticity index and thermally induced volumetric strain. The trend presents that with increasing plasticity index, thermal deformation increases too. Soft Bangkok clay showed the same trend with plasticity index, P.I. = 60 and 6% induced volumetric strain. Di Donna (2014) studied the same relationship with undisturbed samples and her findings agreed with Abuel-Naga’s findings. In addition, Di Donna (2014) studied the effect of the initial void ratio on the thermal deformation, which shows that with increasing initial void ratio, higher thermal deformation (thermal collapse) will be induced. She also found a non-linear proportional relationship between thermo-plastic deformation and plasticity index and initial void ratio. Sultan et al. (2002) found that soils with high Plasticity Index tend to show more volumetric deformation under heating. Tidfors and Sallfors (1989) calculated the normalized slope of reduction of preconsolidation pressure with temperature for several tests and clays and plotted it with liquid limit. The impact of temperature on preconsolidation pressure increases with increasing LL (Tidfors and Sallfors, 1989).
2.7. Anisotropic Condition

According to Laloui and Di Donna (2013), in the case of deviatoric stress (\(d_s\)), it represents an additional axis to the plane to form a 3D space for representing the elastic domain, as shown in Figure 2.7.

---

Figure 2.6. Dependency of thermal induced deformation on plasticity index for various clays (Abuel-Naga et al., 2006-b).

Figure 2.7. Thermal-stress paths in the mean effective stress–deviatoric stress-temperature space (Laloui and Di Donna, 2013).
where \( q = \sqrt[\frac{3}{2}]{\text{tr}(d^2s)} \)

\( \text{tr} \) is the trace of the deviatoric stress tensor \((s)\), in this case, an OC material at a higher temperature will reach the elastic surface at lower deviatoric stress than that at a lower temperature.

Abuel-Naga et al. (2007) investigated the effect of anisotropic consolidation with temperature. They found that Roscoe surface becomes steeper at high temperatures as the ratio between the initial and peak mean effective stresses reduces and the peak deviatoric stress increases. It can be shown that the slope of the compression line \( \lambda \) does not depend on temperature since the lines moved to the left side but with nearly the same slope. This movement indicates a reduction in the elastic zone size with temperature, which can be described by the preconsolidation pressure evolution at a constant plastic strain.

The slope of the swelling line (reloading) \( \kappa \) depends on temperature. In contrast to other researchers who found that the swelling line slope is temperature independent (e.g., Campanella and Mitchell, 1968; Hueckel and Baldi, 1990). Abuel-Naga et al. (2007) found that the critical state line has a slope \( M=0.8 \) in the \( q-p \) plane and it seems to be independent of temperature. However, they did not find a clear trend for the critical state line behavior in the volumetric plane.

### 2.8. Effect of Cycles Under Drained and Fully Saturation Conditions

For energy geostructures, previous experiments showed that for NC clay, most of the irreversible deformation is removable or achieved in the first thermal load cycle. Moreover, subsequent cycles produce a small increment of that irreversible deformation and it decreases with increasing cycles (Laloui and Di Donna, 2013), as shown in Figure 2.8. Furthermore, the same observations were found by Di Donna (2014) after performing consolidation tests on modified oedometers. The
expectation was that after one or more cycles, the shear strength is also affected by the initial OCR (Laloui and Di Donna, 2013).

![Figure 2.8. Thermal cyclic effects on NC clays (Laloui and Di Donna, 2013).](image)

Abuel-Naga et al. (2006-a) applied thermal cyclic load on a NC specimen of soft Bangkok clay with one gap of thermal cycle using an oedometric consolidation test. They found that a much higher load was required to consolidate the sample again (thermal consolidation), an increase in shear strength was also observed. The specimen switched from NC state to OC after heating. In addition, it shows that the induced overconsolidation behavior does not depend on the magnitude of effective stress. Similar findings were observed by Burghignoli et al. (2000). Burghignoli et al. (2000) conducted triaxial tests on undisturbed and remolded clay samples under temperature variations between 20 and 60°C. It was found that with thermal cycles, the initial stiffening of soil for the first cycle was observed, which showed (in addition to the plastic deformation) a reduction with increasing the number of cycles.

Abuel-Naga et al. (2007) observed a higher strength for the clay that had been heated and cooled down at lower temperatures with comparison to a heated clay at a higher temperature. Furthermore, Di Donna (2014) found the same observation regarding the transition of the NC clays to OC clays after a certain amount of plastic deformation takes place in NC clays due to cyclic thermal loading.
The configuration of NC clays becomes more stable and results in a transition of the NC clays to OC clays. Firstly, the sample was mechanically consolidated before applying four thermal cycles (5° to 60°C), then it was mechanically loaded to 200kPa. When the material was subjected to mechanical loading after thermal cyclic loading, it showed an elastic response until it touched the NCL. Then, it followed a plastic behavior. This shows us that before thermal loading the material followed the behavior of NC material. However, after the thermal loading, it followed the behavior of OC material. For OC material, there is no plastic deformation in the heating-cooling cycles and no change in the elastic domain. In addition, no effect on shearing strength because no permeant change in the void ratio occurs. This is in contrast to NC or slightly OC condition, which will have strain hardening as it enters the plastic deformation and the material ends up as OC at T₀ (initial temperature). This process is called thermally induced overconsolidation (Laloui and Di Donna, 2013).

2.9. Concrete-Soil Interface Behavior at Different Temperatures

Di Donna (2014) reported the interface volumetric response of the surrounding soil by the effect of stress history of the surrounding clay material and the degree of compaction for sandy soils. In addition, it was reported that an OC clay and compacted sand shows a dilation behavior upon shearing while a NC clay and loose sand shows a contraction behavior upon shearing. However, this volumetric change is not free to develop at the interface due to the confinement, and just a portion of it will be developed, this condition is called constant normal stiffness (CNS).

2.9.1. Testing Program- Experimental Investigation of the Concrete-Soil Interface

Di Donna (2014) conducted several tests using a modified direct shear device to investigate the thermo-mechanical behavior of the concrete-soil interface shear strength and soil-soil shear strength. The tests were conducted under both drained and fully saturation conditions on a
concrete-sand interface and a concrete-clay interface. It was also performed under different concrete surface roughness, a range of temperature from 20°C to 60°C, and three values of normal stresses of 50, 100, and 150kPa. In addition, clay-concrete interface and sand-sand interface were investigated. The material of the tested sand was selected to be quartz sand, and Illite for the clayey soil. All the interface tests started with a consolidation phase. The drained heating condition was maintained by imposing a thermal loading rate of 2°C/h to allow for the dissipation of pore water pressure. The direct shear device was modified to account for temperature control and for ensuring full contact of the concrete-soil interface. This was done by designing another bottom shear box that is larger in length (from 60mm to 105mm) to ensure the full contact area during the test. A heating tissue was also installed at the bottom of the bottom shear box, under the concrete, and was controlled by a data acquisition system.

Di Donna (2014) performed the modified direct shear tests on clay, sand, sand-concrete interface, and clay-concrete interfaces. Full saturation condition was set on all clay and clay-concrete interface. In contrast, dry condition was set for all sand and sand-concrete interfaces except for two tests. The considered normal stresses are 50, 100, and 150kPa, which represent the stresses acting on soil-pile interface and the investigated temperatures are in the range between 20°C and 60°C. All tests conducted on interface started from a consolidation phase. Then thermal loading was applied under drained conditions before shearing. The drained condition was ensured for the clay by imposing a 2°C/h heating rate. The shearing phase was applied after reaching the target temperature and the stabilization of the associated deformation. Shearing was conducted with different conditions (e.g. CNL {constant normal load} or CNS {constant normal stiffness}).

The used clay in the tests was Illite clay, it was selected due to its sensitivity to temperature. It has 86.6% fine fractions <0.06 mm. It consists of 77% illite, 10% kaolinite, 12% calcite, and a very
small quantity of feldspar, quartz, and traces. It was prepared to have a 1.2 g/cm³ dry density, 1.21 void ratio, and a water content corresponding to the fully saturated condition of 46%. To keep the full saturation condition during a test, the shear box was filled with distilled water, and the compaction took place inside the shear box under the consolidation phase to be a NC clay. With an assumed horizontal displacement at failure of 8 mm, Di Donna (2014) selected 0.006 mm/min to be the shearing rate of clayey soils. This shearing rate is conservative regarding shearing of clay-concrete since the shearing zone is thinner and will be easily drained.

Yazdani et al. (2019-b) performed the tests under three normal stress: 150, 225, and 300kPa, which correspond to the lateral pressure at 22, 33, and 44m depths. The number of cycles they performed was 10, 20, and 40 cycles, each cycle was heated from 24°C to 34°C and cooled back to 24°C (to simulate the temperatures generated in geothermal piles in summertime), and sheared at 24°C. They performed their tests using a conventional direct shear device having a box dimension of (100 × 100 × 40) mm³. They modified it by embedding a 6.4mm copper tube within the concrete in the lower box and connecting the copper tube with a heat pump to circulate the heated water through it. They monitored the temperature by placing two thermocouples with miniature sizes on the top surface of the concrete plate. They performed their tests on remolded kaolin HC-77 (CM) clay. After adding double the liquid limit of water to the clay powder, it was consolidated in incremental loading up to 100kPa. The normal roughness of the concrete surface was ranging from 0.88 and 5.38. The final dimensions of the used concrete plate were (150 × 150 × 25) mm³. The monotonic thermal was from 24°C to 34°C and sheared at 34°C. A heating rate of 3.33°C/h was proposed by other researchers to ensure a drainage condition. In each cycle, the heating rate in the first 10 min was more than 3.33°C/h and was 1°C/h after the 10 min. However, they sheared the sample 210 min after reaching the desired temperature to ensure the drainage of excess pore water.
pressure. The time of 210 minutes was selected based on the duration required for the dissipation of excess pore water pressure under the same thermal loading (10°C heating) from the same clay material having 5×10 cm² dimensions performed in a triaxial device (Yazdani et al., 2019-b).

According to Xiao et al. (2014), the range of temperature induced by the geothermal piles is from 1°C to 31°C. They investigated the effect of heat variation on soil strength and soil-pile interface strength by direct shear tests. They performed the tests at 4°C, 21°C, 30°C, and 38°C for soil tests, and 6°C and 21°C for the soil-pile interface. The clay they investigated is silt from Bonny dam. The classification of the soil according to the Unified Soil Classification System (USCS) is ML (inorganic low plasticity silt). After shearing moisture content ranged from 12.6% to 14.6% while their target moisture content and the optimum moisture content (associated with standard Proctor test) are 13% and 13.6%, respectively. For achieving a drainage condition, their rate of shearing was 0.2 mm/min. After the end of consolidation, heating stage was started, and shearing was right after the target temperature was reached and maintained constant during shearing.

Yavari et al. (2016) studied the effect of heat variation on the shear strength of clay, sand and that of clay-concrete interface performed in direct shear tests at 5°C, 20°C, and 40°C and normal stresses from 5 to 80 kPa. They found the effect of temperature on the shear strength of soils and clay-concrete interface to be negligible. However, the clay-concrete interface showed a softening behavior under shearing in contrast to that of clay tests.

They performed the tests on kaolin clay, Fontainebleau sand, and kaolin clay for the clay-concrete interface. In their test, the shear box container was filled with water, and a copper tube circulating water from a water bath was installed around the shear box. Thermocouples were placed below the shear box, at the water surface, and two were placed inside the soil specimen. The Fontainebleau sand properties are: maximum void ratio $e_{\text{max}} = 0.94$, minimum void ratio $e_{\text{min}} =$
0.54, mean diameter $D_{50} = 0.23$ mm, and particle density $\rho_s = 2.67$ Mg/m$^3$. The sand was compacted to a 1.5 Mg/m$^3$ density, which corresponds to 46% of relative density. The Kaolin clay properties are particle density $\rho_s = 2.60$ Mg/m$^3$, liquid limit $LL=57\%$, and plastic limit $PL=33\%$. After adding 1.5 LL distilled water to the clay, it was consolidated under 100kPa vertical stress. The void ratio after consolidation is $e_f=1.35$. The maximum roughness of the concrete was in the order of 0.7 mm, and this concrete sample was used in all tests. For all tests, 100kPa (preconsolidation pressure) vertical stress was applied in 20kPa steps (each net loading step was applied after the stabilization of vertical displacement) at 20°C, and the temperature was raised from 20°C to 40°C in 5°C steps (each step was held for 15 min, which corresponds to the time required for stabilization of vertical displacement) while keeping the 100kPa vertical stress. It took 3 hours to change the temperature from 20°C to 40°C by a 7°C/h average rate, and 10°C/h actual rate of raising temperature (without the 15 min). For the dissipation of pore water pressure, the soil was kept under the desired temperature for 2 hours. The selected rate of shearing was $14\mu$m/min to avoid any development of pore water pressure under shearing following Bhat et al. (2013) work. Furthermore, the rate of shearing for the sand was 0.2mm/min.

2.9.2. Experimental Results

Regarding the effect of temperature on sand-concrete interface in the performed tests under 100 and 150kPa, Di Donna (2014) found no difference in the response at 20°C and 60°C. However, volume response under shearing under 50kPa normal stress showed different behavior between 20°C and 60°C, but it was related it to a difference in fabric.

Regarding the effect of temperature on clay-concrete interface, Di Donna (2014) performed three tests at 20°C under CNL and at 50°C of NC clay on a high concrete roughness. There was observed to be an increase in shear strength of interface and a decrease in volumetric contraction under
shearing. This is because at a higher temperature the clay contracted under thermal loading, which is densification. In addition, between the soil and the concrete asperities, the contact area increased. The shear strength at failure (large displacement) was also increased with a much lower percentage, except for the case conducted under 150kPa normal stress, which showed no increase. She also expected no effect of heating on the shear strength if the failure took place within the soil. Through the Mohr plane of these three tests, it can be shown that the friction angle at the interface was decreased from 25° at 20°C to an angle of 23° at 50°C, but with an increase of cohesion from 7kPa (at 20°C) to 20kPa (at 50°C).

Under 150kPa normal stress, peak shear stress under heat cycles was increased 5%, but strain at the peak stress was slightly decreased, and no effect was observed under monotonic heating regarding the peak shear strength (Yazdani et al., 2019-b). For the shear strength at ultimate, 6% increase was observed under heat cycles and 3.5% decrease under monotonic heating. In addition, the contraction behavior under shearing was decreased, which is believed to be due to the stiffening of the NC (thermal strengthening) clay under heating. Under cyclic and monotonic heating and for both normal stresses, the peak and at failure shear strength increased, but the increase was higher for cyclic heating. The same observation was more clearly found under these normal stresses with respect to the decrease in contraction during shearing.

They also studied the effect of OCRs, by performing tests under OCR = 2 and 5 under 150kPa normal stress. A reduction at peak and at failure shear strengths was found under monotonic and cyclic heating with more of a reduction under the cyclic heating. In addition, thermal softening was observed at large displacements. They also found an increase in interface friction angle by 27% and 25% for cyclic and monotonic heating, respectively, and a 17-20% reduction in adhesion with temperature from 24°C to 34°C.
Xiao et al. (2014) found that the ultimate shear strength of the silt at 38°C was 15.21% higher than that at 4°C. When comparing the temperature cycles effect with the monotonic temperature, it was observed that the ultimate shear strength after four cycles between 26°C and 38°C was higher than that at 38°C. The angle of friction at 21°C is higher than that at 6°C, but with less cohesion than that at 6°C. The ultimate shear strength at the soil-concrete interface at 21°C was found to be 5% to 16% higher than that at 6°C. However, the soil water content at 6°C all different normal stress was found to be around 0.6% larger than that at 21°C. Yavari et al. (2016) found a negligible effect of temperature on soil shear strength. Figure 2.9 shows a summary of other results as well as the results found by Yavari et al. (2016).

![Figure 2.9. Effect of temperature on friction angle. (Yavari et al., 2016)](image)

### 2.9.3. Scale-Model Pile Testing

McCarty and Rosenberg (2011) performed a study to investigate the heat exchange effect on the side shear of thermo-active foundations. Before applying mechanical load, they heated a scale-model pile in a geotechnical centrifuge to a different temperature. For evaluating the side shear variations due to heating, load transfer analysis (T-z) was used. They observed an increase in side shear for piles loaded to failure after heating. This increase was observed to be proportional to the
change in temperature. They simulated a 381mm length and 76.2mm diameter precast concrete pile with a 9.1m length and 1.8m diameter pile. They performed their test on a Bonny silt soil with P.I.=4 and 84% fines, and it was compacted with 13.2% water content (optimum water content). After the test, the water content in the first 15 cm was reduced by 3% and beyond that, it has the same water content. The temperature was raised from 15°C to 50°C and 60°C. The test was done in three phases; the first phase consists of stabilization of pile due to centrifuge load. Phase 2 consists of heating the pile to the desired temperature until it reaches a steady state temperature. Phase 3 consists of loading the pile under a constant rate of displacement of 0.2mm/min.

According to McCartney and Rosenberg (2011), due to the lateral expansion of pile under heating, the soil will compress and an increase in interface shear stress will be observed in a drained condition, as shown in Figure 2.10. Figure 2.10 shows the load settlement curves of the four tested piles with different temperatures. As shown, there is a clear trend that with increasing temperature, settlement decreases. However, a plunging failure was observed for higher temperatures (i.e. more brittle behavior) because of higher induced lateral stresses due to the pile lateral expansion as a result of heating.

Figure 2.10. Load-settlement curves for scale-model foundations in prototype scale (McCartney and Rosenberg, 2011).
2.9.4. Pull-out Test

Elzeiny et al. (2018) conducted two pull out tests in poorly graded dry sand with SP classification according to USCS, one on an energy concrete pile after five heating cycles by raising the temperature 20°C from the room temperature and another test on a concrete pile at ambient temperature. The pile has a 101.6mm diameter and 1.383m length and was loaded under load control in both tests. Elzeiny et al. (2018) are among the very few researchers who implemented heating cycles in their tests on piles. A 28% increase in shaft resistance was observed for the energy pile at failure after thermal cycles when compared to the concrete pile tested at ambient temperature. However, axial load capacities were not the same for both tests in a way that the energy pile was loaded in three stages, from 0 N to 450 N, from 450 N to 1640 N, and from 1640 N up to failure by increments of 67 N, 111 N, and 222 N, respectively. The concrete pile at ambient temperature was loaded in two stages, from 0 N to 530 N, and from 530 N up to failure by increments of 89 N and 222 N, respectively.

2.10. Drained/Undrained Tests Under Drained/Undrained Heating

Abuel-Naga et al. (2006-b) studied the induced volume changes under drained heating, while under undrained heating, they studied the induced excess PWP. Abuel-Naga et al. (2007) also conducted compression triaxial tests with a drained heating approach on Bangkok clay soil with modifications to account for the range of temperature from 25°C to 90°C. The soft Bangkok clay consists of 54% to 71% smectite (illite and montromonolite), 28% to 36% kaolinite, and mica with plasticity index of 60. In addition, they reported two heating approaches to reserve the drained heating condition, first method by Del Olmo et al. (1996), which is mainly about applying a very low thermal loading rate and monitoring the induced pore water pressure by a computer to keep the pore water pressure equal to zero. The second method by Towhata et al. (1993), and Delage et al. (2000), which is
applying thermal load (increasing temperature) incrementally after the stabilization of volume change of the previous thermal load. In addition, Abuel-Naga et al. (2007, 2006-a, 2006-b) adopted the second approach for ensuring the dissipation of PWP.

2.10.1. Oedometer Experimental Program

For the oedometer, three sets of tests with four specimens in each set were tested by Abuel-Naga et al. (2006-a) to investigate the induced volume change under thermal cyclic loading of 22° - 90° - 22°C. Different normal stress were used in each set (100, 200, and 300kPa), which is higher than the preconsolidation pressure of 70kPa. Three of the specimens were unloaded to different OCRs of 2, 4, and 8. After that, all specimens in all sets were subjected to thermally cyclic loading from 22°C-90°C-22°C and vertical deformation was recorded under heating and cooling.

Three sets with three specimens in each set were used for studying the induced overconsolidation behavior of NC clays as a result of heating under different stress levels and thermal cycles. In each set, three specimens were consolidated under (100, 200, and 300) kPa normal stresses. Then they were subjected to different thermal cyclic loading such as (25° - 50° - 25°)C, (25° - 70° - 25°)C, and (25° - 90° - 25°)C. For investigating the induced preconsolidation pressure change, all specimens were consolidated after thermal cyclic loading. They also observed that the induced volume change is independent of the applied stress, where that induced change in void ratio is nearly the same under 100, 200, and 300kPa stresses. However, what matters are the OCRs.

2.10.2. Triaxial Experimental Program

The modified triaxial apparatus was used by Abuel-Naga et al. (2006-a) to investigate two main things. First is the behavior of shear strength. This was done by performing undrained and drained shearing in a triaxial test (shearing rate was selected to be 2% strain/hour and 0.1% strain/hour for undrained and drained conditions, respectively) after drained heating at 70 and 90°C on normally
consolidated samples at 300kPa. For investigating the thermal cyclic response on undrained shear behavior, samples that are normally consolidated at 200kPa were sheared after being under drained thermal cycles of 25-70-25°C and 25-90-25°C.

Second is the temperature effect on NCL (Normal Consolidate Line). This was done by achieving an OCR = 12 by isotropic consolidating the sample under 300kPa effective stress after saturation and unload it to 25kPa effective stress. Then, consolidating the samples under three different temperatures of 25, 70, and 90°C. After reaching the desired temperatures, the specimens were reconsolidated (while keeping the target temperature constant) by 25kPa steps until reaching a 600kPa effective stress. The NCL is moving to the left with increasing temperatures with an almost constant slope.

The results of undrained shearing at different thermal cyclic loading and different temperatures show an increase in the peak of normalized deviatoric stress with increasing temperature and thermal cycle even though that shearing was at a temperature of 25°C after thermal cycles. However, for specimens sheared after thermal cycles, the peak was at smaller strains. A decrease in EPWP was also observed with increasing temperature and thermal cycles. Which makes sense for a NC clay, which means that if NC clay was subjected to an increase in temperature, a permanent increase will be observed of the undrained peak deviatoric stress (Abuel-Naga et al., 2006-a).

The results of shearing under drained condition on NC at 300kPa shows that at higher temperatures, higher peak shear strengths with more strain softening and less volumetric strain were observed with a strain hardening for those at room temperature. However, the residual stresses were found to be independent of temperature.
They found that in the q-p plane, the slope M of shear strength envelope at a critical state is independent of temperature. However, an increase in normalized undrained and drained secant modulus with temperature and temperature history was observed. Those with temperature history showed higher secant modulus than those at higher temperatures, the same was observed with a drained condition when compared to the undrained condition.

Their shear strength results under undrained condition of isotropic NC specimens under 25°C, 70°C, and 90°C temperatures and a 300kPa preconsolidated pressure shows the results of shear strength under undrained condition of OC specimens with the same preconsolidation pressure and 1.5, 3, and 9 OCRs under temperatures of 25°C and 70°C. An increase in undrained shear strength with temperature can be shown for all OCRs. In addition, the PWP decreased with increasing temperatures for NC and slightly OC specimens while it shows an increase with increasing temperature for highly OC specimens. (Abuel-Naga et al., 2007) To study the effect of temperature history (cycles), specimens with 1, 2, and 4 OCRs were subjected to temperature cycles (25° - 70° - 25°)C and (25° – 90° - 25°)C before shearing. The undrained shear strength increased with increasing temperature cycles while the PWP decreased for all OCRs (Abuel-Naga et al., 2007).

For studying the induced excess PWP under undrained heating, Abuel-Naga et al. (2006-b) tested 5 specimens, three of which, were NC at 200, 300, and 400kPa for studying the stress level effect while two other specimens were overconsolidated at OCRs= 2 and 4 with 200kPa preconsolidation pressure for studying the effect of stress history on induced excess PWP. Increasing the temperatures to 90°C was done in steps of 10°C under undrained condition and the developed PWP was measured for each step. As it is shown, the excess PWP depends on the OCR and it decreases with increasing OCR, and that the induced excess PWP is irreversible for the OC clays,
which reaches negative values after cooling while reversible for NC. They also referred to the reason behind the induced excess PWP (the difference between thermal expansions).

According to Abuel-Naga et al. (2006-b), another factor on the induced excess PWP is the mean effective stress $p_{\text{cons}}$, which restrict the expansion of soil skeleton. The excess PWP increases when increasing the mean effective stress. For NC samples, the heating and cooling paths are coincident. In contrast to the cooling path for OC samples, which indicates that cooling paths depend on stress history.

The explanation of the induced irreversible excess PWP of OC samples can be achieved by the aid of unloading reloading hysteresis concept in a curve similar to that of soil consolidation. Furthermore, the unloading modulus increases with decreasing the stress. Moreover, for OC samples and undrained heating, increasing the temperature will increase the PWP, which will decrease the effective stress and the path that the sample will follow with increasing void ratio. When the sample is cooled, the PWP will decrease, which will increase the effective stress. The difference between the unloading and reloading modules will produce plastic expansion volume change, which will produce a decrease in PWP under cooling. For NC samples, there is no significant difference in the unloading and reloading modules, which describe the reversible behavior (Abuel-Naga et al., 2006-b).

Another valid explanation for the volumetric and PWP behavior under undrained heating is based on the effect of temperature on physico-chemical interparticle forces. PWP and physico-chemical interparticle forces will increase with increasing temperature under undrained heating. Then, viscous shear interparticle resistance of adsorbed water and effective stress will decrease. As a result, induced volume expansion that is reversible and excess PWP will be generated for the NC samples. However, for the OC samples, unstable stress conditions will be created by the induced
excess PWP, which will result in irreversible fabric disintegration. This is why with increasing OCRs, irreversible induced PWP and volume change will be observed because of induced fabric change by thermal effect under low eternal stress condition (Abuel-Naga et al., 2006-a)

Burghignoli et al. (2000) conducted compression triaxial tests (both drained and undrained) with modifications to account for temperature variations from 20° to 60°C on undisturbed and remolded clay samples. The investigation of thermo-mechanical behavior of clayey soils was done by investigating the thermal load and mechanical load separately. No much temperature dependency of soil strength was found (this was investigated by plotting the effective stress path in the p’-q space, the effective stress paths were not identical, but they were touching the critical state line at almost the same point. However, stiffening of clay samples after thermal cycles was observed. Similar findings were reported by Houston et al. (1985). In drained condition, volume change observations confirmed the previously mentioned framework. They suggest that these observations with thermal load is due to soil skeleton viscosity by the rearrangement of particles and grain volume change with temperature. Nevertheless, they found that volume change (or void ratio) depends on many other factors besides the stress history, these other factors are time after the primary consolidation (due to the final mechanical load) ends and starting the thermal load (the change of void ratio, either positive or negative change, decreases as the time increases), recent stress history, thermal history (an overheated material showed higher initial stiffness compared to a normally heated material even if the normally heated material was subjected to a higher temperature than the overheated material), and time of heating (the change of void ratio, either positive or negative change, decreases as the time increases), and time of constant temperature phases. Hueckel and Pellegrini (1989), Miliziano (1992), and Lingnau et al. (1995) also found that for temperature less than 60°C, the shear strength dependence of temperature is insignificant.
According to Lalouï (2001), some researchers included that temp effect slightly increased soil strength with increasing temperature while others concluded that increasing temperature decreases strength. It seems that soil strength-temp relationship is related to the nature of considered soil.

The modified density (critical pressure) produced by thermal hardening, produces a different temp at various initial states for the soil. Therefore, shear strength will depend on temperature by the effect of the different critical pressures and friction angle. When the undrained (water undrained) condition exists, it will increase pore water pressure.

Samarakoon et al. (2018) observed an increase in undrained shear strength with increasing temperature under undrained heating condition for NC clays, and higher undrained shear strength was observed with cycles. They also found that the relationship between the undrained shear strength and the initial mean effective stress to be inversely proportional.

Takai et al. (2016) investigated the thermal volume change behavior of drained and undrained NC kaolinite clay using a modified triaxial device. The induced temperature was in the range of 23°C to 60°C. For the undrained condition, the clay showed an elastic expansion and the measured pore water pressures agreed with what was predicted based on the mechanistic model. An almost linear relationship was observed between both the volume change and temperature change, and pore water pressure and the temperature change in undrained heating. For the drained condition, the clay showed nonlinear irreversible contraction, which is underestimated with the predicted volume change of drained condition using the measured pore water pressures in the undrained condition, they suggested the reasons to be either from thermal creep following thermal consolidation or due to parametric errors like the measured compression line slope and thermal expansion coefficient.
2.11. Geothermal Energy Application

Geothermal energy is the energy stored in the earth because of radioactive decay of materials, mainly potassium, uranium and thorium, which can be extracted as clean renewable energy. (Lund, 2005). As a heat source, geothermal energy is classified as the second on earth in terms of abundance. The first experiment for electricity production was done by Prince Ignace Conti in Italy in 1904-1905 and the first power plant was made in 1913 at Larderello, Italy (Bjelm, 2008).

For generating electrical power resources with temperatures above 150°C are used in most practical cases, for direct usage resources with temperatures below 150°C are most frequently used. For heating and cooling via Ground Source Heat Pump (GSHP) temperatures ranging between 5°C and 30°C can be used (Bjelm, 2009).

Deep geothermal energy is utilized by building geothermal power plants that use steam from heated water deep underground to drive turbines and generate electricity. These plants are built near tectonic plate boundaries because the high temperature could be reached much closer to the surface (greater than approximately 400m in depth) (Bjelm, 2009). Shallow geothermal energy usually falls in the first 100m (Sanner, 2001). The temperature above the ground affects a very shallow depth, usually a couple of meters (Ghasemi-Fare, 2015), which depends on the climate, the region and the type of soil. Shallow geothermal energy is usually utilized by a Ground Source Heat Pumps (GSHP) to provide heating and cooling to buildings by both space or district heating systems with closed or open loops, usually less than 100m.

Closed and open loop heating systems are shown in Figure 2.11-a and Figure 2.11-b respectively. Figure 2.11-a shows the closed-loop system in the vertical and horizontal directions, in this system the loops are closed and connected to the building by a GSHP. A refrigerant is circulated inside the loops by the GSHP to extract heat from the ground in the winter and to inject heat in summer.
Figure 2.11-b shows an open-loop system in two separated wells (on left) and in the same source of water (on right). A GSHP is also used, but the water is pumped from one well to the building and then either pumped to another separated well or to the same water source (Bjelm, 2009).

2.11.1 Heating and Cooling System Methodology via Ground Source Heat Pump

To understand how GSHP works, the principle of vapor compression refrigeration must be understood. The vapor-compression uses a circulating liquid refrigerant as the medium, which absorbs and removes heat from the space to be cooled and subsequently rejects that heat elsewhere. Any system employing this principle (GSHP or air source heat pump, i.e. ASHP, or any other systems) must have four components, which are a compressor, condenser, thermal expansion valve, and an evaporator.

In cooling mode, the circulating refrigerant enters the compressor in the thermodynamic state known as a saturated vapor and is compressed to a higher pressure, resulting in a higher temperature as well what is known as a superheated vapor state. Superheated vapor can be
condensed into a saturated liquid with either cooling water or cooling air flowing across the coil or tubes in the condenser where the circulating refrigerant rejects heat from the system. The saturated liquid is next routed through an expansion valve where it undergoes an abrupt reduction in pressure. That pressure reduction results in the evaporation of a part of the liquid refrigerant and lowers the temperature of the liquid and vapor refrigerant mixture to where it is colder than the temperature of the enclosed space to be refrigerated. The cold mixture is then routed through the coil or tubes in the evaporator. A fan circulates the warm air in the enclosed space across the coil or tubes carrying the cold refrigerant liquid and vapor mixture. That warm air evaporates the liquid part of the cold refrigerant mixture. At the same time, the circulating air is cooled and thus lowers the temperature of the enclosed space to the desired temperature. This saturated vapor goes back to the compressor to complete the cycle again. This process is depicted in Figure 2.12, where, in heating mode, the process is reversed in such a way that the place to be heated will play the role of the condenser and the heat source will play the role of the evaporator. Akrouch (2014) provided a very good illustration of GSHP in cooling mode with examples of temperatures.

Figure 2.12. Process of Vapor Compression Refrigeration (Akrouch, 2014).
2.11.2. Thermo-Mechanical Behavior of Soils and Energy Piles

When the vertical pile is subjected to thermal load, it experiences additional thermal strains, referred to as $\varepsilon_{T-Observed}$, which is the measured strain resulting from the thermal load, around a neutral point (NP). When the pile is heated, it experiences expansion and it moves upward above the NP and downward below it, while the opposite is correct when the pile is cooled. Another part of the vertical strain is restrained due to soil resistance $\varepsilon_{T-Restrained}$. The sum of $\varepsilon_{T-Observed}$ and $\varepsilon_{T-Restrained}$ is the free strain $\varepsilon_{T-free}$, which is the strain that the pile would experience if it was not inhibited by the soil and the structure. The thermal stresses ($\sigma_T$) resulting from the difference between the free and observed strain and the thermally induced load, $PT$, can be calculated using the below equation. In this equation, the negative sign means that the restrained thermal strains result in a force in the opposite direction of the pile movement (Akrouch, 2014).

$$PT = -EA\varepsilon_{T-Restrained} = -EA(\varepsilon_{T-free} - \varepsilon_{T-observed}) = -EA(\alpha\Delta T - \varepsilon_{T-observed}) = \sigma_TA$$

In energy piles, the total load in the pile is the sum of the mechanical and thermal load. Abdelaziz (2013) showed an illustration for cooling and heating modes.

2.11.3 Thermo-Mechanical Tests

Raul (2016) published a paper presenting a finite difference solution to the fully coupled formulation to study the development of excess pore water pressures in geothermal piles and its impact on the shaft friction at the pile-soil interface at the middle of the pile. The study was made on the pile with varying the hydraulic conductivity, soil compressibility and temperature of fluid such that, $k= 1\times10^{-8}$, $1\times10^{-9}$, $1\times10^{-10}$, $1\times10^{-11}$, $1\times10^{-12}$ m/s, $ks=2\times10^6$, $2\times10^7$, $2\times10^8$, $2\times10^9$, $2\times10^{10}$ Pa and $T_f= 20°C$, $30°C$, $40°C$, $50°C$. It was shown that excess pore water pressure around the pile increasing with increasing hydraulic conductivity, soil compressibility, and fluid temperature. In return, this will decrease the effective stress and hence decrease the shaft friction.
at the interface. The increase with $K$ is much lower than with $K_s$. However, this test was exaggerating the values of $K_s$ and assumed the soil profile as homogenous soil material, which is far from reality.

Kramer (2014) performed a thermo-mechanical test on a 100-mm-diameter precast concrete pile with an embedment depth of 1.22m was used in the experiments described in this paper. One U-shaped PVC circulation tube with an inner diameter of 12.4mm was embedded in the concrete pile to allow circulation of the heat carrier fluid (1:1 mixture of ethylene glycol and distilled water) during thermal loading.

Each branch of the U-tube was at a distance 23mm away from the center. The pile was put inside a 1.83 × 1.83m tank composed of a 1.22-m high lower half and the upper half of height 0.91m with distributed thermocouples in the soil. Kramer (2014) applied a temperature gradient of 21°C on the pile. Radial heat transfer from the pile to the surrounding soil was observed for most of the pile length, except near the pile head due to the presence of a convective boundary at the soil surface.

Mechanical load tests were performed on the model pile before and after the heat exchange operation. Pile head and base displacements were measured continuously during the load tests. A load increment of 0.1kN was used for all tests. Akrouch et al. (2014) performed an in-situ thermo-mechanical test also on pile in high plasticity clay, the pile was 0.18m in diameter and 5.5m long and was reinforced with a 25-mm-diameter steel bar and single U-shape pipe. Akrouch et al. (2014) applied five tension loads on the pile with a tension force $T$ of 40, 100, 150, 200, and 256kN. In each test, the pile was mechanically loaded for 1 h (60 min). After 1 h of applying the load, the water pump was turned on to circulate the water into the pile. The temperature gradient added by the water refrigerant was 10-15°C in magnitude. The water pump was run for 4 hours after finishing the mechanical loading step.
CHAPTER 3.
LOW P.I. SOIL UNDER ONE HEATING-LOADING CYCLE AT 70° C

This chapter starts by describing the small-size and large-size direct shear devices and the modification that were made on the large-size direct shear device to allow for the heating condition. In addition, the preparation process and procedures are discussed. The characteristics of the low P.I. clay soil are presented in detail along with the shear strength properties obtained from the small-size direct shear device. In addition, heating methodology, calibration, consolidation, and thermal loading were presented. The interface shear strength results obtained from the large-size direct shear device for the tests that were subjected to a heating-cooling cycle at 70°C temperature and the tests that were tested without heating along with the thermally induced volumetric strains were analyzed and discussed in detail.

3.1. Direct Shear Device

The direct shear device provides reliable test results, but less reliable than a full scale testing of a pile, though with a lower price and completed in less time. There are two main types of shear tests used. The simple direct shear test, which simulates plane strain loading condition, and the direct shear test, which is used to determine the shear strength parameters of soils on a predetermined horizontal surface failure. The direct shear device could be used to tests fine- and coarse-grained material. The direct shear apparatus consists mainly with a two halves of direct shear box, the vertical (for applying the confining vertical load) and the horizontal (for applying the horizontal shearing load) load cells along with a vertical and a horizontal Linear variable differential transformer (LVDT) for measuring the vertical and horizontal displacements, respectively. Due to its predetermined horizontal failure surface, it could also be used to test the interface between steel, concrete, or wood and the soil to simulate different types of pile material and the adjacent soil.
This is achieved by installing a concrete block in the lower half apparatus and the soil (either fine- or coarse-grained soil) in the upper half.

Mainly, there are two sizes of the direct shear devices, the large-size direct shear device (LDSD) and the small-size direct shear device (SDSD). Usually the size of shear box of the LDSD is 12” x 12” x 8”, and the size of shear box of the SDSD is usually 4” x 4” x 2”. In literature, almost all the tests that were performed to assess the effect of heating were done using the SDSD. However, it is believed that the LDSD would reflect better and more reliable results as compared with the SDSD due to the larger contact area between the soil and the concrete (i.e. the contact area of the LDSD is 9 times larger than that of the SDSD). In addition, the thickness of the clay layer could be up to 4” in the LDSD instead of just 1” in the case of SDSD. Having a higher clay thickness is better to allow for a full shearing zone.

Both the small- and large-size direct shear devices (SDSD and LDSD) that are available at the Louisiana Transportation Research Center (LTRC), LA, were used in this research study. However, the SDSD was only used for testing the shear strength of clay (no interface) without heating to accelerate the testing timeframe.

3.2. Modified Direct Shear Test Device

Figure 3.1 and Figure 3.2 show the large-size direct shear device at LTRC that was used in this research study. The large-size direct shear device was never used in research for studying the effect of heating on shear strength of clay or clay-concrete surface. In this study, the LDSD was selected to investigate the effect of heating-cooling cycle(s) on the shear strength parameters of clay-concrete interface as it is believed to reflect a better and more clear results than of the small-size direct shear device. This device can endure temperatures up to 80 °C. Therefore, the highest temperature was applied in this study was selected to be 70 °C.
The device consists of two boxes inside a water bath, the upper box is stationary; while the lower box is moving. The size of the upper box is 4’’ x 12’’ x 12’’, which is shown in Figure 3.4, and the size of lower box is 4’’ x 16’’ x 12’’, which is shown in Figure 3.3, and the water bath is shown in Figure 3.5. 4’’ more in length is available in the lower shear box than the upper shear box to provide a constant shearing area under shearing.

The material of the shear boxes is aluminum. One side of the upper box was replaced by another side, of which three grooves were added to provide a space for the wires of the thermocouples.
The grooves were made at the two ends and one at the center as shown in Figure 3.6. The grooves start at 0.25’’ from the bottom, and continue up to the top. A concrete box with less size than the size the lower box by 2mm from all sides was placed inside the lower box as shown in Figure 3.7. The clayey soil was placed in the upper box up to 2’’ thickness as shown in Figure 3.8.

![Image](image1.jpg)  ![Image](image2.jpg)

Figure 3.6. The replaced right side of the upper shear box to accommodate the thermocouples.  
Figure 3.7. The concrete block inside the lower shear box.

![Image](image3.jpg)

Figure 3.8. The clay placed inside the upper shear box after compaction and leveling.

Two Watlow Cartridge Heaters were submerged in the water bath to heat the water as shown in Figure 3.9. A pump that can function up to 95°C temperature was used to circulate the water from the water bath to the top of the cap covering the shear box and going downward to the water bath.
as shown in Figure 3.10. This circulation of water is necessary to ensure uniform temperature for the whole system, to accelerate the heating process, and to create a continuous source of water from the top of the clay for saturation.

Figure 3.9. Top view shows the two rod heaters at the corners.  
Figure 3.10. Heating setup with the water circulation by water pump. 

Each heater was connected to a controller in order to control the temperatures. Figure 3.11 shows the controller while working. Four thermocouples were used, one inside the water bath and three thermocouples were inserted inside the clay specimen at a 0.25” above the concrete surface, at the center, at the corner, and at the middle of the side. These four thermocouples were connected to a thermometer thermocouple (data logger) and the temperatures were recorded in a one-minute step both during heating and cooling as shown in Figure 3.12. Figure 3.13 shows the LDSD under testing during heating.
Figure 3.11. Heating controller.

Figure 3.12. Thermometer thermocouple (data logger).

Figure 3.13. The full setup of large direct shear tests under heating used in this study.
3.3. Material Properties and Sample Preparations

3.3.1. Concrete

In an attempt to mimic the pile surface, a concrete box with the same roughness was token from a contracting company in Baton Rouge, LA. The size of the concrete box is 16” x 16” x 2” as shown in Figure 3.14. One of the sides was reduced to be 12”, and the thickness was increased to be 4” to fit inside the lower shear box. Increasing the thickness was achieved by pouring concrete over the bottom surface of the concrete block, and adhesion (epoxy) material and screw studs were used to fix the old concrete block to the new poured concrete, in addition to roughing the old concrete block as shown Figure 3.15.

![Figure 3.14. 16’’x16’’x2’’ Concrete block.](image1)

![Figure 3.15. Prior to pouring 2’’ concrete layer under the old cut concrete block.](image2)

The gaps between the concrete block and the lower shear box were filled by sand, since sand is incompressible material.
3.3.2. Porous Stone

A porous stone was needed to be installed at the top of the clay layer to allow for escaping the excess pore water pressure. In order to do so, a porous stone with the size of 12’’ x 12’’ x 1’’ was created, as shown in Figure 3.16. The porous stone was created by mixing, by weight, 95% of coarse sand that is passing sieve # 10 and retained on sieve # 30 and 5% of epoxy (Ghaawd et al., 2018). Stainless steel reinforcement was placed at the middle of the porous stone to improve its ability for resisting any tension stresses during sample preparation, testing, or storing. Four T-nut female thread were placed at the corners of the porous stone to help in lifting the porous stone.

![Figure 3.16. Porous stone after hardening.](image)

3.3.3. Clay Characteristics

The geotechnical properties of the lean clay used in this part of the project was a low P.I. soil with a liquid limit = 33, plastic limit = 21, plasticity index = 12. Furthermore, 83% of the material passing sieve # 200, and 100% passing sieve # 4. The soil is classified as low plasticity clay CL according to USCS soil classification, and as silty clay loam as A-6 according to AASHTO soil classification. It has a specific gravity of 2.65, and it consists of 55% silt, 16% sand, and 29% clay
and the preconsolidation pressure is 3.47 psi. The optimum moisture content, according to the standard compaction test, is 17.3% and a maximum wet density of 1.993 g/cm$^3$.

The direct shear tests of the clay-clay interface were performed in a small-size direct shear device (SDSD). The size of the sample was 4” x 4’’ x 1” (16in$^3$). Under shearing, there is a reduction in length and/or area of the sheared interface. Therefore, the shear stresses shown in Figure 3.17 are corrected. Figure 3.17 shows the corrected shear stress response with horizontal displacement under shearing for the low P.I. soil under 4.35 psi, 10 psi, 16 psi, and 21.8 psi normal stresses. Figure 3.18 shows the vertical displacement response under shearing of the same clay. Following the Mohr column criteria, the shear strength parameters of the high plasticity clay are; the internal friction angle ($\phi$) at the peak is 31.1, cohesion at the peak is 1.21 psi, as shown in Figure 3.19.

![Figure 3.17. Shear stress with horizontal displacement without heating for low P.I. clay.](image1)

![Figure 3.18. Vertical displacement behavior under shearing without heating for low P.I. clay.](image2)
3.4. Clay-Concrete Tests

Eight tests were conducted using this soil. The soil was tested under four different normal stresses for both; with heating and without heating conditions. The four selected different normal stresses (4.35psi, 10psi, 16psi, and 21.8psi) were selected for shear testing.

These stresses were selected to depict the confining stress of four shallow depths on a pile. The normal stress was increased from the sitting load of 0.5psi to the targeted normal stress. The clay specimens were left under consolidation until the primary consolidation was completed. The consolidation stage took around one day. For tests that are not subjected to thermal loading, shearing starts directly after the end of consolidation.

3.5. Sample Preparation

The concrete block was first placed and fixed at the bottom section of the shear box. The dried clay was mixed with 26% (79% of the liquid limit) of the water and left for one day to cure and then placed on top of the shear box section. This process was done outside the large-size direct
shear device. The first layer of the soil was compacted to 1.0 in final thickness, 70 blows were applied to each soil layer using the modified proctor hammer, by targeting the standard compaction effort of 12400 ft-lbf/ft³. As previously mentioned, four Thermocouples were placed within the clay specimen, with one at the center, one at the corner, and two at the far-middle side of the clay sample. However, only three thermocouples were used as denoted in Figure 3.20 by T1, T2, and T3, T4 (the fourth thermocouple) was inserted inside the water in the water bath. The second soil layer was added and compacted the same as the first layer. Figure 3.20 shows four steps pf preparation of lean clay of the first layer. The soil was put over the concrete block with the thermocouples in step 1. In step 2, the thermocouples were installed to 0.25 in above the concrete block. Step 3 shows the first layer after compaction. Step 4 shows that the material was roughened before placing the second layer of clay. At the end of the compaction, the wet density was 1.86 g/cm³, moisture content was 26%, and dry density was 1.49 g/cm³. The initial degree of saturation (S) was 96.5%, and the void ratio (e) was 0.89. After preparing the clay specimen, a filter paper was saturated and placed on top of the clay surface. Then, the porous stone was placed over the filter. The shear box was placed inside the water bath of the large-size direct shear device and left submerged in the circulated water for 24 hours under normal stress of 0.5psi to ensure saturation condition is met and to prevent swelling.
3.6. Heating Methodology

Thermal loading was applied immediately after consolidation for the tests that are subjected to a heating-cooling cycle. The system was covered by aluminum foil to reduce the amount of evaporated water as much as possible. The temperature of the system was monitored during heating via the four thermocouples. The duration of phase 2 was controlled by three conditions that need to be satisfied before starting phase 3. These three conditions are:
1- The temperature of the system must reach a homogenous or steady-state (i.e., all four thermocouples (three inside the clay specimen and one in the water bath) reached the target temperature ±2 °C, and no further change occur).

2- No further soil volumetric strain was observed (i.e., ensuring a full dissipation of the induced excess pore water pressure).

3- Heating will stop after two days of heating (around 3000 minutes) if condition 2 is not met. When the above conditions were met, the heaters were shut down, and the temperatures dropped to the room temperature again.

3.7. Shearing

The last step of this test is shearing. This was done by removing the bolts that were connecting both upper and lower shear boxes and creating a gap of around 0.06in. A constant shearing rate condition was targeted during shearing. Since the drained shearing condition is targeted, a very low shearing rate was selected to ensure a drained condition. The lowest shear rate recorded in literature was adopted, which is 0.005 mm/min=0.0002”/min. The horizontal deformation at failure was assumed to be 0.6” or 1.5 cm. It took two days to end shearing, the circulation of water was kept active.

3.8. Calibration

The volume change data during both heating and cooling needed calibration due to the sensitivity of the LVDT and thermal expansion of shear box, concrete block, and aluminum cap. This calibration was achieved by performing four tests in the consolidation phase under the three different normal stresses of 10psi, 16psi, and 21.755psi, in which heating and cooling were included. In these tests, the clay specimens were removed from the system (i.e., the concrete block in the lower shear box, filter paper, and porous stone in the upper shear box). For these tests, the
volume change was recorded along with the temperatures. The recorded volume change is due to the whole system without the clay specimen. Figure 3.21 and Figure 3.22 show the temperatures and corresponding volume change under heating and cooling of the calibration test under 10psi, 16psi, and 21.8psi, respectively.

Therefore, these volume changes were removed from the original test results according to the value of temperature and its corresponding volume change in the calibrated test. As a result, the final value of vertical displacement is the calibrated value for the clay specimen alone under heating and cooling.

3.9. Consolidation

Figure 3.23, Figure 3.25, Figure 3.27, and Figure 3.29 show the logarithmic plot of the consolidation for the tests that were subjected to a heating-cooling cycle under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. Figure 3.24, Figure 3.26, Figure 3.28, and Figure 3.30 show the logarithmic plot of the consolidation for the tests that were not subjected to a heating-cooling cycle.
under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. The values and shape of the consolidation curves are very similar between the tests that are subjected to a heating-cooling cycle and those that were not, under the same normal stress. This is because heating starts after the end of primary consolidation. Therefore, the similarities between the curves give a good indication of a good repeatability between the tests up to the end of primary consolidation. Figure 3.23 to Figure 3.30 also show that all tests reached to the end of primary consolidation before either heating or shearing start.

The value of primary consolidation is about 0.11 in, 0.145 in, 0.19 in, 0.25 in under 4.35psi, 10psi, 16psi, and 21.8psi normal stresses, respectively. It took around 1500 minutes for all tests to reach the end of primary consolidation.

Figure 3.23. Consolidation of low P.I. clay under 4.35psi normal stress and with one heating cycle up to 70°C.

Figure 3.24. Consolidation of low P.I. clay under 4.35psi normal stress without heating cycle.
Figure 3.25. Consolidation of low P.I. clay under 10psi normal stress and with one heating cycle up to 70°C.

Figure 3.26. Consolidation of low P.I. clay under 10psi normal stress without heating cycle.

Figure 3.27. Consolidation of low P.I. clay under 16psi normal stress and with one heating cycle up to 70°C.

Figure 3.28. Consolidation of low P.I. clay under 16psi normal stress without heating cycle.
Figure 3.29. Consolidation of low P.I. clay under 21.8psi normal stress and with one heating cycle up to 70°C.

Figure 3.30. Consolidation of low P.I. clay under 21.8psi normal stress without heating cycle

3.10. Thermal Loading

Figure 3.31-a, Figure 3.32-a, Figure 3.33-a, and Figure 3.34-a show the temperatures of the four thermocouples of the four tests under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. Where T1 is the temperature at the middle of the side, T2 is the temperature at the corner, T3 is the temperature at the center, and T4 is the temperature of the water in the water bath, as shown in Figure 3.20.

Figure 3.31-b, Figure 3.32-b, Figure 3.33-b, and Figure 3.34-b show the difference between the temperatures of the four thermocouples and their average temperature of the four tests under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. These are plotting the difference between T1, T2, T3, T4, and the average is essential for showing if T1, T2, T3, and T4 are in a homogenous or steady-state condition or not.

Condition 1 (i.e., temperature stabilization) was reached quickly within around 80 min after the first thermocouple reached the target temperature (70°C). At stage 1 of the thermal loading process (i.e., heating stage), the deviation between the thermocouples temperatures is high. It can be seen in Figure 3.31-b, Figure 3.32-b, Figure 3.33-b, and Figure 3.34-b that for all tests under the four
different normal stress, difference 2 and 4 are always positive values in stage 1, while differences 1 and 3 are always negative in stage 1. That means T1 and T4 are higher than the average temperature and that T2 and T3 are lower than the average temperature in the heating stage. This behavior is because T1 (adjacent to the shear box) and T4 (water temperature) are closer to the heat source (i.e., heaters). Therefore, T1 and T4 temperatures are higher than T2 (at the corner of the clay layer) and T3 (at the middle of the clay layer), and therefore, higher than the average temperature of T1, T2, T3, and T4. Following the same concept, it is clear how T2 and T3 would be less than the average temperature during the heating stage. Furthermore, the maximum difference between the thermocouples’ temperatures can be found between T2 and T3, which ranges between 19°C and 22°C in the four tests.

At the beginning of stage 2 of thermal loading (i.e., constant heating at the target temperature), the differences decrease rapidly to a point where the differences between the four temperatures and the average become very close to the x-axis. The steady-state or homogeneity of temperatures is reached at this point when the differences reach this level (i.e., very close to the x-axis) with a constant value or with few fluctuations ±1°C. Under 16psi and 21.8psi, there seem to be higher fluctuations in stage 2 of heating. This is due to the fluctuations within the temperatures themselves due to uncontrolled room temperature in the laboratory where the experiments were held. However, the fluctuation is small (about +−2°C), which is believed to be negligible. When the heaters are shut down and cooling stage (i.e., stage 3) starts, the temperatures of the thermocouples become highly heterogeneous at the beginning of the cooling stage after the temperature of the system stabilized and all temperatures (T1, T2, T3, and T4) reached the target temperature. Moreover, the differences profile shown in Figure 3.31-b, Figure 3.32-b, Figure 3.33-b, and Figure 3.34-b show that differences 2 and 3 are positive, and differences 1 and 4 are negative. This
behavior indicates that temperatures T2 and T3 are higher than the average temperature, while temperatures T1 and T4 are less than the average temperature. The reason is that water temperature (T4) and T1 would lose temperature higher than T2 and T3 because they are the closest to the heat source. Furthermore, the highest difference between the thermocouples (i.e., between T4 and T3) ranges between 10°C and 14°C during the cooling stage. As cooling proceeded, the differences between temperatures and the average temperature decrease until nearing or reaching the room temperature of 20°C ±2°C.

Yazadani et al. (2018) conducted direct shear tests under thermal cyclic loading up to 34°C of medium P.I. (P.I. =20) clay-concrete interface. By studying the temperature profile with time presented by Yazadani et al. (2018), the same observation is found, where they have three thermocouples, two at the interface, and one inside the water. During the heating phase, Yazadani et al. (2018) had water temperature lower than the average temperature, the difference between temperatures and the average decreases as they enter phase 2 of heating and water temperature becomes higher than the average in the cooling stage.
Figure 3.31. Under 4.35 psi and one cycle of temperature up to 70°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

Figure 3.32. Under 10 psi and one cycle of temperature up to 70°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.
Figure 3.33. Under 16psi and one cycle of temperature up to 70°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

Figure 3.34. Under 21.8psi and one cycle of temperature up to 70°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

3.11. Vertical Displacement Response Under the Heating-Cooling Cycle

Figure 3.35-a, Figure 3.36-a, Figure 3.37-a, and Figure 3.38-a show the average temperature of the four thermocouples under normal stresses of 4.35psi, 10psi, 16psi, and 21.8psi. Figure 3.35-b, Figure 3.36-b, Figure 3.37-b, and Figure 3.38-b show the calibrated values and the uncalibrated (i.e., with heating) values of vertical displacement under normal stresses of 4.35psi, 10psi, 16psi, and 21.8psi. The target temperature was 70 °C. In stage 1 (i.e., heating), the temperature was
increased from room temperature (usually around 20-23 °C) to 70 °C in approximately 150-200 minutes. After reaching the target temperature of 70°C, the temperature was kept constant at 70°C in stage 2. Condition 2 was satisfied after around 100 min after reaching the target temperature (70°C) for all tests except for 4.35psi, which needed 400 min, as shown in figure 3.35-b. However, stage 2 for all tests was kept for 500-550 min. Figure 3.35-b, Figure 3.36-b, Figure 3.37-b, and Figure 3.38-b show there was no further change in volume (i.e., vertical displacement) before cooling. In addition, under cooling in phase 3, there is a volume change. Before starting shearing, no further volume change occurred. Furthermore, Figure 3.35-b, Figure 3.36-b, Figure 3.37-b, and Figure 3.38-b show two values of vertical displacement, with heating and calibrated values. These two values will be discussed furthermore in the next sections. During the cooling stage, the heaters were shut down, and the temperatures dropped to the room temperature again. The cooling stage took around 750 min.
Figure 3.35. Low P.I. soil under 4.35psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.

Figure 3.36. Low P.I. soil under 10psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.
3.12. Test Results and Discussion

Figure 3.37. Low P.I. soil under 16psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.

Figure 3.38. Low P.I. soil under 21.8psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.

Figure 3.39 presents the horizontal displacements versus the vertical displacements during shearing under the four different normal stresses of the clay-concrete interface. It also shows the results for the tests that were subjected to one heating-cooling cycle and those without heating. The maximum values of vertical displacements for the tests that were subjected to a heating-cooling cycle ranged between 0.01in and 0.024in under the four normal stresses. Furthermore, it ranged between 0.015in and 0.031in for the tests that were not subjected to a heating-cooling cycle.
Figure 3.39. Vertical displacement vs. horizontal displacement behavior under shearing without heating and with heating.

Figure 3.40 depicts the shear strength versus horizontal displacement under the four different normal stresses of the clay-concrete interface with and without one heating-cooling cycle. It is clear that the shear strength of clay-concrete increases with increasing normal stress. The shear strength of clay-concrete also increased after applying a heating-cooling cycle up to 70°C with comparison to the tests that were not subjected to a heating-cooling cycle. The maximum shear strength is 2.43psi, 5.22psi, 7.86psi, and 11.7psi under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. In addition, displacement at maximum stresses ranged between 0.183in to 0.199in. The maximum shear strength for the tests without heating-cooling cycle are 1.88psi, 4.31psi, 6.65psi, and 10.4psi under 4.35psi, 10psi, 16psi, and 21.8psi, respectively, with a range of displacements between 0.175in and 0.22in. Table 3.1 shows the detailed results of the low P.I. Clay-concrete interface shear strength results.
Figure 3.40. Shear stress with a horizontal displacement of clay concrete interface without heating and with heating.

Table 3.1. Interface shear strength results of low P.I. clay-concrete.

<table>
<thead>
<tr>
<th>Heating Condition</th>
<th>Normal Stress (psi)</th>
<th>Maximum Shear Strength (psi)</th>
<th>Displacement at Maximum Shear Strength (in)</th>
<th>Residual Shear Strength (psi)</th>
<th>% Increase in Shear Strength (peak) after heating</th>
<th>% Increase in Shear Strength (residual) after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Heating</td>
<td>4.35</td>
<td>2.43</td>
<td>0.1985</td>
<td>2.27</td>
<td>29.3</td>
<td>36.7</td>
</tr>
<tr>
<td>Without Heating</td>
<td>4.35</td>
<td>1.88</td>
<td>0.176</td>
<td>1.66</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>10</td>
<td>5.22</td>
<td>0.19</td>
<td>4.64</td>
<td>21.1</td>
<td>11.0</td>
</tr>
<tr>
<td>Without Heating</td>
<td>10</td>
<td>4.31</td>
<td>0.22</td>
<td>4.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>16</td>
<td>7.86</td>
<td>0.183</td>
<td>7.0</td>
<td>18.2</td>
<td>21.5</td>
</tr>
<tr>
<td>Without Heating</td>
<td>16</td>
<td>6.65</td>
<td>0.175</td>
<td>5.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>21.8</td>
<td>11.7</td>
<td>0.197</td>
<td>11.2</td>
<td>12.5</td>
<td>17.2</td>
</tr>
<tr>
<td>Without Heating</td>
<td>218</td>
<td>10.4</td>
<td>0.187</td>
<td>9.56</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The peak and residual shear strengths of all tests on the shear strength-normal stress plane, adopting the Mohr-coulomb failure criteria, were plotted, as shown in Figure 3.41 and Figure 3.42.
respectively. To provide a better fit for the data, the intercepts of the failure envelopes were fixed at zero following Murphy and McCartney (2014) work.

Figure 3.41. Effect of heating cooling-cycle on the peak shear strength with normal stresses at failure envelope.

Figure 3.42. Effect of heating cooling-cycle on the residual shear strength with normal stresses at failure envelop.

Figure 3.43 shows the shear stress response of clay-clay and clay-concrete under both, with and without heating cycles under 4.35psi, 10psi, 16psi, and 21.8psi. As shown, the shear strength of clay-clay is higher than that of the clay-concrete interface. Table 3.2 present the clay-concrete shear strength with and without heating as a percentage of clay-clay shear strength. The shear strength of clay-concrete without heating ranges between 52% to 72% of clay-clay shear strength under all normal stress from 4.35psi to 21.8psi. These percentages increased to be within the range of 60% to 81% after a heating-cooling cycle up to 70°C.
Figure 3.43. Shear stress response of low P.I. clay of clay-clay and clay-concrete tests.

Table 3.2. Low P.I. clay-concrete shear strength as a percentage of clay-clay shear strength.

<table>
<thead>
<tr>
<th>Normal stress (psi)</th>
<th>$\frac{\tau_u (Clay-Concrete)}{\tau_u (Clay-Clay)}$ % Without heating</th>
<th>$\frac{\tau_u (Clay-Concrete)}{\tau_u (Clay-Clay)}$ % With heating</th>
<th>% Difference after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.35</td>
<td>52</td>
<td>60</td>
<td>8</td>
</tr>
<tr>
<td>10</td>
<td>61</td>
<td>76</td>
<td>15</td>
</tr>
<tr>
<td>16</td>
<td>64</td>
<td>74</td>
<td>10</td>
</tr>
<tr>
<td>21.8</td>
<td>72</td>
<td>81</td>
<td>9</td>
</tr>
</tbody>
</table>

Figure 3.44 shows the vertical displacement response under shearing of clay-clay and clay-concrete under both, with and without heating cycles under 4.35psi, 10psi, 16psi, and 21.8psi. The vertical displacement of clay-clay tests is higher than clay-concrete tests, with and without heating under all normal stresses.
Figure 3.44. Vertical displacement response of low P.I. clay of clay-clay and clay-concrete tests.

The sudden heating of the saturated clay will cause pore thermal water pressure generation since the rate of heating is faster than the rate of the thermal pore water pressure dissipation (Abuel-Naga et al., 2006; Lalouï, 2001; Towhata et al., 1993). Reduction in the volumetric strain was observed due to the dissipation of pore water pressure and physico-chemical structure interaction (Campanella and Mitchell, 1968). Following Figure 3.35-b, Figure 3.36-b, Figure 3.37-b, and Figure 3.38-b a volume reduction is observed until reaching the target temperature followed by fluctuations but tending to have a constant volume response during the constant high temperature. In contrast, in Figure 3.35-b, under normal stress of 4.35psi, slight volume expansion is observed at the beginning of heating before following the same trend shown Figure 3.36-b, Figure 3.37-b, and Figure 3.38-b. In addition, under 4.35psi, the volumetric deformation took more time to stabilize compared to the behavior of other tests. An excessive contraction is observed during the cooling phase under all normal stresses. This behavior is well known for the normally consolidated soil (Abuel-Naga et al., 2006; Lalouï, 2001; Towhata et al., 1993). The volumetric strain due to heating-cooling cycle was 0.8%, 0.5%, 0.6%, and 0.7% under 4.35psi, 10psi, 16psi, and 21.8psi,
respectively, as shown in Figure 3.45. At the heating stage, all tests have a positive Thermally Induced Volumetric Strain (TIVS) (contraction) up to around 0.3%, except for the tests under 4.35psi where it has a negative TIVS (expansion) up to -0.1%. Stage 2 of heating (heating at the target temperature) is depicted as a horizontal line in Figure 3.45, where all tests show an increase in TIVS (contraction) in this stage. In stage 3 of heating (i.e., cooling), small contraction is observed for all tests except for the 4.35psi test, where it has a high contraction. The same observations with regards to TIVS trend were found for the normally consolidated clay by Towhata et al. (1994), Baldi et al. (1988), Graham et al. (2000), Burghignoli et al. (1999), Abuel-Naga et al. (2006-a, 2007), and other studies. When comparing the final TIVS after the heating cycle with respect to the normal stresses (4.35psi, 10psi, 16psi, and 21.8psi), it is found that TIVS does not change when increasing the normal stress, which agrees with Abuel-Naga et al. (2007) and Burghignoli et al. (2000) work. TIVS is independent of normal stress because TIVS is controlled by the amount and rate of dissipation of pore water pressure, which depends mainly on thermal load (applied temperature) and material properties (hydraulic conductivity, thermal conductivity, plasticity index, …, etc.). Since the four tests have the same material properties and the same thermal load (ΔT=50°C), it is reasonable to have similar TIVS. However, they are not exactly the same, which is attributed to preparation conditions as described by Yazadani et al. (2019).

The volumetric response with the horizontal displacement of the clay-concrete interface without heating (Figure 3.39) shows that the vertical displacement under shearing increases with increasing the normal stress. The rate of vertical displacement is much higher within the first 0.2 inches of horizontal displacement as compared to the next 0.4 inches.
This observation is almost the same under all normal stresses but at a higher rate with increasing normal stress. In the same figure, the results of the tests that were subjected to one heating-cooling cycle almost follow the same trend to those that were not subjected to the heating cycle. The vertical displacement increases with increasing normal stress. The rate of vertical displacement with horizontal displacement in the first 0.2 in is much higher than the following. However, when comparing the shear tests that were subjected to heating cycle with those without heating, under the same normal stress, higher vertical displacement was observed for the tests that were not subjected to heating cycle under all normal stresses. After heating-cooling cycle, the maximum vertical displacement was reduced by 30.0%, 24.4%, 11.3%, and 24.2% under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. The reason behind this volumetric observation under shearing after a heating-cooling cycle is because the material was further consolidated due to thermal loading, which reduced the initial void ratio, which is known as a thermally induced overconsolidation effect (Di Donna et al., 2015).
The results of shear strength versus horizontal displacement curves for the clay-concrete interface with and without heating-cooling cycle (Figure 3.40) demonstrate that, in all cases, the interface shear strength of specimens subjected to one heating-cooling cycle is higher than those without heating. This observation shows that the increase in peak shear strength is 29.3%, 21.1%, 18.2%, and 12.5% under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. Moreover, the increase in residual shear strengths are 36.7%, 11.0%, 21.35%, and 17.15% under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. Di Donna et al. (2015) conducted direct shear tests on the clay-concrete interface at 20°C and 50°C without heating cycles using illite clay with P.I. =23.4. The percentage increases found by Di Donna et al. (2015) was about 36%, 38%, and 18% under normal stresses of 7.25psi (50kPa), 14.5psi (100kPa), and 21.8psi (150kPa), respectively. Therefore, the range of percentages increase in this study falls within the range that was found by Di Donna et al. (2015).

At the peak shear strength, the interface friction angle, $\delta$, increased by 13.6%, from 24.3° to 27.6° (Figure 3.41); while $\delta$ increased by 15.6%, from 22.4° to 25.9°, at the residual strength (Figure 3.42). Furthermore, the percentage of the interface friction angle, $\delta$ to the clay-clay friction angle, $\phi$, has increased from 78% to 89% at the peak after heating, and from 72% to 83% at the residual. These percentages range between 72% and 89% indicates that the failure would happen at the clay-concrete interface (Di Donna et al., 2015). According to Di Donna (2014), Di Donna and Laloui (2013), Yavari et al. (2016), and Yazdani et al. (2019), the reason behind the increase in shear strength after the heating-cooling cycle is the thermal consolidation or thermal solidification. The same explanation is believed to be the reason behind the increase in shear strengths and shear strength parameters in this study.
CHAPTER 4.
MEDIUM P.I. SOIL UNDER ONE HEATING-LOADING CYCLE AT 70°C

This chapter starts by describing the characteristics of the medium P.I. clay soil in detail along with the shear strength properties obtained from the small-size direct shear device. In addition, heating methodology, calibration, consolidation, and thermal loading were presented. The interface shear strength results obtained from the large-size direct shear device for the tests that were subjected to a heating-cooling cycle at 70°C temperature and the tests that were tested without heating along with the thermally induced volumetric strains were analyzed and discussed in detail.

4.1. Clay Characteristics

The clay used in this part of the project was a Medium P.I. soil with a liquid limit = 49, plastic limit = 20, plasticity index = 29. In addition, 88% passing sieve # 200, and 100% passing sieve # 4. According to AASHTO classification, this soil is classified as clayey soil as a silty clay in A-7-5 soil group. According to USCS classification, this soil is classified as low plasticity-high plasticity clay as CL-CH soil. It has a specific gravity of 2.67, it consists of 46% silt, 11% sand, and 43% clay, and the preconsolidation pressure is 6.25psi. The optimum moisture content, according to the standard compaction test, is 27.2%.

The direct shear tests of the clay-clay interface were performed in a small-size direct shear device (SDSD). Figure 4.1 show the corrected shear stress response with horizontal displacement under shearing for the medium P.I. soil under 10psi, 16psi, and 21.8psi normal stresses. Figure 4.2 shows the vertical displacement response under shearing. Following the Mohr column criteria, the shear strength parameters of the medium plasticity clay are the internal friction angle (φ) at the peak is 17.0°, while cohesion at the peak is 1.4psi, as shown in Figure 4.3.
4.2. Clay-Concrete Tests

Unlike the low P.I. clay, six tests were conducted using this soil. The soil was tested under three different normal stresses for both heating and unheating conditions. The three selected normal stresses (10psi, 16psi, and 21.8psi) were selected for shear testing.

![Figure 4.1. Shear stress with horizontal displacement without heating for medium plasticity clay.](image)

![Figure 4.2. Vertical displacement vs. horizontal displacement behavior under shearing without heating for medium plasticity clay](image)

![Figure 4.3. Shear strength parameters of medium plasticity clay without heating.](image)
4.3. Sample Preparation

The clay specimen was prepared using the same way and procedure that were applied to the low P.I. clay. It was compacted to the wet density of 1.77 g/cm³ at a moisture content of 33.5% (dry density of 1.32g/cm³) by targeting the standard compaction effort of 12400ft-lbf/ft³. The initial degree of saturation (S) is 88% and 1.01 void ratio (e).

4.4. Consolidation

Figure 4.4, Figure 4.6, and Figure 4.8 show the logarithmic plot of the consolidation for the tests that were subjected to a heating-cooling cycle under 10psi, 16psi, and 21.8psi, respectively. Figure 4.5, Figure 4.7, and Figure 4.9 show the logarithmic plot of the consolidation for the tests that were not subjected to a heating-cooling cycle under 10psi, 16psi, and 21.8psi, respectively. The values and shape of the consolidation curves are very similar between the tests that are subjected to a heating-cooling cycle and those that were not, under the same normal stress. This is because heating starts after the end of primary consolidation, just like the low P.I. clay. Figure 4.4 to Figure 4.9 also show that all tests reached to the end of primary consolidation before either heating or shearing start.

The value of primary consolidation is about 0.175 in, 0.23 in, 0.29 in under 10psi, 16psi, and 21.8psi normal stresses, respectively. It took around 3200 minutes for all tests to reach to the end of primary consolidation.
Figure 4.4. Consolidation of medium P.I. clay under 10psi normal stress and with one heating cycle up to 70°C.

Figure 4.5. Consolidation of medium P.I. clay under 10psi normal stress without heating cycle.

Figure 4.6. Consolidation of medium P.I. clay under 16psi normal stress and with one heating cycle up to 70°C.

Figure 4.7. Consolidation of medium P.I. clay under 16psi normal stress without heating cycle.
Figure 4.8. Consolidation of medium P.I. clay under 21.8psi normal stress and with one heating cycle up to 70°C.

Figure 4.9. Consolidation of medium P.I. clay under 21.8psi normal stress without heating cycle.

4.5. Thermal Loading

Figure 4.10-a, Figure 4.11-a, and Figure 4.12-a show the temperatures of the four thermocouples of the three tests under 10psi, 16psi, and 21.8psi, respectively. Figure 4.10-b, Figure 4.11-b, and Figure 4.12-b show the difference between the temperatures of the four thermocouples and their average temperature of the three tests under 10psi, 16psi, and 21.8psi, respectively. Condition 1 (i.e., temperature stabilization) was reached quickly within 60 minutes after the first thermocouple reached the target temperature (70 °C). Unlike the results of the low P.I. clay, it can be seen in Figure 4.10-b, Figure 4.11-b, and Figure 4.12-b that differences 1, 2, and 4 are always positive values in stage 1, while difference 3 are always negative in stage 1 under normal stresses of 16psi and 21.8psi. Under 10psi, differences 2 and 4 are positive, difference 3 is negative, and difference 1 is very close to the zero value (i.e., the average) with a small negative value. That means T1, T2, and T4 are higher than the average temperature and that T3 is lower than the average temperature in the heating stage under 16 and 21.8psi. The same observation applies for 10psi normal stress tests, but except for T1, which has a value very close to the average temperature. In addition, T3
is less than the average temperature during the heating stage. The maximum difference between the thermocouples’ temperatures can be found between T2 and T3, which ranges between 22°C and 25°C in the three tests, unlike the low P.I. soil, which has a range of 19°C to 22°C.

At the beginning of stage 2 of thermal loading, the differences decrease rapidly to a point where the differences between the four temperatures and the average become very close to the x-axis ($\pm 1.5^\circ$C) under 10psi and 16psi.

Figure 4.10. Under 10psi and one cycles of temperature up to 70°C for medium P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

Figure 4.11. Under 16psi and one cycles of temperature up to 70°C for medium P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.
Figure 4.12. Under 21.8psi and one cycles of temperature up to 70°C for medium P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

Under 21.8psi, the differences are within ±3°C from the negative axis because of a drop in temperature in the laboratory during the time of testing. In the cooling stage, the differences profile shown in Figure 4.10-b, Figure 4.11-b, and Figure 4.12-b show that difference 3 is positive, and differences 2 and 4 are negatives for all tests. However, difference 1 is very close to the average value (i.e., x-axis) under 10psi and has a negative value under 16psi and 21.8psi. This observation indicates that temperature T3 is higher than the average temperature, while temperatures T2 and
T4 are less than the average temperature. T1 is less than the average temperature under 16psi and 21.8psi while close to the average under 10psi. Furthermore, the highest difference between the thermocouples (i.e. between T4 and T3) ranges between 11°C and 14°C during the cooling stage, which is close to that of the low P.I. soil (between 10°C and 14°C). Once again, as cooling proceeded, the differences between temperatures and the average temperature decrease until reaching or coming close to the room temperature of 20°C ±2°C.

**4.6. Vertical Displacement Response Under the Heating-Cooling Cycle**

Same as the low P.I. clay, thermal loading was applied immediately after consolidation for the tests that are subjected to a heating-cooling cycle, and the target temperature was 70°C. Figure 4.13-a, Figure 4.14-a, and Figure 4.15-a show the average temperature of the four thermocouples under normal stresses of 10psi, 16psi, and 21.8psi. Figure 4.13-b, Figure 4.14-b, and Figure 4.15-b show the calibrated volume change with the uncalibrated (with heating) of the three tests.

The temperature was increased from room temperature (usually from 20-23°C) to 70°C in around 150-200 minutes, as shown in Figure 4.13-a, Figure 4.14-a, and Figure 4.15-a. The cooling stage took around 700 minutes. Condition 2 (volume stabilization due to heating) was satisfied after around 600 minutes after reaching the target temperature (70 °C) for the tests under 10 and 21.8psi, while it took 800 min for the tests that were under normal stress of 16psi to reach to a volume stabilization as shown in Figure 4.14-b. However, stage 2 for all tests was kept for around 1000 minutes. Figure 4.13-b, Figure 4.14-b, and Figure 4.15-b show that there was no further change in volume (i.e., vertical displacement) before cooling. In addition, under cooling in phase 3, an excessive reduction in vertical displacement is observed until reaching room temperature.
Figure 4.13. Medium P.I. clay under 10psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.

Figure 4.14. Medium P.I. clay under 16psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.
Figure 4.15. Medium P.I. clay under 21.8psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.

4.7. Test Results and Discussions

Figure 4.16 presents the horizontal displacements versus the vertical displacements during shearing under the three different normal stresses of medium P.I. clay-concrete interface. It also shows the results for the tests that were subjected to one heating-cooling cycle and those without heating. The maximum values of vertical displacements for the tests that were subjected to a heating-cooling cycle ranged between 0.0165in and 0.015in under the three normal stresses. In
addition, it ranged between 0.019in and 0.033in for the tests that were not subjected to a heating-cooling cycle.

Figure 4.16. Vertical displacement vs. horizontal displacement behavior under shearing without heating and with heating.

Figure 4.17 depicts the shear strength versus horizontal displacement under the three different normal stresses of the medium P.I. clay-concrete interface with and without one heating-cooling cycle. The maximum shear strength is 2.92psi, 4.9psi, and 6.75psi under 10psi, 16psi, and 21.8psi, respectively, for the tests that were subjected to a heating-cooling cycle up to 70°C. Furthermore, displacement at maximum stresses ranged between 0.056in to 0.072in. The maximum shear strength for the tests without heating-cooling cycle are 2.26psi, 3.85psi, and 5.33psi under 10psi, 16psi, and 21.8psi, respectively, with a range of displacements between 0.11in and 0.18in. Table 4.1 show detailed results of the large-size direct shear tests.
Figure 4.17. Shear stress with a horizontal displacement of clay concrete interface without heating and with heating.

<table>
<thead>
<tr>
<th>Heating Condition</th>
<th>Normal stress (psi)</th>
<th>Maximum shear strength (psi)</th>
<th>Displacement at maximum shear strength (in)</th>
<th>Residual Shear strength (psi)</th>
<th>% Increase in shear strength at the peak after heating</th>
<th>% Increase in shear strength at residual after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Heating</td>
<td>10</td>
<td>2.92</td>
<td>0.06</td>
<td>2.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without Heating</td>
<td>10</td>
<td>2.26</td>
<td>0.11</td>
<td>1.85</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>16</td>
<td>4.9</td>
<td>0.56</td>
<td>4.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without Heating</td>
<td>16</td>
<td>3.85</td>
<td>0.167</td>
<td>3.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>21.8</td>
<td>6.75</td>
<td>0.072</td>
<td>5.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without Heating</td>
<td>218</td>
<td>5.33</td>
<td>0.18</td>
<td>4.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The peak and residual shear strengths of all tests on the shear strength-normal stress plane, adopting the Mohr-coulomb failure criteria, were plotted, as shown in Figure 4.18 and Figure 4.19,
respectively. To provide a better fit for the data, the intercepts of the failure envelopes were fixed at zero following the work of Murphy and McCartney (2014).

Figure 4.18. Effect of heating cooling-cycle on the peak shear strength with normal stresses at failure envelope.

Figure 4.19. Effect of heating cooling-cycle on the residual shear strength with normal stresses at failure envelope.

Figure 4.20 shows the shear stress response of medium P.I. clay-clay and medium P.I. clay-concrete under both, with and without heating cycles under 10psi, 16psi, and 21.8psi. Table 4.2 present the clay-concrete shear strength at peak with and without heating as a percentage of clay-clay shear strength. The shear strength of clay-concrete without heating ranges between 50% to 66% of clay-clay shear strength under all normal stress from 10psi to 21.8psi. These percentages increased to be within the range of 65% to 83.5% after a heating-cooling cycle up to 70 C.
Figure 4.20. Shear stress response of medium plasticity clay of clay-clay and clay-concrete tests.

Table 4.2. Medium P.I. Clay-concrete shear strength as a percentage of clay-clay shear strength.

<table>
<thead>
<tr>
<th>Normal stress (psi)</th>
<th>$\frac{\tau_u (\text{Clay-Concrete})}{\tau_u (\text{Clay-Clay})}$% Without heating</th>
<th>$\frac{\tau_u (\text{Clay-Concrete})}{\tau_u (\text{Clay-Clay})}$% With heating</th>
<th>% Difference after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>50.4</td>
<td>65.2</td>
<td>14.7</td>
</tr>
<tr>
<td>16</td>
<td>61.9</td>
<td>78.8</td>
<td>16.9</td>
</tr>
<tr>
<td>21.8</td>
<td>66</td>
<td>83.5</td>
<td>17.6</td>
</tr>
</tbody>
</table>

Figure 4.21 shows the vertical displacement response under shearing of medium P.I. clay-clay and medium P.I. clay-concrete under both, with and without heating cycles under 10psi, 16psi, and 21.8psi. The vertical displacement of clay-clay tests is higher than clay-concrete tests, with and without heating under all normal stresses. However, the vertical displacement under shearing of clay-clay tests is very close in value to each other. In addition, no clear observation could be stated about the differences between the different tests under different normal stresses in this regard.
Figure 4.21. Vertical displacement response of medium plasticity clay of clay-clay and clay-concrete tests.

Following Figure 4.13-b, Figure 4.14-b, and Figure 4.15-b, a volume expansion is observed at the beginning of heating followed by a volume contraction just before reaching the target temperature of 70°C. In stage 2 of heating (i.e., heating at a constant target temperature of 70°C), excessive contraction is observed until reaching stabilization of volume. A further excessive contraction is observed during the cooling phase under all normal stresses. This behavior is well known for the normally consolidated soil (Abuel-Naga et al., 2006; Laloui, 2001; Towhata et al., 1993). The volumetric strain due to the heating-cooling cycle was 1.0%, 0.91%, and 0.89% under 10 psi, 16 psi, and 21.8 psi, respectively, as shown in Figure 4.22. At heating stage, no TIVS is observed (0.0 ± 0.1%) for all tests. All tests show an increase in TIVS in stage 2 of heating by almost the same value. In stage 3 of heating, small contraction is observed for all tests. Figure 4.22 shows that all tests of the medium P.I. clay has a very similar shape and values of TIVS. The same observations with regards to TIVS trend were found for the normally consolidated clay by Towhata et al. (1994), Baldi et al. (1988), Graham et al. (2000), Burghignoli et al. (1999), Abuel-Naga et al. (2006-a, 2007), and other researches. Figure 4.22 also shows that TIVS after the heating-cooling cycle
remains the same with increasing the normal stress (Abuel-Naga et al., 2007). Furthermore, when comparing the TIVS values of the medium P.I. to those of low P.I., it is clear that TIVS increase with increasing plasticity index, which is also shown by Abuel-Naga et al. (2007).

Figure 4.22. Profile of thermally induced volumetric strain with temperature difference for the medium P.I. clay-concrete interface under 70°C heating cycle.

The volumetric response with the horizontal displacement of the clay-concrete interface without heating (Figure 4.16) shows that the vertical displacements under shearing increase with increasing normal stress. The rate of vertical displacement is much higher within the first 0.3 inches of horizontal displacement as compared to the next 0.3 inches under 16psi and 21.8psi. Under 10psi, the high rate of vertical displacement stops at 0.4 inches, but at a value that is half of that at 16psi and 21.8psi. In addition, the rate of vertical displacement increases with increasing normal stress under all normal stresses. In the same figure, the results of the tests that were subjected to one heating-cooling cycle almost follow the same trend regarding shape and value. Therefore, no significant difference in the vertical displacement values or shape is observed for the tests with heating. The rate of vertical displacement with horizontal displacement in the first 0.25 inches is much higher than the following 0.35 inches. However, when comparing the shear tests that were
subjected to heating cycle with those without heating, under the same normal stress, higher vertical displacement was observed for the tests that were not subjected to heating cycle under all normal stresses. After the heating-cooling cycle, the maximum vertical displacement was reduced by 13.2%, 43.8%, 54.0% under 10psi, 16psi, and 21.8psi, respectively. This reduction (contraction) in vertical displacement is attributed to the thermally induced overconsolidation effect and reduction in void ratio before shearing (Di Donna, 2015).

The results of shear strength versus horizontal displacement curves for the clay-concrete interface with and without heating-cooling cycle (Figure 4.17) demonstrate that, in all cases, the interface shear strength of specimens subjected to one heating-cooling cycle is higher than those without heating. This observation shows that the increase in peak shear strength is 29.2%, 27.3%, and 26.6%, under 10psi, 16psi, and 21.8psi, respectively. Furthermore, the increase in residual shear strengths are 35.1%, 37.0%, and 35.2% under 10psi, 16psi, and 21.8psi, respectively. These ranges fall within the range reported by Di Donna et al. (2015). At the peak shear strength, the interface friction angle, $\delta$, increased by 25.0%, from $17.0^\circ$ to $13.6^\circ$ (Figure 4.18); while $\delta$, increased by 35.5%, from $14.9^\circ$ to $11.0^\circ$, at the residual strength (Figure 4.19). According to Di Donna (2014), Di Donna and Laloui (2013), Yavari et al. (2016), and Yazdani et al. (2019), the reason behind the increase in shear strength after the heating-cooling cycle is the thermal consolidation or thermal solidification. The same explanation is believed to be the reason behind the increase in shear strengths and shear strength parameters in this study.
CHAPTER 5.
HIGH P.I. SOIL UNDER ONE HEATING-LOADING CYCLE AT 70°C

This chapter starts by describing the characteristics of the high P.I. clay soil in detail along with the shear strength properties obtained from the small-size direct shear device. In addition, heating methodology, calibration, consolidation, and thermal loading were presented. The interface shear strength results obtained from the large-size direct shear device for the tests that were subjected to a heating-cooling cycle 70°C temperature and the tests that were tested without heating along with the thermally induced volumetric strains were analyzed and discussed in detail.

5.1. Clay Characteristics

The geotechnical properties of the Fat clay used in this part of the project for high P.I. are a liquid limit = 96, plastic limit = 36, plasticity index = 60. In addition, 100% passing sieve #4 and 99% passing sieve #200. It has a specific gravity of 2.727, and it consists of 26% silt and 73% clay, and the preconsolidation pressure is 35psi. The soil is classified as a high plasticity clay or CH according to USCS soil classification, and a silty clay in A-7-6 group according to AASHTO soil classification. The optimum moisture content, according to the standard compaction test, is 46.2%.

Like the low P.I. clay and Medium P.I. clay, the direct shear tests of the clay-clay interface were performed in a small-size direct shear device (SDSD). Figure 5.1 shows the corrected shear stress response with horizontal displacement under shearing for the high P.I. clay under 10psi, 16psi, and 21.8psi normal stresses. Figure 5.2 shows the vertical displacement response under shearing of the same clay. Following the Mohr column criteria, the shear strength parameters of the high plasticity clay are; the internal friction angle at the peak ( φₚ ) is 12.6°, cohesion at the peak is 1.95psi, internal friction angle at the residual ( φᵣ ) is 11.0°, and the cohesion at the residual is 1.76psi, as shown in Figure 5.3.
Figure 5.1. Shear stress with horizontal displacement without heating for high plasticity clay.

Figure 5.2. Vertical displacement vs. horizontal displacement behavior under shearing without heating for high plasticity clay.

Figure 5.3. Shear strength parameters of high plasticity clay without heating.
5.2. Clay-Concrete Tests

Like the medium P.I. clay, six tests were conducted using this soil. The soil was tested under three different normal stresses for both, with heating and without heating conditions. The three selected different normal stresses (10psi, 16psi, and 21.8psi) were selected for shear testing.

5.3. Sample Preparation

The clay specimen was prepared inside the shear box over the concrete block. It was compacted to the wet density of 1.70g/cm$^3$ at a moisture content of 55% (dry density of 1.10g/cm$^3$) and 1.49 void ratio ($e$) by targeting the standard compaction effort of 12400ft-lbf/ft$^3$. The soil was taken from the Pavement Research Facility (PRF) located on a six-acre site near LA 1 south, across the Mississippi River from Baton Rouge, LA, from a shallow depth and sealed in plastic bags with 99% degree of saturation (S).

5.4. Consolidation

Figure 5.4, Figure 5.6, and Figure 5.8 show the logarithmic plot of the consolidation for the tests that were subjected to a heating-cooling cycle under 10psi, 16psi, and 21.8psi, respectively. Figure 5.5, Figure 5.7, and Figure 5.9 show the logarithmic plot of the consolidation for the tests that were not subjected to a heating-cooling cycle under 10psi, 16psi, and 21.8psi, respectively. Once again, the values and shape of the consolidation curves are very similar between the tests that are subjected to a heating-cooling cycle and those that were not, under the same normal stress. Figure 5.4 to Figure 5.9 also show that all tests did not reach to the end of primary consolidation before either heating or shearing start.

It took around 3000 minutes (two days) for all tests under 10psi, 16psi, and 21.8psi before starting the heating. The value of consolidation before heating is about 0.11in, 0.145 in, 0.19 in under 10psi, 16psi, and 21.8psi normal stresses, respectively.
Figure 5.4. Consolidation of high P.I. clay under 10psi normal stress with one heating cycle up to 70°C.

Figure 5.6. Consolidation of high P.I. clay under 16psi normal stress with heating cycle up to 70°C.

Figure 5.5. Consolidation of high P.I. under 10psi normal stress without one heating cycle.

Figure 5.7. Consolidation of high P.I. under 16psi normal stress without heating cycle.
5.5. Thermal Loading

Figure 5.8. Consolidation of high P.I. clay under 21.8psi normal stress with one heating cycle up to 70°C.

Figure 5.9. Consolidation under 21.8psi normal stress without heating cycle.

Figure 5.10-a, Figure 5.11-a, and Figure 5.12-a show the temperatures of the four thermocouples of the three tests under 10psi, 16psi, and 21.8psi, respectively. Figure 5.10-b, Figure 5.11-b, and Figure 5.12-b show the difference between the temperatures of the four thermocouples and their average temperature of the three tests under 10psi, 16psi, and 21.8psi, respectively. Temperature stabilization was reached quickly within 80 min after the first thermocouple reached the target temperature (70 °C). Like the low P.I. clay, it can be seen in Figure 5.10-b, Figure 5.11-b, and Figure 5.12-b that difference 2 and 4 are always positive values in stage 1, while differences 1 and 3 are always negative in stage 1 under normal stresses 16psi and 21.8psi. However, under 10psi, the behavior of differences 1 to 4 is similar to those of medium P.I. under 10psi, where differences 2 and 4 are positive, difference 3 is negative, and difference 1 is very close to zero value with a small negative value. That means T2 and T4 are higher than the average temperature and that T1 and T3 are lower than the average temperature in the heating stage under 16 and 21.8psi. The same observation applies for 10psi normal stress tests, but with the exception of T1, which has a value
very close to the average temperature. Furthermore, T3 is less than the average temperature during
the heating stage. The maximum difference between the thermocouples’ temperatures can be found
between T3 and T4, which is around 25°C under 16psi, 15°C under 21.8psi, but no exact value
under 10psi since heating did not start at 70°C. The ranges between the highest and lowest
temperature of medium P.I. clay is between 22°C and 25°C in the three tests, and between 19°C
and 22°C for the case of the low P.I. soil. Just like the previous clays, at the beginning of stage 2
of thermal loading, the differences decrease rapidly to a point where the differences between the
four temperatures and the average become very close to the x-axis (±3°C) under 10psi and 16psi.
Under 21.8psi, the differences are within ±2°C from the –axis. These ranges are comparable with
that of the medium P.I. clay results.

In cooling stage, the differences profile shown in Figure 5.10-b, Figure 5.11-b, and Figure 5.12-b
show that difference 3 is positive, and differences 2 and 4 are negatives for all tests. However,
difference 1 is very close to the average value (i.e. x-axis) but with a small negative value under
10psi and small positive value under 16psi and 21.8psi. This behavior indicates that temperature
T3 is higher than the average temperature, while temperatures T2 and T4 are less than the average
temperature. T1 is less than the average temperature under 16psi and 21.8psi while being close to
the average under 10psi. In addition, the highest difference between the thermocouples (i.e.,
between T4 and T3) ranges between 13°C under 10psi and 16psi while it is around 8°C under
21.8psi during the cooling stage, which is close to that of the low P.I. soil (between 10°C and
14°C). Once again, as cooling proceeded, the differences between temperatures and the average
temperature decrease until reaching or coming close to the room temperature of 20°C±2°C.
Figure 5.10. Under 10psi and one heating-cycle up to 70°C for high P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

Figure 5.11. Under 16psi and one heating-cycle up to 70°C for high P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.
Figure 5.12. Under 16psi and one heating-cycle up to 70°C for high P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

5.6. Vertical Displacement Response Under the Heating-Cooling Cycle

Unlike the medium and low P.I. clays, because primary consolidation did not finish after two days, it was determined to start heating after two days of consolidation. Like the medium and low P.I. clays, the temperature was increased from room temperature (usually from 20-23°C) to 70°C. Figure 5.13-a, Figure 5.14-a, and Figure 5.15-a show the average temperature of the four thermocouples and the corresponding volume change of the three phases under normal stresses of
10psi, 16psi, and 21.8psi. Figure 5.13-b, Figure 5.14-b, and Figure 5.15-b show the calibrated volume change with the uncalibrated (with heating) of the three tests.

It took around 200-300 minutes for heating to reach the target temperature for the tests that were under 16psi and 218psi. However, under 10psi, the temperature did not go up to 70°C at the beginning. This is because the test that was under 10psi normal stress was the first test subjected to heating, and therefore, heating setup and thermal loading were not well predicted. However, heating went to around 40°C and 45°C with some flocculation and increased to around 70°C at 2600 min, as shown in Figure 5.13-b.

There are multiple flocculation and a noticeable drop in temperatures for all tests. This is because the temperature at night used to drop because the water filling the water bath would be evaporated. The issue of evaporating water (i.e., drop in temperature) was solved later by having another two cells filled with water and connecting a very slow water charge pipe between the water cells to the water bath inside the large-size direct shear device. The issue of temperature fluctuation was solved by connecting the heaters to an eternal k-type thermocouple that is adjacent to the heaters instead of the heaters’ original thermocouple, which inserted inside it. Furthermore, the cooling stage took around 800 to 900 minutes.

Unlike the case of medium and low P.I. clays, condition 2 was not satisfied after two days of heating. Therefore, and based on condition 3 of heating duration, the heating was stopped after two days of heating. In addition, under cooling in phase 3, an excessive reduction in vertical displacement is observed until reaching room temperature.
Figure 5.13. High P.I. clay under 10psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.

Figure 5.14. High P.I. clay under 16psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.
Figure 5.15. High P.I. clay under 21.8psi. a. Vertical displacement with time for the calibrated and uncalibrated values, b. Average temperature.

5.7. Test Results and Discussions

Figure 5.16 presents the horizontal displacements versus the vertical displacements during shearing under the three different normal stresses of high P.I. clay-concrete interface. It also shows the results for the tests that were subjected to one heating-cooling cycle and those without heating. The maximum values of vertical displacements for the tests that were subjected to a heating-cooling cycle ranged between 0.001in and 0.003in under the three normal stresses. Furthermore,
it ranged between 0.020in and 0.027in for the tests that were not subjected to a heating-cooling cycle.

![Graph showing vertical displacement vs. horizontal displacement behavior](image)

**Figure 5.16.** Vertical displacement vs. horizontal displacement behavior under shearing without heating and with heating.

**Figure 5.17** depicts the shear strength versus horizontal displacement under the three different normal stresses of the high P.I. clay-concrete interface with and without one heating-cooling cycle. The maximum shear strength is 2.96psi, 4.6psi, and 5.8psi under 10psi, 16psi, and 21.8psi, respectively, for the tests that were subjected to a heating-cooling cycle up to 70°C. Moreover, displacement at maximum stresses ranged between 0.021in to 0.04in. The maximum shear strengths for the tests without heating-cooling cycle are 2.96psi, 3.7psi, and 4.77psi under 10psi, 16psi, and 21.8psi, respectively, with a range of displacements between 0.034in and 0.08in. Table 5.1 show detailed results of the large-size direct shear tests.
Figure 5.17. Shear stress with horizontal displacement of clay concrete interface without heating and with heating.

Table 5.1. Interface shear strength results of medium P.I. clay-concrete.

<table>
<thead>
<tr>
<th>Heating Condition</th>
<th>Normal stress (psi)</th>
<th>Maximum shear strength (psi)</th>
<th>Displacement at maximum shear strength (in)</th>
<th>Residual Shear strength (psi)</th>
<th>% Increase in shear strength at the peak after heating</th>
<th>% Increase in shear strength at residual after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Heating</td>
<td>10</td>
<td>3.54</td>
<td>0.036</td>
<td>2.3</td>
<td>19.6</td>
<td>6.5</td>
</tr>
<tr>
<td>Without Heating</td>
<td>10</td>
<td>2.96</td>
<td>0.08</td>
<td>2.16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>16</td>
<td>4.6</td>
<td>0.021</td>
<td>2.85</td>
<td>24.3</td>
<td>1.8</td>
</tr>
<tr>
<td>Without Heating</td>
<td>16</td>
<td>3.7</td>
<td>0.034</td>
<td>2.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>21.8</td>
<td>5.8</td>
<td>0.04</td>
<td>3.65</td>
<td>21.6</td>
<td>1.7</td>
</tr>
<tr>
<td>Without Heating</td>
<td>218</td>
<td>4.77</td>
<td>0.04</td>
<td>3.59</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The peak and residual shear strengths of all tests on the shear strength-normal stress plane, adopting the Mohr-coulomb failure criteria, were plotted, as shown in Figure 5.18 and Figure 5.19, respectively.
Figure 5.18. Effect of heating cooling-cycle on the peak shear strength with normal stresses at failure envelope.

Figure 5.19. Effect of heating cooling-cycle on the peak shear strength with normal stresses at failure envelope.

Figure 5.20 shows the shear stress response of high P.I. clay-clay and high P.I. clay-concrete under both, with and without heating cycles under 10psi, 16psi, and 21.8psi. Table 5.2 presents the clay-concrete shear strength at peak with and without heating as a percentage of clay-clay shear strength. The shear strength of clay-concrete without heating ranges around 69% of clay-clay shear strength under all normal stress from 10psi to 21.8psi. These percentages increase to be around 84% after a heating-cooling cycle up to 70°C.
Figure 5.20. Shear stress response of high plasticity clay of clay-clay and clay-concrete tests.

Table 5.2. High P.I. Clay-concrete shear strength as a percentage of clay-clay shear strength.

<table>
<thead>
<tr>
<th>Normal stress (psi)</th>
<th>$\frac{\tau_u (\text{Clay-Concrete})}{\tau_u (\text{Clay-Clay})}$ %</th>
<th>$\frac{\tau_u (\text{Clay-Clay})}{\tau_u (\text{Clay-Clay})}$ %</th>
<th>% Difference after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>69.5</td>
<td>83.1</td>
<td>13.6</td>
</tr>
<tr>
<td>16</td>
<td>68.8</td>
<td>85.5</td>
<td>16.7</td>
</tr>
<tr>
<td>21.8</td>
<td>69.1</td>
<td>84.1</td>
<td>14.9</td>
</tr>
</tbody>
</table>

Figure 5.21 show the vertical displacement response under shearing of medium P.I. clay-clay and high P.I. clay-concrete under both, with and without heating cycles under 10psi, 16psi, and 21.8psi. The vertical displacements of clay-clay tests are higher than clay-concrete tests, with and without heating under all normal stresses. However, the vertical displacements under shearing of clay-concrete after one heating-cooling cycle up to 70°C are very close in value to each other. In addition, the vertical displacement of clay-clay tests under shearing increases with increasing normal stress. That is, the highest vertical displacement under shearing is observed under 21.8psi, and the lowest is under 10psi.
Following Figure 5.13-b, Figure 5.14-b, and Figure 5.15-b, a volume expansion (i.e., reduction in vertical displacement) is observed at the beginning of heating before reaching the target temperature of 70°C. At the heating stage, all tests have a negative TIVS (expansion) up to around -0.5%, except for the tests under 10psi where it did not show the same behavior because it was not heated to 70°C directly as described earlier. This observation is because, for overconsolidated clay, a negative pore water pressure is developed under heating, which requires the sample to absorb more water, which translates into an expansion (Abuel-Naga et al., 2006-a, 2007). At stage 2 of heating, all tests show an increase in TIVS (contraction) without reaching the stabilization of volume in this stage. However, when the temperature drops in this stage, further TIVS is observed, and when temperature increases again, TIVS decreases slightly. The reason behind this contraction in stage 2 might not be due to thermal effect, but rather could be since the material did not reach the primary consolidation and more consolidation was taking place at the same time. A further excessive contraction is observed during the cooling phase under all normal stresses. The slope of TIVS with $\Delta T$ that shown in Figure 5.22 in stage 3 for all tests are very similar to each other and
very similar to the slope of TIVS in stage 1 under 16psi and 21.8psi. This indicates that the same amount of expansion occurs in stage 1 (heating) was fully recovered in stage 3 (cooling). The literature overwhelmingly agrees with this observation for overconsolidated clay under thermal loading (Towhata et al., 1994; Baldi et al., 1988; Graham et al., 2000; Burghignoli et al., 1999; Abuel-Naga et al., 2006-a, 2007), and other studies. The volumetric strain due to the heating-cooling cycle was 3.1%, 2.5%, and 3.35% under 10psi, 16psi, and 21.8psi, respectively, as shown in Figure 5.22.

The same observation is found once again for the overconsolidated high P.I. clay, where TIVS does not change with increasing the normal stress (Abuel-Naga et al., 2007; Burghignoli et al., 2000). However, under 16psi, TIVS is 2.5%, which is lower than under 10psi and 21.8psi due to the differences in heating conditions as well as preparation conditions as described by Yazadani et al. (2019). The average TIVS under all tests is 2.98%.

When comparing TIVS results for the different types of clay used in this study (i.e., low P.I. {12}, medium P.I. {30}, and high P.I. {60}), it is found that TIVS increases with increasing plasticity index. Abuel-Naga et al. (2006) found that TIVS is controlled by the plasticity index. Abuel-Naga et al. (2006) also gathered reported data from literature of TIVS values under thermal loading of about 65°C to 70°C of normally consolidated soil and different plasticities. The results of low P.I., medium P.I. and high P.I. were included in Figure 5.23. Based on Figure 5.23, it is shown that for our study with thermal loading of about 50°C (70°C-20°C=50°C) have a good fit. However, as mentioned earlier, the high P.I. is overconsolidated, and the TIVS in stage 2 might not be entirely attributed to thermal effect but to primary consolidation instead. If the value of TIVS gained under phase 2 of heating was removed, the final value of TIVS would be close to zero, which is agreed
upon in the literature (Towhata et al., 1994; Baldi et al., 1988; Graham et al., 2000; Burghignoli et al., 1999; Abuel-Naga et al., 2006-a, 2007), and other studies.

The volumetric response with the horizontal displacement of the clay-concrete interface without heating (Figure 5.16) shows that the vertical displacements under shearing increase with increasing...
normal stress. The rate of vertical displacement is much higher within the first 0.3 in horizontal displacement as compared to the next 0.3. This observation is almost the same under all normal stresses, but with a higher rate with decreasing normal stress. Figure 5.17 presents the results of vertical displacement with horizontal displacement for all tests (i.e., with heating and without heating). In this figure, it can be seen that vertical displacement under shearing decreases with increasing normal stress after 0.2 inches of horizontal displacement for the tests that were subjected to the heating-cooling phase. However, in the first 0.2in of horizontal displacement, the behavior under 10psi and 16psi is very similar, but less than the vertical displacement under the 21.8psi normal stress. It can also be noticed that the behavior for all the three tests that were subjected to the heating-cooling cycle shows a contraction followed by expansion followed by a contraction in a sinusoidal trend. Furthermore, the maximum value of vertical displacement for the three tests that were subjected the heating cycle never exceeded 0.004in, but 0.027in for those that were not subjected to the heating cycle. However, when comparing the shear tests that were subjected to heating cycle with those without heating, under the same normal stress, higher vertical displacement was observed for the tests that were not subjected to heating cycle under all normal stresses. After the heating-cooling cycle, the maximum vertical displacement was reduced by 85%, 87.7%, 96% under 10psi, 16psi, and 21.8psi, respectively. The reason behind this volumetric observation under shearing is because the material was further consolidated due to thermal loading, which reduced the initial void ratio (Di Donna et al., 2015).

The results of shear strength versus horizontal displacement curves for the clay-concrete interface with and without heating-cooling cycle (Figure 5.17) clearly demonstrate that, in all cases, the interface shear strength of specimens subjected to one heating-cooling cycle is higher than those without heating. This observation shows that the increase in peak shear strength is 19.6%, 24.3%,
and 21.6%, under 10psi, 16psi, and 21.8psi, respectively. In addition, no significant increase in the residual shear strengths was observed under the same tests. The reason behind the developed higher after the heating cycle is the increase in the overconsolidation ratio after the heating cycle (Di Donna et al., 2015; Abuel-Naga et al., 2007). Abuel-Naga et al. (2007) reported similar results with regards to increasing in peak shear strength (i.e., the peak developed and increased with increasing the temperature) while no increase of residual shear strength with comparison to the tests performed without heating of normally consolidated soil in a triaxial test. At the peak shear strength, the interface friction angle, $\delta$, increased by 24.1%, from 8.7° to 10.8° (Figure 5.18); while adhesion increased by 17.7% from 1.36psi to 1.6psi at peak (Figure 5.19). According to Di Donna (2014), Di Donna and Laloui (2013), Yavari et al. (2016), and Yazdani et al. (2019), the reason behind the increase in shear strength after the heating-cooling cycle is the thermal consolidation or thermal solidification. The same explanation is believed to be the reason behind the increase in shear strengths and shear strength parameters in this study.

When considering the relationships between the increase in shear strength of normally consolidated tests (low and medium P.I. clays) with plasticity index, it is found that the percentage increases in shear strength increase with increasing the plasticity indexes for peak and residuals as shown in Figure 5.24. The increase in residuals is almost higher than that of the peak for all tests. However, the increase in residuals for the overconsolidated high P.I. clays are almost zero, and the increase in peak strength is almost the same as the increase in normally consolidated medium P.I. clay. This observation is believed to be because of the higher TIVS observed for the medium P.I. compared to the low P.I. clay. While the high P.I. has the highest TIVS; it does not have the highest increase in shear strength due to its stress history condition (overconsolidation). The increase in interface friction angle shown in Figure 5.25 also demonstrates the same results.
Figure 5.24. Percent change in shear strength with plasticity index for all tests of clay-concrete interface.

Figure 5.25. Percent change in interface friction angle, $\delta$, with plasticity index of clay-concrete interface.
Chapter 6.
LOW P.I. SOIL UNDER ONE HEATING CYCLE AT 40°C AND AT 55°C

The low P.I clay was used for testing clay-concrete interface under another two targeted temperatures of 40°C and 55°C under three normal stresses of 10psi, 16psi, and 21.8psi. That is, the heating cycles are; (20°C – 40°C – 20°C) and (20°C – 55°C – 20°C). The preparation procedure followed under without heating condition, and the heating cycle of (20°C – 70°C – 20°C) was exactly followed for the 40°C and 50°C cycles.

6.1. Consolidation

Figure 6.1, Figure 6.3, and Figure 6.5 show the logarithmic plot of the consolidation for the tests that were subjected to a heating-cooling cycle up to 40°C under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. Figure 6.2, Figure 6.4, and Figure 6.6 show the logarithmic plot of the consolidation for the tests that were subjected to a heating-cooling cycle up to 55°C under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. The similarities between the curves can be identified easily through the figures. However, the amount of vertical displacement under consolidation is the final displacement shown in the figure minus the displacement taken under the saturation process (i.e., under the setting load of 0.5psi). Figure 6.1 through Figure 6.6 also show that all tests reached to the end of primary consolidation before either heating or shearing start.
Figure 6.1. Consolidation of low P.I. clay under 10psi normal stress and one heating cycle up to 40°C.

Figure 6.2. Consolidation of low P.I. clay under 10psi normal stress and one heating cycle up to 55°C.

Figure 6.3. Consolidation of low P.I. clay under 16psi normal stress and one heating cycle up to 40°C.

Figure 6.4. Consolidation of low P.I. clay under 16psi normal stress and one heating cycle up to 55°C.
6.2. Thermal Loading

Figure 6.5. Consolidation of low P.I. clay under 21.8psi normal stress and one heating cycle up to 40°C.

Figure 6.6. Consolidation of low P.I. clay under 21.8psi normal stress and one heating cycle up to 55°C.

Figure 6.7-a, Figure 6.9-a, and Figure 6.11-a show the temperatures of the four thermocouples of the four tests under 40°C heating cycle under 10psi, 16psi, and 21.8psi, respectively. Figure 6.7-b, Figure 6.9-b, and Figure 6.11-b show the difference between the temperatures of the four thermocouples and their average temperature of the four tests under 40°C heating cycle under 10psi, 16psi, and 21.8psi, respectively. Figure 6.8-a, Figure 6.10-a, and Figure 6.12-a show the temperatures of the four thermocouples of the four tests under 55°C heating cycle 10psi, 16psi, and 21.8psi, respectively. Figure 6.8-b, Figure 6.10-b, and Figure 6.12-b show the difference between the temperatures of the four thermocouples and their average temperature of the four tests under 55°C under 10psi, 16psi, and 21.8psi, respectively. The temperature stabilization was reached quickly within around 40 minutes to 60 minutes after the first thermocouple reached the target temperatures (40°C and 55°C, respectively). It can be seen in Figure 6.7 through Figure 6.12 that for all tests under the three different normal stress and under the two heating cycles (40°C and 55°C), difference 2 and 4 are always positive values in stage 1, while differences 1 and 3 are
always negative in stage 1. Furthermore, the maximum difference between the thermocouples’
temperatures can be found between T2 and T3, which ranges around 17°C under 40°C heating
cycles under all normal stress, while it is around and 20°C under 55°C heating cycles for all tests.
In addition, these results are less than that of the 70°C heating cycle, which had a range of 19°C
to 22°C difference between T2 and T3. Therefore, this difference increases with increased heating
temperatures.
At the beginning of constant heating at the target temperature stage, the differences decrease
rapidly to a point where the differences between the four temperatures and the average become
very close to the x-axis. The steady-state or homogeneity of temperatures is reached at this point
when the differences reach to this level (i.e., very close to the x-axis) with a constant value or with
a few fluctuations ±2°C. There is no significant difference at this stage between the results under
the 40°C or 55°C heating cycles. This is because, at this stage, what plays a significant role in how
much values the differences are from the x-axis are the thermal conductivity of the clay material
and the temperature at the laboratory where the experiments were held.
In cooling stage, the differences profile shown in Figure 6.7-b through Figure 6.12-b show that
differences 1 and 3 are positive, and differences 2 and 4 are negatives for all tests. Furthermore,
the highest difference between the thermocouples (i.e., between T4 and T3) ranges between 4°C
and 6°C under 40°C heating cycle and between 4°C and 8°C during the cooling stage, which is
less than that of the low P.I. soil (between 10°C and 14°C). Therefore, just like the difference
between T2 and T3 in the heating stage, the difference between T4 and T3 in the cooling stage
increases with increasing the temperature of the heating-cooling cycle. Once again, as cooling
proceeded, the differences between temperatures and the average temperature decrease until
reaching or coming close to the room temperature of 20°C ±0.5°C.
Figure 6.7. Under 10psi and one cycle of temperature up to 40°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

Figure 6.8. Under 10psi and one cycle of temperature up to 55°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.
Figure 6.9. Under 16psi and one cycle of temperature up to 40°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

Figure 6.10. Under 16psi and one cycle of temperature up to 55°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.
Figure 6.11. Under 21.8psi and one cycle of temperature up to 40°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

Figure 6.12. Under 21.8psi and one cycle of temperature up to 55°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

6.3. Vertical Displacement Response Under the Heating-Cooling Cycle

Figure 6.13-a through Figure 6.18-a show the average temperature of the four thermocouples under normal stresses of 10psi, 16psi, and 21.8psi. Figure 6.13-b through Figure 6.18-b show the calibrated volume change with the uncalibrated (with heating) of the three tests. Same as the low P.I. clay under a heating cycle up to 70°C, thermal loading was applied immediately after consolidation for the tests that are subjected to a heating-cooling cycle. In addition, the temperature
was increased from the room temperature (usually from 20-23°C) to 40°C in around 90 minutes, as shown in Figure 6.13b, Figure 6.14-b, and Figure 6.15-b. For the tests that were subjected to a heating cycle up to 55°C, it took around 140 minutes to reach the target temperature. The cooling stage took around 400 min and 550 min for the 40°C and 55°C heating cycles, respectively. Same as the low P.I. under 70°C heating cycle, condition 2 was satisfied after around 300 to 400 minutes after reaching the target temperatures (40°C and 55°C) for all tests under the 40°C and 55°C heating cycles as shown in Figure 6.13-b through Figure 6.18-b, stage 2 for all tests was kept for 650-1000 min. Figure 6.13-b through Figure 6.18-b show the no further change in volume (i.e., vertical displacement) before cooling. In addition, under cooling in phase 3, there is a volume change but less than that of the 70°C heating cycle. Before starting shearing, no further volume change occurred.
Figure 6.13. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 10psi and one cycle of heating up to 40°C.

Figure 6.14. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 16psi and one cycle of heating up to 40°C.
Figure 6.15. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 21.8psi and one cycle of heating up to 40°C.

Figure 6.16. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 10psi and one cycle of heating up to 55°C.
Figure 6.17. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 16psi and one cycle of heating up to 55°C.

Figure 6.18. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 21.8psi and one cycle of heating up to 55°C.

6.4. Results and Discussions

Figure 6.19 and Figure 6.20 present the horizontal displacements versus the vertical displacements during shearing under the three different normal stresses of medium P.I. clay-concrete interface. It also shows the results for the tests that were subjected to one heating-cooling cycle up to 40°C and 55°C, respectively, and those without heating. The maximum values of vertical displacements for the tests that were subjected to a heating-cooling cycle up to 40°C ranged between 0.023 in and 0.03 and between 0.023 and 0.027 under a heating cycle up to 55°C in under the three normal
stresses. As mentioned previously, it ranged between 0.025 in and 0.031 in for the tests that were not subjected to a heating-cooling cycle.

Figure 6.19. Vertical displacement vs. horizontal displacement behavior under shearing without heating and with a heating cycle up to 40°C.

Figure 6.20. Vertical displacement vs. horizontal displacement behavior under shearing without heating and with a heating cycle up to 55°C.

Figure 6.21 and Figure 6.22 depict the shear strength versus horizontal displacement under the four different normal stresses of the low P.I. clay-concrete interface with and without one heating-
cooling cycle up to 40°C and 50°C, respectively. The maximum shear strength is 5.02 psi, 7.01 psi, and 11.05 psi under 10 psi, 16 psi, and 21.8 psi, respectively, for the tests that were subjected to a heating-cooling cycle up to 40°C. Furthermore, displacement at maximum stresses ranged between 0.17 in to 0.258 in. Furthermore, the maximum shear strength is 5.29 psi, 7.2 psi, and 11.5 psi under 10 psi, 16 psi, and 21.8 psi, respectively, for the tests that were subjected to a heating-cooling cycle up to 55°C. Moreover, displacement at maximum stresses ranged between 0.2 in to 0.24 in. The maximum shear strengths for the tests without heating-cooling cycle are 4.31 psi, 6.65 psi, and 10.4 psi under 10 psi, 16 psi, and 21.8 psi, respectively, with a range of displacements between 0.17 in and 0.22 in. Table 6.1 and Table 6.2 show detailed results of the large-size direct shear tests without heating with comparison to those with heating cycle up to 40°C and 50°C, respectively.

![shear stress with horizontal displacement](image)

Figure 6.21. Shear stress with horizontal displacement of clay concrete interface without heating and with heating cycle up to 40°C.
Figure 6.22. Shear stress with horizontal displacement of clay concrete interface without heating and with heating cycle up to 55°C.

Table 6.1. Interface shear strength results of low P.I. clay-concrete with 40°C heating cycle.

<table>
<thead>
<tr>
<th>Heating Condition</th>
<th>Normal stress (psi)</th>
<th>Maximum shear strength (psi)</th>
<th>Displacement at maximum shear strength (in)</th>
<th>Residual Shear strength (psi)</th>
<th>% Increase in shear strength at the peak after heating</th>
<th>% Increase in shear strength at residual after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Heating</td>
<td>10</td>
<td>5.02</td>
<td>0.17</td>
<td>4.34</td>
<td>16.5</td>
<td>3.8</td>
</tr>
<tr>
<td>Without Heating</td>
<td>10</td>
<td>4.31</td>
<td>0.22</td>
<td>4.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>16</td>
<td>7.01</td>
<td>0.215</td>
<td>6.26</td>
<td>5.4</td>
<td>8.7</td>
</tr>
<tr>
<td>Without Heating</td>
<td>16</td>
<td>6.65</td>
<td>0.175</td>
<td>5.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>21.8</td>
<td>11.05</td>
<td>0.258</td>
<td>10.04</td>
<td>6.3</td>
<td>4.9</td>
</tr>
<tr>
<td>Without Heating</td>
<td>218</td>
<td>10.4</td>
<td>0.187</td>
<td>9.57</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 6.2. Interface shear strength results of low P.I. clay-concrete with 55°C heating cycle

<table>
<thead>
<tr>
<th>Heating Condition</th>
<th>Normal stress (psi)</th>
<th>Maximum shear strength (psi)</th>
<th>Displacement at maximum shear strength (in)</th>
<th>Residual Shear strength (psi)</th>
<th>% Increase in shear strength at the peak after heating</th>
<th>% Increase in shear strength at residual after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Heating</td>
<td>10</td>
<td>5.29</td>
<td>0.201</td>
<td>4.5</td>
<td>22.7</td>
<td>7.7</td>
</tr>
<tr>
<td>Without Heating</td>
<td>10</td>
<td>4.31</td>
<td>0.22</td>
<td>4.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>16</td>
<td>7.2</td>
<td>0.24</td>
<td>6.5</td>
<td>8.3</td>
<td>12.8</td>
</tr>
<tr>
<td>Without Heating</td>
<td>16</td>
<td>6.65</td>
<td>0.175</td>
<td>5.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Heating</td>
<td>21.8</td>
<td>11.5</td>
<td>0.217</td>
<td>10.3</td>
<td>10.6</td>
<td>7.6</td>
</tr>
<tr>
<td>Without Heating</td>
<td>218</td>
<td>10.4</td>
<td>0.187</td>
<td>9.57</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The peak shear strengths of all tests on the shear strength-normal stress plane, adopting the Mohr-coulomb failure criteria, were plotted as shown in Figure 6.23 and Figure 6.24 with 40°C and 50°C heating cycles, respectively. In addition, the residual shear strengths of all tests on the shear strength-normal stress plane, adopting the Mohr-coulomb failure criteria, were plotted as shown in Figure 6.25 and Figure 6.26 with 40°C and 50°C heating cycles, respectively.

Following Figure 6.13-b through Figure 6.18-b volume expansion is observed at the beginning of heating followed by a volume contraction just before reaching the target temperatures of 40°C and 55°C. In stage 2 of heating, excessive contraction is observed until reaching stabilization of volume. A further excessive contraction is observed during the cooling phase under all normal stresses. This behavior is well known for the normally consolidated soil (Abuel-Naga et al., 2006; Laloui, 2001; Towhata et al., 1993). The volumetric strain due to the heating-cooling cycle up to
40°C was 0.38%, 0.35%, and 0.365% under 10psi, 16psi, and 21.8psi, respectively, as shown in Figure 6.27.

The volumetric strain due to the heating-cooling cycle up to 55°C was 0.47%, 0.47%, and 0.58% under 10psi, 16psi, and 21.8psi, respectively, as shown in Figure 6.28. At heating stage, all tests...
have a positive thermally induced volumetric strain (TIVS) (contraction). At stage 2 of heating, all tests show an increase in TIVS (contraction) under 40 and 55 heating cycle. In the cooling stage, small contraction is observed for 10psi (40°C and 55°C) and 16psi (40°C), while higher TIVS is observed for the rest of the tests, as shown in Figure 6.27 and Figure 6.28. The same observations with regards to TIVS trend were found for the normally consolidated clay by Towhata et al. (1994), Baldi et al. (1988), Graham et al. (2000), Burghignoli et al. (1999), Abuel-Naga et al. (2006-a, 2007), and other researches. When comparing the final TIVS after the heating cycle with respect to the normal stresses (4.35psi, 10psi, 16psi, and 21.8psi), it is found that TIVS does not change with increasing the normal stress (Abuel-Naga et al., 2007), but with some small differences due to preparation conditions (Yazadani et al., 2019).

The average TIVS under 40°C, 55°C, and 70°C are around 0.365%, 0.507%, and 0.65%, respectively. This is clearly showing that TIVS increases with increasing thermal load for the reasons described in chapter 2.

Figure 6.27. Profile of thermally induced volumetric strain with temperature difference for the low P.I. clay-concrete interface under 40°C heating cycle.

Figure 6.28. Profile of thermally induced volumetric strain with temperature difference for the low P.I. clay-concrete interface under 55°C heating cycle.
The same observations found for the low P.I. vertical displacement results can be stated based on the results shown in Figure 6.19 and Figure 6.20 when it comes to the relationship between the increase in vertical displacement and increasing vertical stress. That also applies to the higher rate of vertical displacement at the first 0.2 inches of horizontal displacement as compared to the next 0.4 inches. In the same figures, the results of the tests that were subjected to one heating-cooling cycle up to 40°C and 50°C almost follow the same trend to those that were not subjected to heating cycle. However, higher vertical displacement was observed for the tests that were not subjected to heating cycle under all normal stresses. Furthermore, vertical displacement under shearing decreases with increasing the applied temperature under the same normal stresses, as shown in Figure 6.29. Furthermore, it is noticeable that under 40°C and 55°C heating cycles, the vertical displacement starts to fluctuate (increasing then decreasing again) after around 0.4 inches, while it was not observed under 70°C heating cycle and without heating. After heating-cooling cycle up to 40°C, the maximum vertical displacement was reduced by 7.0%, 7.8%, and 4.8%, and for the 55°C heating cycle, it was reduced by 6.0%, 7.9%, and 12.7%, under 10psi, 16psi, and 21.8psi, respectively. The reason behind this volumetric observation under shearing is because the material was further consolidated due to thermal loading, which reduced the initial void ratio (Di Donna et al., 2015).
Figure 6.29. Vertical displacement vs. horizontal displacement behavior under shearing without heating and with heating cycles up to 40°C, 55°C, and 70°C.

The results of shear strength versus horizontal displacement curves for the clay-concrete interface with and without heating-cooling cycle (Figure 6.21 and Figure 6.22) clearly demonstrate that, in all cases, the interface shear strength of specimens subjected to one heating-cooling cycle up to 40°C and 55°C is higher than those without heating. However, the increase under the 40°C heating cycle is less than that of the 55°C heating cycle, which is less than the 70°C heating cycle, as shown in Figure 6.30. The reason behind the increase in shear strength with increasing thermal load (i.e., temperature difference) is that under higher temperatures, higher pore water pressure is generated and dissipated. Therefore, higher thermally induced overconsolidation (Di Donna et al., 2015), which requires a higher load for shearing the material under the same normal stress. Furthermore, the increases in shear strength at peak under 10psi and 40°C (16.5%) and 55°C (22.7%) are much higher than those under 16psi and 21.8psi. Where the increase in peak shear strength is 5.4%, and 8.3% under 16psi, and 6.3% and 10.6% under 21.8psi under a heating cycle of 40°C and 55°C, respectively. In addition, the increase in residual shear strengths under 16psi and 40°C (8.7%) and 55°C (12.8%) are higher than those under 10psi and 21.8psi. Where, the
increase in residual shear strength is 3.8% and 7.7% under 10psi, and 49% and 7.6% under 21.8psi, under 40°C and 55°C heating cycles, respectively.

Figure 6.30. Shear stress with horizontal displacement of clay concrete interface without heating and with heating cycles up to 40°C, 55°C, and 70°C.

Figure 6.31 shows the peak and residuals shear strength of the low P.I. clay-concrete interface under 10psi, 16psi, and 21.8psi under one heating cycles up to 40°C (ΔT=20), 55°C (ΔT=35), 70°C (ΔT=50). As shown, the relationship between the shear strength and the temperature difference is almost always proportional under all normal stresses at the peak and residuals. The slopes of the best fit lines of the peak shear strengths are 0.0266psi/°C, 0.023psi/°C, and 0.0188psi/°C under 21.8psi, 16psi, and 10psi, respectively. The slopes of the best fit lines of the residual shear strengths are 0.031psi/°C, 0.0238psi/°C, and 0.00917psi/°C under 21.8psi, 16psi, and 10psi, respectively. Therefore, a higher peak and residual shear strength increase rate is observed under higher normal stresses. Furthermore, when comparing the residual increase rate with the peak increase rate, it is found to be a higher rate under 21.8psi, the same under 16psi, and a lower rate under 10psi.
Figure 6.31. Shear strength with temperature difference of low P.I. clay-concrete interface under one heating cycles.

At the peak shear strength, the interface friction angle, $\delta$, increased by 6.6%, from $25.9^\circ$ to $24.3^\circ$ under 40°C heating cycle (Figure 6.23); while $\delta$, increased by 9.9%, from $26.7^\circ$ to $24.3^\circ$, under 55°C heating cycle (Figure 6.24). Under 40°C heating cycle, the interface friction angle, at the high displacement (residuals), $\delta$, increased by 4.9% and by 8.0% under the 55°C heating cycle from the without heating interface friction angle, $\delta$, of a $22.4^\circ$ value. According to Di Donna (2014), Di Donna and Laloui (2013), Yavari et al. (2016), and Yazdani et al. (2019), the reason behind the increase in shear strength after the heating-cooling cycle is the thermal consolidation or thermal solidification. The same explanation is believed to be the reason behind the increase in shear strengths and shear strength parameters in this study. Figure 6.32 (at peak) and 6.33 (at the residuals) show the Mohr’s column results of the interface friction angle, $\delta$, under 20°C (without heating), 40°C, 55°C, and 70°C heating cycles. As shown, $\delta$, increases with increasing the target temperature in the heating-cooling cycles for both, the peak and residuals. However, it clearly demonstrates that the increase at the peak is higher than that at the residuals.
Figure 6.32. Effect of heating under all heating cycles on the peak shear strength with normal stresses at failure envelope.

Figure 6.33. Effect of heating under all heating cycles on the residual shear strength with normal stresses at failure envelope.
CHAPTER 7.
LOW P.I. SOIL UNDER MULTIPLE HEATING-COOLING CYCLES

The low P.I clay was used for testing clay-concrete interface under multiple heating-cooling cycles. Four tests were performed under the same normal stress of 16psi and several heating-cooling cycles. These four tests were performed under four heating cycles up to 70°C, four heating cycles up to 55°C, four heating cycles up to 40°C, and nine heating cycles up to 55°C. The preparation procedure followed in the previous section using the low P.I. Clay-concrete interface was exactly followed for the multiple cycles testing.

7.1. Consolidation

Figure 7.1, Figure 7.2, Figure 7.3, and Figure 7.4 show the logarithmic plot of the consolidation for the tests that were subjected to four heating-cooling cycles up to 40°C, 55°C, 70°C, and nine heating-cooling cycles up to 55°C under 16psi, respectively. Figure 7.1 to Figure 7.4 also show that all tests reached the end of primary consolidation before either heating or shearing starts with a value of around 0.13in.

Figure 7.1. Consolidation of low P.I. clay under 16psi normal stress and four heating cycles up to 40°C.

Figure 7.2. Consolidation of low P.I. clay under 16psi normal stress and four heating cycles up to 55°C.
Figure 7.3. Consolidation of low P.I. clay under 16psi normal stress and four heating cycles up to 70°C.

Figure 7.4. Consolidation of low P.I. clay under 16psi normal stress and nine heating cycles up to 55°C.

7.2. Thermal Loading

Figure 7.5-a, Figure 7.6-a, Figure 7.7-a, and Figure 7.8-a show the temperatures of the four thermocouples of the four tests under four heating cycles up to 40°C, 55°C, 70°C, and nine heating cycles up to 55°C under 16psi, respectively. Figure 7.5-b, Figure 7.6-b, Figure 7.7-b, and Figure 7.8-b show the difference between the temperatures of the four thermocouples and their average temperature of the four tests under four heating cycles up to 40°C, 55°C, 70°C, and nine heating cycles up to 55°C under 16psi, respectively. The temperature stabilization was reached after the first thermocouple reached the target temperatures within around 40 min, 60 min, and 80 min under the targeted temperature of 40 °C, 55°C, and 70°C, respectively, for all cycles. Therefore, clearly, the time needed for temperature stabilization increases with increasing the target temperature. It can be seen in Figure 7.5-b, Figure 7.6-b, Figure 7.7-b, and Figure 7.8-b that for all tests under all heating cycles under all temperatures (40 °C, 55°C, and 70°C), difference 2 and 4 are always positive values in stage 1, while differences 1 and 3 are always negative in stage 1. Furthermore, the maximum difference between the thermocouples’ temperatures can be found between T2 and
T3, which ranges around 16°C under 40°C heating cycles, 19°C under 55°C heating cycles, and around 21°C under 70°C heating cycles for the majority of tests. Therefore, this difference increases with increased heating temperature. However, some cycles have different shapes and/or values than the majority of cycles, such as the heating stage of the first cycle under 40°C, the heating stage of the fourth cycle under 55°C, the heating stage of the first cycle under 70°C, and the fifth cycle under the nine cycles heating test under 55°C. The reason is, as mentioned previously, the uncontrolled temperature at the laboratory where the experiments were held. At the beginning of constant heating at the target temperature stage, the differences decrease rapidly to a point where the differences between the four temperatures and the average become very close to the x-axis. The steady-state or homogeneity of temperatures is reached with a constant value of ±1°C to ±2°C. There is no significant difference at this stage between the results under the 40°C, 55°C, or 70°C heating cycles, but a little difference is observed for the cycles at 70°C.

In the cooling stage, the differences profile shown in Figure 7.5-b, Figure 7.6-b, Figure 7.7-b, and Figure 7.8-b show that differences 1 and 3 are positive, and differences 2 and 4 are negatives for all tests. Furthermore, the highest difference between the thermocouples (i.e., between T4 and T3) ranges between 4°C and 6°C under 40°C heating cycles and between 4°C and 8°C under 55°C heating cycles, and between 8°C and 12°C under 70 heating cycles. These observations are very much like that of the previous observation of the low P.I. low clay-concrete interface.

In the cooling stage, the differences profile shown in Figure 7.5-b, Figure 7.6-b, Figure 7.7-b, and Figure 7.8-b show that differences 1 and 3 are positive, and differences 2 and 4 are negatives for all tests. Also, the highest difference between the thermocouples (i.e., between T4 and T3) ranges between 4°C and 6°C under 40°C heating cycles and between 4°C and 8°C under 55°C heating
cycles, and between 8°C - 12°C under 70°C heating cycles. These observations very much like that of the previous observation of the low P.I. low clay-concrete interface.

Figure 7.5. Under 16psi and four cycles of temperature up to 40°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.
Figure 7.6. Under 16psi and four cycles of temperature up to 55°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.
Figure 7.7. Under 16psi and nine cycles of temperature up to 55°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.
Figure 7.8. Under 16psi and four cycles of temperature up to 70°C for low P.I. clay. a. Temperature profile with time for the four thermocouples, b. Profile of the differences between thermocouples temperatures and the average temperature.

7.3. Vertical Displacement Response Under the Heating-Cooling Cycles

Figure 7.9-a, Figure 7.10-a, Figure 7.11-a, and Figure 7.12-a show the average temperature of the three phases under 40°C (four cycles), 55°C (four cycles), 55°C (nine cycles), and 70°C (four cycles), respectively, under normal stresses of 16psi. Figure 7.9-b, Figure 7.10-b, Figure 7.11-b, and Figure 7.12-b show the calibrated volume change with the uncalibrated (with heating) of the four tests. The temperature was increased from the room temperature (usually from 20-23 °C) to 40 °C, 55°C, and 70°C in around 90 minutes, 140 minutes, and 180 minutes as shown in Figure
The cooling stage took around 400 minutes, 550 minutes, and 700 minutes for the 40°C, 55°C, and 70°C heating cycles, respectively. Condition 2 was satisfied after around 450 minutes, 350 minutes, and 300 minutes, after reaching the target temperatures (40°C, 55°C, 70°C) for the first cycle under 40°C, 55°C, and 70°C heating cycles, respectively, as shown in Figure 7.9-b, Figure 7.10-b, Figure 7.11-b, and Figure 7.12-b. However, condition 2 needed less time in the subsequent cycles, which is around 250 minutes, 200 minutes, and 150 minutes under the 40°C, 55°C, and 70°C heating cycles, respectively.

Figure 7.9. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 16psi and four cycles of heating up to 40°C.
Stage 2 for all tests was kept for 1000 to 1500 minutes. Figure 7.9-b, Figure 7.10-b, Figure 7.11-b, and Figure 7.12-b show there was no further change in volume (i.e. vertical displacement) before cooling. Furthermore, before starting shearing, no further volume change occurred.

Figure 7.10. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 16psi and four cycles of heating up to 55°C.
Figure 7.11. a. Average temperature, b. Vertical displacement with time of low P.I. clay for the calibrated and uncalibrated values under normal stress of 16psi and nine cycles of heating up to 55°C.
Results and Discussion

Figure 7.13 presents the horizontal displacements versus the vertical displacements during shearing under all different heating conditions under 16psi of low P.I. clay-concrete interface. The maximum values of vertical displacements for the tests that were subjected to heating-cooling cycles up to 40°C, 55°C, and 70°C ranged between 0.0236in and 0.025in, between 0.023in and 0.032in and between 0.0226in and 0.027in, respectively, under 16psi. Table 7.1 shows the maximum vertical displacement for all tests shown in Figure 7.13.
Figure 7.13. Vertical displacement vs. horizontal displacement behavior of low P.I. clay-concrete interface under shearing for all tests under 16psi.

Table 7.1. Details of vertical displacement under shearing for all tests under 16psi of the low P.I. clay-concrete interface.

<table>
<thead>
<tr>
<th>Normal Stress (psi)</th>
<th>Target Temperature (°C)</th>
<th>Number of Heating Cycles</th>
<th>Maximum Vertical Displacement Under Shearing (in)</th>
<th>% Increase or Decrease after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>Without Heating</td>
<td>0</td>
<td>0.0255</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td>40°C</td>
<td>1</td>
<td>0.02355</td>
<td>-7.6</td>
</tr>
<tr>
<td></td>
<td>40°C</td>
<td>4</td>
<td>0.0252</td>
<td>-1.2</td>
</tr>
<tr>
<td></td>
<td>55°C</td>
<td>1</td>
<td>0.0233</td>
<td>-8.6</td>
</tr>
<tr>
<td></td>
<td>55°C</td>
<td>4</td>
<td>0.0323</td>
<td>26.7</td>
</tr>
<tr>
<td></td>
<td>55°C</td>
<td>9</td>
<td>0.0286</td>
<td>12.2</td>
</tr>
<tr>
<td></td>
<td>70°C</td>
<td>1</td>
<td>0.0226</td>
<td>-11.4</td>
</tr>
<tr>
<td></td>
<td>70°C</td>
<td>4</td>
<td>0.0269</td>
<td>5.5</td>
</tr>
</tbody>
</table>

Figure 7.14 depicts the shear strength versus horizontal displacement under all different heating conditions (i.e., without heating, one heating cycle - 40°C, four heating cycles – 40°C, one heating cycle – 55°C, four heating cycles – 55°C, nine heating cycles – 55°C, one heating cycle – 70°C, and four heating cycles – 70°C) under 16psi of the low P.I. clay-concrete interface. The maximum shear strength ranges between 6.65psi to 8.65psi, while the residual shear strength ranges between
5.76psi to 8.25psi. In addition, displacement at maximum strengths ranged between 0.175in to 0.325in. Table 7.2 show detailed results of the large-size direct shear tests of the low P.I. clay-concrete interface for all tests performed under 16psi.

![Figure 7.14. Shear stress with horizontal displacement of low P.I. clay concrete interface for all tests under 16psi.](image)

Table 7.2. Interface shear strength results of low P.I. clay-concrete under 16psi and all different heating conditions.

<table>
<thead>
<tr>
<th>Target Temperature (°C)</th>
<th>Number of Heating Cycles</th>
<th>Maximum shear strength (psi)</th>
<th>Displacement at maximum shear strength (in)</th>
<th>Residual shear strength (psi)</th>
<th>% Increase in shear strength at peak after heating</th>
<th>% Increase in shear strength at residual after heating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without Heating</td>
<td>0</td>
<td>6.65</td>
<td>0.175</td>
<td>5.76</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td>40°C</td>
<td>1</td>
<td>7.01</td>
<td>0.215</td>
<td>6.26</td>
<td>5.4</td>
<td>8.7</td>
</tr>
<tr>
<td>40°C</td>
<td>4</td>
<td>7.58</td>
<td>0.231</td>
<td>6.68</td>
<td>14.0</td>
<td>16.0</td>
</tr>
<tr>
<td>55°C</td>
<td>1</td>
<td>7.23</td>
<td>0.243</td>
<td>6.49</td>
<td>8.7</td>
<td>12.7</td>
</tr>
<tr>
<td>55°C</td>
<td>4</td>
<td>7.8</td>
<td>0.247</td>
<td>7.15</td>
<td>17.3</td>
<td>24.1</td>
</tr>
<tr>
<td>55°C</td>
<td>9</td>
<td>8.65</td>
<td>0.216</td>
<td>7.84</td>
<td>30.1</td>
<td>36.1</td>
</tr>
<tr>
<td>70°C</td>
<td>1</td>
<td>7.86</td>
<td>0.183</td>
<td>7.0</td>
<td>18.2</td>
<td>21.5</td>
</tr>
<tr>
<td>70°C</td>
<td>4</td>
<td>8.44</td>
<td>0.325</td>
<td>8.25</td>
<td>26.9</td>
<td>43.2</td>
</tr>
</tbody>
</table>
Following Figure 7.9-b, Figure 7.10-b, Figure 7.11-b, and Figure 7.12-b, a volume contraction is observed at the beginning of the first, third, and fourth heating cycles of 40°C and a small volume expansion at the beginning of the second heating cycle followed by a stabilization of volume until reaching the target temperature. Under 55°C and 70°C heating cycles for both tests (i.e., four and nine cycles) have a volume contraction at the beginning of the first heating cycle, but small expansion for the rest of cycles followed by volume contraction. This behavior is well known for the normally consolidated soil (Abuel-Naga et al., 2006; Laloui, 2001; Towhata et al., 1993). Figure 7.15, and Figure 7.17 show the volumetric strain of the low P.I. clay-concrete interface under 40°C, 55°C, and 70°C, respectively, under all cycles. The volumetric strain due to heating-cooling cycles up to 40°C was 0.46% (one cycle), 0.52% (four cycles), for the 55°C heating cycles, it is about 0.47% (one cycle), 0.485% (four cycles), and 1.12% (nine cycles), and for the 70°C heating cycles, it is about 0.58% (one cycle) and 1.27% (four cycles).

At heating stage for 40°C, 55°C, and 70°C heating cycles under 16psi, all tests have a positive thermally induced volumetric strain (TIVS) (contraction) in the first cycles, but almost no change in TIVS at the heating stage of the subsequent cycles as shown in Figure 7.15, and Figure 7.17. At stage 2 of heating, all tests show an increase in TIVS in the first cycle, while a much smaller increase in TIVS is shown in the subsequent cycles. In the cooling stage, small contraction is observed for all tests in the first cycle and a much smaller change in TIVS in the subsequent cycles. Therefore, when comparing the change of TIVS between the fourth or ninth cycle to the first cycle, it is found that the change in TIVS in the first cycle is greater than the change in TIVS between the last cycle and the first cycle for all temperatures. This observation was also found by Yazadani et al. (2019) after 10 and 20 heating cycles. This observation is clearly shown in Figure 7.18, which shows the TIVS at the end of each cycle with the number of heating cycles. The profile of TIVS
(Figure 7.18) shows that after the first cycle, TIVS is either almost constant, slightly increasing, or fluctuating.

Figure 7.15. Profile of thermally induced volumetric strain with temperature difference for the low P.I. clay-concrete interface under 40°C, between 1 and 4 heating cycles.

Figure 7.16. Profile of thermally induced volumetric strain with temperature difference for the low P.I. clay-concrete interface under 55°C, between 1, 4, and 9 heating cycles.

Figure 7.17. Profile of thermally induced volumetric strain with temperature difference for the low P.I. clay-concrete interface under 70°C, between 1 and 4 heating cycles.

Figure 7.18. Thermally induced volumetric strain at the end of each cycle with the number of heating-cooling cycles under 16psi of the low P.I. clay-concrete interface.
The same observations were found for the low P.I. vertical displacement results and can be stated based on the results shown in Figure 7.13 when it comes to the relationship between the increase in vertical displacement and increasing vertical stress. That also applies to the higher rate of vertical displacement at the first 0.2 inches of horizontal displacement as compared to the next 0.4 inches. When comparing the vertical displacement of the tests that were not subjected to heating cycles to others, higher vertical displacement was observed for the tests that were subjected to four and nine 55°C heating cycles and four 70°C heating cycles. Other tests have less vertical displacement than that of the without heating condition test. After heating-cooling cycle up to 40°C, the maximum vertical displacement was reduced by 7.6% (one cycle) and 1.2% (four cycles), for the 55°C heating cycles, the change in vertical displacement is a reduction of 8.6% (one cycle), an increase of 26.7% (four cycles), and an increase of 12.2% (nine cycles). Under 70°C heating cycles, a reduction of 11.4% (one cycle) and an increase of 5.5% (four cycles) were observed in vertical displacement under shearing.

The results of shear strength versus horizontal displacement curves for the clay-concrete interface with and without heating-cooling cycles (Figure 7.14) demonstrate that, in all cases, the interface shear strength increases with heating and the number of heating cycles. Through Table 7.2, and when comparing different temperatures with the same number of cycles, an increase in shear strength can be found. For example, under one cycle, the peak shear strength and residual shear strengths increased by 5.4% and 8.7% (40°C), 8.7% and 12.7% (55°C), and 18.2% and 21.5% (70°C), respectively. In addition, under four cycles, the peak shear strength and residual shear strengths increased by 14.0% and 16.0% (40°C), 17.3% and 24.1% (55°C), and 26.9% and 43.2% (70°C), respectively. Furthermore, under nine cycles of 55°C heating cycles, it has the highest peak shear strength of 8.65psi with an increase of 30.1% and the second highest residual strength
of 7.84 psi with an increase of 36.1%. When comparing the shear strength results after one cycle to the increase in shear strength after multiple cycles, it is found to be not proportional to the increase in TIVS. Therefore, the explanation of this higher increase in TIVS might be attributed to the change in physico-chemical structure interaction, as proposed by Campanella and Mitchell (1968). Yazadani et al. (2019) found a 16% and 10% increase in medium P.I. (P.I.=20) clay-concrete interface shear strength after 10 heating cycles (24°C - 34°C - 24°C) under 225kPa and 300kPa normal stresses, respectively. However, the increase in shear strength at 34°C was 6% and 9% higher than at 24°C (Yazadani et al., 2018).

Figure 7.19 shows the shear strength with a number of heating cycles under 16 psi for all heating cycles (40°C, 55°C, and 70°C) at peak and residuals. As shown, the peak and residual shear strength increase with increasing the number of cycles. However, the increase rate after the first cycle is clearly much higher than the increase rate after four and nine cycles at peak and residuals. Furthermore, the increase rate after nine cycles is almost the same compared to after four cycles, at which the shear strength (peak) after nine cycles (55°C) is higher than the shear strength (peak) after four cycles (70°C).
Figure 7.19. Shear strength with number of cycles of the low P.I. clay-concrete interface under 16psi.
CHAPTER 8
CONCLUSIONS AND RECOMMENDATIONS

8.1. Conclusions
The shear strength characteristics of clay-concrete interface were evaluated under different heating-cooling cycles conditions. The tested clays are low plasticity index, P.I., clay (PI=12), medium plasticity index, P.I., clay (PI=30), and high plasticity index, P.I., clay (PI=60). A large-size direct shear test device (dimensions = 12 in × 12 in × 8 in) was modified and used in this study to evaluate the clay-concrete interface shear strength with and without one heating-cooling cycles for potential application to increase the pile capacity through heating; also to study the potential geothermal pile applications. All clay specimens of each clay type were prepared under the same conditions with respect to unit weight, moisture content, consolidation, and heating-cooling cycle. The clay specimens were first consolidated under different normal stresses of 10psi, 16psi, and 21.8psi, (and 4.35psi for the low P.I. clay) and then subjected to shearing with and without heating. Heating consists of phase 1 (i.e., heating phase), phase 2 (i.e., heating at the target temperature), and phase 3 (cooling phase). The vertical displacement under heating condition were calibrated to provide correct results for the thermally induced volumetric strains. All three types of clay soils were tested under one heating-cooling cycle of (20° - 70° - 20°) C temperatures. Only the low P.I. clay soil was also tested under two different heating-cooling cycles of (20° - 55° - 20°) C temperatures and (20° - 40° - 20°) C temperatures. In addition, it was tested under 16psi and four number of heating-cooling cycles of (20° - 70° - 20°) C, (20° - 55° - 20°) C, and (20° - 40° - 20°) C temperatures. Furthermore, the low P.I. clay soil was also tested under nine number of heating-cooling cycles of (20° - 55° - 20°) C temperatures under 16psi. The following sections summarize the findings and conclusions of each of different chapters with different heating-loading conditions.
8.1.1. Chapter 3 (Low P.I. Under One Heating-Cooling Cycle at 70°C)

The increase in peak shear strength is 29.3%, 21.1%, 18.2%, and 12.5% under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. In addition, the increase in residual shear strengths are 36.7%, 11.0%, 21.35%, and 17.15% under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. After heating-cooling cycle, the maximum vertical displacement was reduced by 30.0%, 24.4%, 11.3%, and 24.2% under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. At the peak shear strength, the interface friction angle, \( \delta \), increased by 13.6%, from 24.3° to 27.6°; while \( \delta \) increased by 15.6%, from 22.4° to 25.9°, at the residual strength.

A volume reduction is observed until reaching the target temperature followed by fluctuations but tending to have a constant volume response during the constant high temperature. An excessive contraction is observed during the cooling phase under all normal stresses. This behavior is well known for the normally consolidated soil (Abuel-Naga et al., 2006; Laloui, 2001; Towhata et al., 1993). The volumetric strain due to heating-cooling cycle was 0.8%, 0.5%, 0.6%, and 0.7% under 4.35psi, 10psi, 16psi, and 21.8psi, respectively. In addition, when comparing the final TIVS after the heating cycle with respect to the normal stresses (4.35psi, 10psi, 16psi, and 21.8psi), it is found that TIVS is independent of normal stress. This is because TIVS is controlled by the amount and rate of dissipation of pore water pressure, which depends mainly on thermal load (applied temperature) and material properties (hydraulic conductivity, thermal conductivity, plasticity index, …, etc.).

8.1.2. Chapter 4 (Medium P.I. Under One Heating-Cooling Cycle at 70°C)

The increase in peak shear strength is 29.2%, 27.3%, and 26.6%, under 10psi, 16psi, and 21.8psi, respectively. Furthermore, the increase in residual shear strengths are 35.1%, 37.0%, and 35.2% under 10psi, 16psi, and 21.8psi, respectively. After the heating-cooling cycle, the maximum
vertical displacement was reduced by 13.2%, 43.8%, 54.0% under 10psi, 16psi, and 21.8psi, respectively. This reduction (contraction) in vertical displacement is attributed to the thermally induced overconsolidation effect and reduction in void ratio before shearing (Di Donna, 2015). At the peak shear strength, the interface friction angle, $\delta$, increased by 25.0%, from 17.0° to 13.6°; while $\delta$, increased by 35.5%, from 14.9° to 11.0°, at the residual strength. According to Di Donna (2014), Di Donna and Laloui (2013), Yavari et al. (2016), and Yazdani et al. (2019), the reason behind the increase in shear strength after the heating-cooling cycle is the thermal consolidation or thermal solidification.

A volume expansion is observed at the beginning of heating followed by a volume contraction just before reaching the target temperature of 70°C. In stage 2 of heating (i.e., heating at a constant target temperature of 70°C), excessive contraction is observed until reaching stabilization of volume. A further excessive contraction is observed during the cooling phase under all normal stresses. The volumetric strain due to the heating-cooling cycle was 1.0%, 0.91%, and 0.89% under 10psi, 16psi, and 21.8psi, respectively.

8.1.3. Chapter 5 (High P.I. Under One Heating-Cooling Cycle at 70°C)

The increase in peak shear strength is 19.6%, 24.3%, and 21.6%, under 10psi, 16psi, and 21.8psi, respectively. In addition, no significant increase in the residual shear strengths was observed under the same tests. The reason behind the developed higher after the heating cycle is the increase in the overconsolidation ratio after the heating cycle (Di Donna et al., 2015; Abuel-Naga et al., 2007). After the heating-cooling cycle, the maximum vertical displacement was reduced by 85%, 87.7%, 96% under 10psi, 16psi, and 21.8psi, respectively. At the peak shear strength, the interface friction angle, $\delta$, increased by 24.1%, from 8.7° to 10.8°; while adhesion increased by 17.7% from 1.36psi to 1.6psi at peak.
A volume expansion (i.e., reduction in vertical displacement) is observed at the beginning of heating before reaching the target temperature of 70°C. At the heating stage, all tests have a negative TIVS (expansion) up to around -0.5%, except for the tests under 10psi where it did not show the same behavior because it was not heated to 70°C directly as described earlier. This observation is because, for overconsolidated clay, a negative pore water pressure is developed under heating, which requires the sample to absorb more water, which translates into an expansion (Abuel-Naga et al., 2006-a, 2007). At stage 2 of heating, all tests show an increase in TIVS (contraction) without reaching the stabilization of volume in this stage. A further excessive contraction is observed during the cooling phase under all normal stresses. The volumetric strain due to the heating-cooling cycle was 3.1%, 2.5%, and 3.35% under 10psi, 16psi, and 21.8psi, respectively.

When considering the relationships between the increase in shear strength of normally consolidated tests (low and medium P.I. clays) with plasticity index, it is found that the percentage increases in shear strength increase with increasing the plasticity indexes for peak and residuals. The increase in residuals is almost higher than that of the peak for all tests. However, the increase in residuals for the overconsolidated high P.I. clays are almost zero, and the increase in peak strength is almost the same as the increase in normally consolidated medium P.I. clay. This observation is believed to be because of the higher TIVS observed for the medium P.I. compared to the low P.I. clay. While the high P.I. has the highest TIVS; it does not have the highest increase in shear strength due to its stress history condition (overconsolidation).

8.1.4. Chapter 6 (Low P.I. Under One Heating-Cooling Cycle at 40°C and at 55°C)

The increases in shear strength at peak under 10psi and 40°C (16.5%) and 55°C (22.7%) are much higher than those under 16psi and 21.8psi. Where the increase in peak shear strength is 5.4%, and
8.3% under 16psi, and 6.3% and 10.6% under 21.8psi under a heating cycle of 40°C and 55°C, respectively. In addition, the increase in residual shear strengths under 16psi and 40°C (8.7%) and 55°C (12.8%) are higher than those under 10psi and 21.8psi. Where, the increase in residual shear strength is 3.8% and 7.7% under 10psi, and 49% and 7.6% under 21.8psi, under 40°C and 55°C heating cycles, respectively. After heating-cooling cycle up to 40°C, the maximum vertical displacement was reduced by 7.0%, 7.8%, and 4.8%, and for the 55°C heating cycle, it was reduced by 6.0%, 7.9%, and 12.7%, under 10psi, 16psi, and 21.8psi, respectively. At the peak shear strength, the interface friction angle, \( \delta \), increased by 6.6%, from 25.9\(^\circ\) to 24.3\(^\circ\) under 40°C heating cycle; while \( \delta \), increased by 9.9\%, from 26.7\(^\circ\) to 24.3\(^\circ\), under 55°C heating cycle. Under 40°C heating cycle, the interface friction angle, at the residuals, \( \delta \), increased by 4.9\% and by 8.0\% under the 55°C heating cycle from the without heating interface friction angle, \( \delta \), of a 22.4\(^\circ\) value.

A volume expansion is observed at the beginning of heating followed by a volume contraction just before reaching the target temperatures of 40°C and 55°C. In stage 2 of heating, excessive contraction is observed until reaching stabilization of volume. A further excessive contraction is observed during the cooling phase under all normal stresses. The volumetric strain due to the heating-cooling cycle up to 40°C was 0.38\%, 0.35\%, and 0.365\% under 10psi, 16psi, and 21.8psi, respectively. The volumetric strain due to the heating-cooling cycle up to 55°C was 0.47\%, 0.47\%, and 0.58\% under 10psi, 16psi, and 21.8psi, respectively.

8.1.5. Chapter 7 (Low P.I. Under 16psi with Multiple Heating-Cooling Cycles at 40°C, 55°C, and at 70°C)

Under four cycles, the peak shear strength and residual shear strengths increased by 14.0\% and 16.0\% (40°C), 17.3\% and 24.1\% (55°C), and 26.9\% and 43.2\% (70°C), respectively. Furthermore, under nine cycles of 55°C heating cycles, it has the highest peak shear strength of 8.65psi with an
increase of 30.1% and the second highest residual strength of 7.84psi with an increase of 36.1%. After heating-cooling cycle up to 40°C, the maximum vertical displacement was reduced by 7.6% (one cycle) and 1.2% (four cycles), for the 55°C heating cycles, the change in vertical displacement is a reduction of 8.6% (one cycle), an increase of 26.7% (four cycles), and an increase of 12.2% (nine cycles). Under 70°C heating cycles, a reduction of 11.4% (one cycle) and an increase of 5.5% (four cycles) were observed in vertical displacement under shearing.

A volume contraction is observed at the beginning of the first, third, and fourth heating cycles of 40°C and a small volume expansion at the beginning of the second heating cycle followed by a stabilization of volume until reaching the target temperature. Under 55°C and 70°C heating cycles, both tests (i.e., four and nine cycles) have a volume contraction at the beginning of the first heating cycle, but small expansion for the rest of cycles followed by volume contraction. This behavior is well known for the normally consolidated soil (Abuel-Naga et al., 2006; Laloui, 2001; Towhata et al., 1993). The volumetric strain due to heating-cooling cycles up to 40°C was 0.46% (one cycle), 0.52% (four cycles), for the 55°C heating cycles, it is about 0.47% (one cycle), 0.485% (four cycles), and 1.12% (nine cycles), and for the 70°C heating cycles, it is about 0.58% (one cycle) and 1.27% (four cycles).

8.2. Recommendations

The following six recommendations are recommended for future work that is required to provide a better understanding of the subject of the behavior of clay soils under thermal loading.

1-More excessive laboratory testing is required to examine the behavior of NC and OC clay under heating and cooling with drained and undrained conditions using triaxial tests.

2-Studying the behavior of overconsolidated clay under thermal loading with plasticity index is required.
3-There is a need to perform a full-scale pile test to evaluate more complex issues like downdrag and pile setup due to the thermally induced consolidation.

4-Soil screening before and after applying thermal loading would give a better understanding at the microscope level.

5-Conducting interface shearing at high and low temperatures is also required instead of just heating-cooling cycle(s).

6-Conducting tests on Illite, Kaolinite, and Montmorillonite clays would give a better understanding of the clay response under thermal loading.
Figure 1. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 4.35psi and one cycle of heating up to 70°C.
Figure 2. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 10psi and one cycle of heating up to 70°C.
Figure 3. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 16psi and one cycle of heating up to 70°C.
Figure 4. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 21.8psi and one cycle of heating up to 70°C.
Figure 5. Vertical displacement under all stages of the clay-concrete test of medium P.I. soil under 10psi and one cycle of heating up to 70°C.
Figure 6. Vertical displacement under all stages of the clay-concrete test of medium P.I. soil under 16psi and one cycle of heating up to 70°C.
Figure 7. Vertical displacement under all stages of the clay-concrete test of medium P.I. soil under 21.8psi and one cycle of heating up to 70°C.
Figure 8. Vertical displacement under all stages of the clay-concrete test of high P.I. soil under 10psi and one cycle of heating up to 70°C.
Figure 9. Vertical displacement under all stages of the clay-concrete test of high P.I. soil under 16psi and one cycle of heating up to 70°C.
Figure 10. Vertical displacement under all stages of the clay-concrete test of high P.I. soil under 21.8 psi and one cycle of heating up to 70°C.
Figure 11. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 10psi and one cycle of heating up to 40°C.
Figure 12. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 16psi and one cycle of heating up to 40°C.
Figure 13. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 16psi and four cycles of heating up to 40°C.
Figure 14. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 21.8psi and one cycle of heating up to 40°C.
Figure 15. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 10psi and one cycle of heating up to 55°C.
Figure 16. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 16psi and one cycle of heating up to 55°C.
Figure 17. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 16psi and four cycles of heating up to 55°C.
Figure 18. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 16psi and nine cycles of heating up to 55°C.
Figure 19. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 21.8psi and one cycle of heating up to 55°C.
Figure 20. Vertical displacement under all stages of the clay-concrete test of low P.I. soil under 16psi and four cycles of heating up to 70°C.
REFERENCES


VITA

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