Summer 2011

A deterministic method of predicting long-term pile capacity in cohesive soils after the dissipation of excess pore water pressure

Eric J. Steward

Follow this and additional works at: https://digitalcommons.latech.edu/dissertations

Part of the Civil Engineering Commons, and the Geological Engineering Commons
A DETERMINISTIC METHOD OF PREDICTING LONG-TERM PILE CAPACITY IN COHESIVE SOILS AFTER THE DISSIPATION OF EXCESS PORE WATER PRESSURE

by

Eric J. Steward, B. S.

A Dissertation Presented in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy

COLLEGE OF ENGINEERING AND SCIENCE
LOUISIANA TECH UNIVERSITY

August 2011
We hereby recommend that the dissertation prepared under our supervision by Eric John Steward entitled
A Deterministic Method of Predicting Long-term Pile Capacity in Cohesive Soils after the Dissipation of Excess Pore Water Pressure
be accepted in partial fulfillment of the requirements for the Degree of Doctor of Philosophy in Engineering.

[Signatures]

Supervisor of Dissertation Research
Head of Department

Advisory Committee

Approved:
Director of Graduate Studies
Dean of the College

Approved:
Dean of the Graduate School
ABSTRACT

Pile setup or freeze is a phenomenon where the bearing capacity of a pile increases over time both during and after the dissipation of pore pressure and the stabilization of lateral earth pressure. Driven piles gain capacity after installation because of the dissipation of excess pore pressure and soil aging. Incorporating accurate setup prediction into the design of the piles can significantly reduce the cost of many projects by reducing the size or number of required piles. Various simple, empirical mathematical models have been developed to predict pile setup. However, these models are often unreliable, resulting in very conservative designs. Currently, an effective deterministic pile setup prediction model accounting for pore pressure dissipation and soil aging is not available.

This study establishes a mechanistically-determined prediction model for pile setup in clay due to aging. It incorporates the remolded friction angle increase with time along the pile wall after the dissipation of excess pore water pressure induced during installation and the Over-Consolidation Ratio (OCR) of the soil. Two coefficients used in the model are determined that appear to be directly related to the properties of the soil. An experimental process is developed using conventional shear strength testing equipment to verify the relationships presented in the model. This new experimental process can be utilized to simulate shear strength increase behavior between a pile and soil over time. The results from the testing program indicate an increase in the residual
shear strength between clay and concrete as time passes. The results also show a greater increase in frictional behavior when the soil has been subjected to a larger stress history prior to shearing. Combining the developed prediction model and the laboratory procedure, a method to predict the frictional resistance of a pile incorporating the pile setup mechanism of soil aging is presented. The calculated time-dependent pile capacity can be applied in the commonly utilized β-method to design the pile dimensions.
APPROVAL FOR SCHOLARLY DISSEMINATION

The author grants to the Prescott Memorial Library of Louisiana Tech University the right to reproduce, by appropriate methods, upon request, any or all portions of this Thesis. It is understood that “proper request” consists of the agreement, on the part of the requesting party, that said reproduction is for his personal use and that subsequent reproduction will not occur without written approval of the author of this Thesis. Further, any portions of the Thesis used in books, papers, and other works must be appropriately referenced to this Thesis.

Finally, the author of this Thesis reserves the right to publish freely, in the literature, at any time, any or all portions of this Thesis.

Author ____________________________

Date 7/22/2011
# TABLE OF CONTENTS

ABSTRACT ....................................................................................................................... iii

LIST OF TABLES .................................................................................................................. ix

LIST OF FIGURES .............................................................................................................. x

LIST OF EQUATIONS ....................................................................................................... xiv

LIST OF SYMBOLS AND ABBREVIATIONS ................................................................. xvi

ACKNOWLEDGEMENTS .................................................................................................. xix

CHAPTER 1  INTRODUCTION .......................................................................................... 1

1.1 General Introduction .................................................................................................. 1

1.2 Objectives .................................................................................................................. 2

1.3 Scope of Work ............................................................................................................ 2

CHAPTER 2  LITERATURE REVIEW ............................................................................... 4

2.1 Pile Setup in Cohesive Soil ....................................................................................... 4

2.1.1 The Existence of Long-Term Pile Setup ............................................................ 4

2.1.2 Mechanistic Theories of Setup ............................................................................ 7

2.1.2.1 Contribution of Setup due to Pore Pressure Dissipation ............................. 7

2.1.2.2 Contribution of Setup due to Soil Aging ...................................................... 8

2.1.3 Current Prediction Methods for Pile Setup ......................................................... 11

2.1.3.1 Empirical Relationships ............................................................................. 11

2.1.3.2 Exploratory Field Test Relationships ....................................................... 15

2.1.3.3 Soil Characteristic Relationships .............................................................. 16
2.2 Laboratory Determination of Interface Shear Strength of Clay and Pile .......... 18
2.3 Concluding Remarks .................................................................................. 22

CHAPTER 3 FRICTIONAL RESISTANCE PREDICTION MODEL .............. 23

3.1 Model Development .................................................................................. 23
  3.1.1 Constitutive Side Shear Resistance Relationships ......................... 24
  3.1.2 Friction Angle Prediction Development ............................................. 30
  3.1.3 Hydrostatic Pore Pressure Determination ....................................... 33

3.2 Model Verification .................................................................................... 34
  3.2.1 Long-Term Field Pile Test Data ......................................................... 35
    3.2.1.1 Bullock Research Summary and Conclusions .............................. 35
    3.2.1.2 Long-Term Field Testing Results ................................................ 39
    3.2.1.3 Estimating the Over-Consolidation Ratio (OCR) ....................... 41
  3.2.2 Time-Dependent Friction Angle Determination ............................... 45
  3.2.3 Pore Pressure Dissipation Determination ......................................... 53
  3.2.4 Comparison Between the Results from the Models and the Field Measurements ............................................. 54

3.3 Summary and Conclusions .................................................................... 62

CHAPTER 4 EXPERIMENTAL INVESTIGATION OF THE LONG-TERM RESIDUAL INTERFACE FRICTION ANGLE INCREASE ........ 64

4.1 Selection of Testing Apparatus .................................................................. 65
4.2 Testing Apparatus ..................................................................................... 69
4.3 Soil and Material Tested ........................................................................... 73
  4.3.1 Construction Material - Concrete ...................................................... 73
  4.3.2 Clay Samples Tested .......................................................................... 75
4.4 Testing Procedure ...................................................................................... 76
  4.4.1 Soil Preparation .................................................................................. 76
LIST OF TABLES

Table 1  A Summary of Pile Setup Factors and Reference Time (Wang 2009) ................................................................. 12
Table 2  A Summary of Pile Setup Equations (Wang, Steward and Verma 2009) ................................................................. 13
Table 3  Pile Segment Skin Resistance Increase due to Aging (Data from Bullock, 1999) .................................................................................................................. 40
Table 4  Back-Calculated Friction Angle Over Time (Data from Bullock, 1999) ................................................................................................................................. 41
Table 5  Estimating OCR using Subsurface Exploration Test Data .................. 43
Table 6  Estimated OCR using Subsurface Exploration Test Data from Bullock (1999) ..................................................................................................................... 44
Table 7  Deterministic Results of Coefficients for Each Pile Location Based on Equation 13 ................................................................................................................ 49
Table 8  Deterministic Results of Coefficients for Each Pile Location Based on Equation 18 ................................................................................................. 52
Table 9  Interface Age Testing Program .......................................................... 81
Table 10 Increased Friction Angle by Direct Shear Age Testing ....................... 93
Table 11 Deterministic Results of Equation 13 Variables $a_o$ and $a_f$ ............... 102
Table 12 Time-Dependent Increased Frictional Resistance Using Soil Aging ...... 107
## LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Time Dependent Capacity Increase from Various Studies (Titi and Wathugala, 1999)</td>
</tr>
<tr>
<td>2</td>
<td>Instrumented Pile Data, Seabreeze Bridge Pile, at 17.8 meters Below Grade (Bullock 1999)</td>
</tr>
<tr>
<td>3</td>
<td>Idealized Schematic of Pile Setup Phases (Komurka et al., 2003)</td>
</tr>
<tr>
<td>4</td>
<td>Pore-Water Pressure, Horizontal Total and Effective Stresses During Consolidation from PLS Cell (Azzouz et al., 1990)</td>
</tr>
<tr>
<td>5</td>
<td>Increased Interface Friction Angle (a, c) with Constant Ratio of Interface and Soil-Soil Friction Angle (b, d) as OCR Increases (Subba Rao et al., 2000)</td>
</tr>
<tr>
<td>6</td>
<td>Cohesion Reduction with Time (Schmertmann, 1991)</td>
</tr>
<tr>
<td>7</td>
<td>Increased Friction Component with Time (Schmertmann, 1991)</td>
</tr>
<tr>
<td>8</td>
<td>Increased Friction Component from OCR (Schmertmann, 1991)</td>
</tr>
<tr>
<td>9</td>
<td>Relationship Between $K$ and OCR using Different Methods</td>
</tr>
<tr>
<td>10</td>
<td>Increase in Pile Side Shear Capacity with Time (Bullock 1999)</td>
</tr>
<tr>
<td>11</td>
<td>Pile Segment Side Shear and Effective Stress Change After PP Dissipation (Bullock, 1999)</td>
</tr>
<tr>
<td>12</td>
<td>Instrumented Pile Data, Aucilla Bridge Pile, at 14 meters Below Grade (Data from Bullock 1999)</td>
</tr>
<tr>
<td>13</td>
<td>Estimated OCR from SPT $N_{60}$ Values, Data from Bullock (1999)</td>
</tr>
<tr>
<td>14</td>
<td>Predicted Residual Friction Angle using $a_{in}$, $a_j$, $a_2$ Calculated for Each Individual Pile Segment</td>
</tr>
<tr>
<td>15</td>
<td>Predicted Residual Friction Angle using $a_{in}$, $a_j$, $a_2$ Calculated from the Average Values of Each Individual Pile Segment</td>
</tr>
</tbody>
</table>
Figure 16 Predicted Residual Friction Angle using $a_0$, $a_1$, $a_2$ Determined for Each Pile ................................................................. 49
Figure 17 Predicted Residual Friction Angle with Varied OCR, AUC 14m Depth........................................................................ 50
Figure 18 Predicted Residual Friction Angle with Varied OCR, Aucilla 14m Depth........................................................................ 51
Figure 19 Predicted Residual Friction Angle using $a_0$, $a_l$ Calculated for Each Individual Pile Segment ........................................ 52
Figure 20 Predicted Residual Friction Angle using $a_0$, $a_l$ Calculated for Each Pile .......................................................................... 53
Figure 21 Frictional Side Shear Resistance Comparisons, Aucilla at 14m Depth........................................................................ 56
Figure 22 Frictional Side Shear Resistance Comparisons, Aucilla at 17.5m Depth........................................................................ 56
Figure 23 Frictional Side Shear Resistance Comparisons, Vilano West at 17.8m Depth................................................................. 57
Figure 24 Frictional Side Shear Resistance Comparisons, Vilano West at 21.2m Depth................................................................. 57
Figure 25 Frictional Side Shear Resistance Comparisons, Seabreeze at 12.3m Depth................................................................. 58
Figure 26 Frictional Side Shear Resistance Comparisons, Seabreeze at 15m Depth................................................................. 58
Figure 27 Frictional Resistance Comparison, Predicted vs. Measured............. 60
Figure 28 Frictional Resistance Comparison, Denver and Skov using Individual Setup Factors vs. Measured........................................... 61
Figure 29 Frictional Resistance Comparison, Denver and Skov using Setup Factor = 0.2 vs. Measured........................................... 61
Figure 30 Shear Box Device .................................................................. 66
Figure 31 Simple Shear Device................................................................. 66
Figure 32 Conventional Shear Box for Direct Shear Testing of Soil............. 69
Figure 33 Top Half of Shear Box on Concrete ........................................ 70
Figure 34 Specimen Placed in the Direct Shear Machine ................................................. 70
Figure 35 Direct Shear Machine ....................................................................................... 71
Figure 36 Schematic of Horizontal Load Application to Specimen ................................. 72
Figure 37 1-Dimensional Consolidation Devices with Interface Samples Submerged ......................................................................................................................... 73
Figure 38 Fabricated Concrete Specimen for Interface Testing ........................................ 75
Figure 39 Residual Shear Stress Versus Displacement, Sample SA3B .............................. 80
Figure 40 Vertical Deformation Versus Root Time, Sample SA1C .................................... 88
Figure 41 Shear Stress Versus Shear Strain for All Samples with OCR = 1 ............... 90
Figure 42 Shear Stress Versus Shear Strain for All Samples with OCR = 3 ............... 90
Figure 43 Shear Stress Versus Shear Strain for All Samples with OCR = 6 .......... 91
Figure 44 Shear Stress Versus Shear Strain at 90 Days of Aging at Various OCR Values ................................................................................................................................. 92
Figure 45 Friction Angle Increase Over Time with Varying OCR .................................. 94
Figure 46 Friction Angle Over Time with Varying OCR .................................................... 95
Figure 47 Location of Critical Failure Plane within the Soil, Leaving Clay Attached to Concrete ......................................................................................................................... 96
Figure 48 Friction Angle Increase Over Time with Varying OCR and Internal Soil Friction Angle ................................................................................................................................. 97
Figure 49 Friction Angle Over Time with Varying OCR and Internal Soil Friction Angle ................................................................................................................................. 98
Figure 50 Comparison of Predicted Residual Friction Angle Using $a_o$ and $a_i$ Calculated Individually for Each OCR Group ................................................................. 102
Figure 51 Comparison of Predicted Residual Friction Angle Using $a_o$ and $a_i$ Calculated from Combined Tests ......................................................................................... 104
Figure 52 Segment Side Resistance Comparisons Between the Aging Process Proposed and the Skov and Denver Method ............................................................ 109
# LIST OF EQUATIONS

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Time Factor</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>Denver and Skov Setup Prediction</td>
<td>12</td>
</tr>
<tr>
<td>3</td>
<td>Total Side Shear Pile Capacity</td>
<td>24</td>
</tr>
<tr>
<td>4</td>
<td>Frictional Resistance of Driven Piles</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>$\beta$ value of Frictional Resistance</td>
<td>25</td>
</tr>
<tr>
<td>6</td>
<td>Expanded Effective Frictional Resistance of Driven Piles</td>
<td>26</td>
</tr>
<tr>
<td>7</td>
<td>Meyerhoff Estimation of Earth Pressure Coefficient $K$</td>
<td>26</td>
</tr>
<tr>
<td>8</td>
<td>Frictional Resistance with Meyerhoff $K$</td>
<td>26</td>
</tr>
<tr>
<td>9</td>
<td>Time Dependent Frictional Resistance with Meyerhoff $K$</td>
<td>28</td>
</tr>
<tr>
<td>10</td>
<td>Mayne &amp; Kulhawy Estimation of $K$</td>
<td>29</td>
</tr>
<tr>
<td>11</td>
<td>Time Dependent Frictional Resistance with Mayne &amp; Kulhawy $K$</td>
<td>30</td>
</tr>
<tr>
<td>12</td>
<td>Initial Remolded Friction Angle Model</td>
<td>32</td>
</tr>
<tr>
<td>13</td>
<td>Remolded Friction Angle Model with Horizontal Stress</td>
<td>32</td>
</tr>
<tr>
<td>14</td>
<td>Time Dependent Pore Water Pressure Estimation</td>
<td>33</td>
</tr>
<tr>
<td>15</td>
<td>Rearrangement of Equation 4</td>
<td>41</td>
</tr>
<tr>
<td>16</td>
<td>Back-Calculated Friction Angle from Equation 14</td>
<td>41</td>
</tr>
<tr>
<td>17</td>
<td>Residuals from the Least Squares Method of Error</td>
<td>46</td>
</tr>
<tr>
<td>18</td>
<td>Final Developed Remolded Friction Angle Model</td>
<td>51</td>
</tr>
</tbody>
</table>
LIST OF SYMBOLS AND ABBREVIATIONS

$\beta$ Conversion Value Between $f_s$ and $\sigma'_v$

$\Delta L$ Incremental Unit Pile Length

$\varepsilon$ Strain

$\sigma'_h$ Horizontal Effective Stress

$\sigma_n$ Normal Stress

$\sigma_v$ Vertical Stress

$\sigma'_v$ Vertical Effective Stress

$\tau$ Shear Stress

$\tau_R$ Residual Shear Stress

$\tau_s$ Shear Stress

$\phi'_R$ Drained Residual Effective Friction Angle (Degrees)

$\phi'_{R_0}$ Initial Remolded Friction Angle (Degrees)

$\phi'_R(t)$ Time Dependent Remolded Friction Angle (Degrees)

$\phi'_R$ Measured Residual Friction Angle

$\phi''_R$ Predicted Residual Friction Angle

$A$ Skov and Denver Setup Factor

$ADU$ Autonomous Data Acquisition Unit

$ASTM$ American Society of Testing and Materials

$AUC$ Aucilla Site
\( a_1 \) Consolidation Factor Coefficient
\( a_2 \) Horizontal Stress Factor Coefficient
\( a_o \) Time Factor Coefficient
\( B \) Pile Width/Diameter
\( \textit{CAPWAP} \) Case Pile Wave Analysis Program
\( c_h \) Coefficient of Horizontal Consolidation
\( \textit{CPT} \) Cone Penetrometer Test
\( \textit{CPTu} \) Cone Penetrometer Test with Pore Pressure Measurement
\( d \) Day
\( D \) Pile Diameter
\( \textit{DMT} \) Dilatometer Test
\( \textit{DS7} \) Datasystem 7 Software
\( \textit{EOD} \) End Of Driving
\( f_s \) Unit Frictional Resistance
\( f_s(t) \) Time Dependent Frictional Resistance
\( g \) Grams
\( K \) Lateral Earth Pressure Coefficient
\( kPa \) KiloPascal
\( kg \) Kilogram
\( \textit{LVDT} \) Linear Voltage Differential Transformer
\( \textit{LaDOTD} \) Louisiana Department of Transportation and Development
\( m \) Meters
\( \textit{min} \) Minute
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>Millimeters</td>
</tr>
<tr>
<td>N</td>
<td>Newtons</td>
</tr>
<tr>
<td>$N_{60}$</td>
<td>SPT Blow Count</td>
</tr>
<tr>
<td>OCR</td>
<td>Over-Consolidation Ratio</td>
</tr>
<tr>
<td>p</td>
<td>Pile Perimeter</td>
</tr>
<tr>
<td>$p_a$</td>
<td>Atmospheric Pressure</td>
</tr>
<tr>
<td>PI</td>
<td>Plasticity Index</td>
</tr>
<tr>
<td>PLS</td>
<td>Piezo-Lateral Stress</td>
</tr>
<tr>
<td>$Q_f$</td>
<td>Final Pile Capacity Resistance</td>
</tr>
<tr>
<td>$Q_i$</td>
<td>Initial Pile Capacity Resistance</td>
</tr>
<tr>
<td>$Q_o$</td>
<td>Reference Pile Capacity Resistance (Initial)</td>
</tr>
<tr>
<td>$Q_s$</td>
<td>Side Shear Pile Resistance</td>
</tr>
<tr>
<td>$Q(t)$</td>
<td>Time-Dependent Pile Capacity</td>
</tr>
<tr>
<td>$R^2$</td>
<td>Square of Correlation Coefficient</td>
</tr>
<tr>
<td>Res</td>
<td>Residuals from Least-Squares Method</td>
</tr>
<tr>
<td>S&amp;D</td>
<td>Skov and Denver</td>
</tr>
<tr>
<td>SBZ</td>
<td>Seabreeze Site</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard Penetrometer Test</td>
</tr>
<tr>
<td>t</td>
<td>Time From the EOD</td>
</tr>
<tr>
<td>$T_h$</td>
<td>Time Factor</td>
</tr>
<tr>
<td>$t_o$</td>
<td>Initial Reference Time</td>
</tr>
<tr>
<td>u</td>
<td>Pore Water Pressure</td>
</tr>
<tr>
<td>$u_o$</td>
<td>Initial Pore Water Pressure at EOD</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td>$u_z$</td>
<td>Hydrostatic Pore Water Pressure</td>
</tr>
<tr>
<td>$u(t)$</td>
<td>Time Dependent Pore Water Pressure</td>
</tr>
<tr>
<td>$VLW$</td>
<td>Vilano West Site</td>
</tr>
</tbody>
</table>
ACKNOWLEDGEMENTS

The information presented in this dissertation required support and dedication from many organizations and people.

First, the Louisiana Tech University Board of Regents and the Louisiana Tech College of Engineering and Science provided the financial support as well as permission to use space and equipment to perform the experimentation. The Louisiana Transportation Research Center (LTRC) and the Louisiana Department of Transportation and Development (LaDOTD) was instrumental in providing the research need and support in the way of soil samples and testing guidance. The helpful and dedicated personnel at the Trenchless Technology Center on the campus of Louisiana Tech University were gracious with the time and resources, without which many of the results would not have been possible.

Individuals that require special acknowledgement are:

- Dr. Erez Allouche and Dr. Raymond Sterling for providing support and encouragement throughout the entire process,
- Fellow Graduate Students and friends Dr. John Matthews, Dr. Ivan Diaz, Dr. Saiprasad Viadya, and Shaurav Alam for continued support and helpful advice.
- Dr. Mike Baumert saw the potential for me to accomplish this long before I thought I could. He has provided immeasurable support and encouragement well beyond expectation; and he is a true friend.
- Dr. Jay Wang has provided guidance, encouragement, and support throughout this process. He presented to me the original ideas that blossomed into this dissertation, all the while allowing me the freedom to explore new ideas.

Most importantly, I would like to thank my wife, Kelly, and our sons, Brayden and Bodie, for their continued support during these years. Kelly, you have been a loving and dedicated wife who has shown grace and patience. Your faith and love has encouraged me throughout and it is to you that this dissertation is dedicated.
In geotechnical engineering practice, the increase in capacity of driven piles after installation is known as pile “setup” or “freeze”. The results of previous research indicate that the majority of setup is due to an increase in frictional resistance, or side shear, between the pile and the surround soil. Although this phenomenon has been recognized for decades, design engineers often neglect to account for setup when designing pile foundation systems due to the variability and lack of reliability of the estimates generated from current prediction techniques. Pile foundations can be a very economically expensive portion of a construction project for which the designed capacity of the piles can play an important role in the overall cost. If it were possible to incorporate setup into the design of the piles, the costs of many projects could be greatly reduced as the size of the piles, the embedment lengths, and the size of the installation crane could be reduced.

Pile setup has been studied for several decades with many empirical, semi-empirical, analytical, and numerical techniques proposed and developed, and although great progress has been made in understanding the short term and long term pile resistance increases, geotechnical engineers currently rely on empirical correlations as
well as their own personal judgment and experience to estimate pile setup. Various 
simple, but completely empirical mathematical models have been developed to correlate 
and predict pile setup. However, these models prove to be site specific, do not provide 
the ultimate pile capacity, and are often unreliable, causing engineers to provide very 
conservative designs. An effective model accounting for pore pressure dissipation and 
soil aging is not currently available.

1.2 Objectives

The main objective of this research was to develop a method to predict the long-
term capacity increase of driven piles in clayey soils using realistically obtainable 
properties of the soil, either by field or laboratory testing. This goal is quite broad, so a 
number of lesser goals were identified.

1. Investigate the soil properties that contribute to setup and develop a potential 
prediction model based on the findings
2. Utilize existing field pile data to verify the developed prediction equation
3. Develop a practical method to simulate the soil properties that contribute to pile 
setup
4. Utilize the simulated results to validate the developed prediction equation
5. Provide recommendations for the utilization of the developed prediction methods 
to implement in design

1.3 Scope of Work

The research described herein investigates the phenomenon of soil aging in clay 
associated with frictional resistance increases contributing to pile setup. Inspired by a 
research project funded by the Louisiana Department of Transportation and Development
(LaDOTD) regarding a new technique to develop a prediction method for south Louisiana clays, a model was developed to predict the increased side shear of piles over time by accounting for time-dependent soil characteristics. It was quickly determined that the increase of the interface friction angle between the pile and the soil should be the focus of the investigation, as it was the basis for the prediction model. Detailed field research data was utilized to demonstrate the usefulness of the prediction model. A laboratory testing program was then developed to study the aging process in a controlled laboratory environment to aid in the enhancement and verification of the model. The results from the laboratory testing program were also used to verify the validity of the prediction model. A summary of the work described as well as the results generated are presented.
CHAPTER 2

LITERATURE REVIEW

2.1 Pile Setup in Cohesive Soil

Long-term setup effects on driven piles in cohesive soils have been observed for over a century, yet there is little research that has defined a clear deterministic method to predict the capacity of piles that incorporates long-term setup effects.

2.1.1 The Existence of Long-term Pile Setup

For thousands of years, pile foundations have been installed for structural support of buildings, bridges and other structures along coastal areas or where weaker soils are present (Augustesen 2006). It is well known throughout the geotechnical industry that after the installation of a driven pile, the axial capacity of the pile can increase as time passes. The increase in pile capacity with time is called setup. Setup occurs primarily with respect to side shear, as dynamic pile tests from research shows little end-bearing increase with time (Bullock 1999). Accounting for or verifying the existence of pile setup can significantly increase the reliable capacity of a foundation system.

Driven piles in sands and clays generally experience setup effects. From a percentage basis, piles driven into soft to stiff saturated clays experience much more setup than in stiff clays or sands (Long et al., 1999). As early as 1900, Wendel (1900) conducted load tests on driven timber piles in clays and reported the effects of the
increase in axial capacity of piles two and three weeks after installation. Since researchers began to utilize experimental tools, such as strain gauges and piezo-electric devices, there exists widespread experimental evidence indicating the increase of axial capacity of driven piles in clay with time (Seed et al., 1955; Bjerrum et al., 1958; Skov and Denver, 1988; Bullock, 1999, Axelsson, 2000). Thompson III et al. (2009) reported an increase in side shear between 1.8 and 3 times the capacity at the end of driving (EOD) in clayey soils along the Mississippi Gulf Coast. The compiled setup data in clay in Malaysia taken from Ng indicated capacity increases from 2 to 6 times the driving capacity after 29 days (Ng et al., 2010). Long et al. (1999) presented a fairly comprehensive database obtained from publications showing the effects of time on the axial capacity of piles grouped by general soil types of (1) predominantly clay, (2) predominantly sand, and (3) a mixture of clay and sand. The results also indicated that the setup effects in clays and mixed soils increase from 2 to 6 times the capacity from the end of driving. Figure 1 provides a graphical interpretation of the increase of pile capacity on a percentage basis with time in a logarithmic scale from the data assembled by Titi and Wathugala (1999) from various studies of setup on friction piles driven in clay.
One conclusion that can be made from a visual inspection of Figure 1 is that time between the installation and the ultimate capacity of a pile varies significantly, even when investigating predominantly clayey conditions. The author cautions that the data presented here was based on the data provided with each individual study with little information available indicating the type of pile installed, detailed soil characteristics, and more significantly, whether attainment of the maximum capacity was truly achieved. If the investigators assumed that the final test was the maximum without further tests showing an equilibrium state, then the percentage of ultimate capacity at the corresponding time may be underestimated. Even with the limitations, Figure 1 provides
evidence that pile setup is a well-documented phenomenon with significant implications for the determination of the design capacity of driven pile foundations.

2.1.2 Mechanistic Theories of Setup

2.1.2.1 Contribution of Setup due to Pore Pressure Dissipation

As a pile is driven, soil is displaced radially, and to a lesser degree vertically, along the shaft resulting in large strains outward to approximately 1 to 2 pile radii depending on the type of soil, generating a completely remolded state. Randolph et al. (1979) stated that piles driven in clay significantly altered the stresses in soil radially outward to about 20 pile radii. These strains can cause an immediate increase in pore pressure which accounts for a reduction in the undrained shear strength (skin friction) due to the reduction in effective stress. After pile driving is complete, the excessive pore pressure in the surrounding soil begins to dissipate causing the shear strength to increase, allowing the pile to regain side shear capacity. Seed and Reese (1955) provided the evidence to support the theory that the initial mechanism of pile setup can be attributed to the dissipation of the excess pore water pressure induced during the pile installation. This was later confirmed by Bjerrum et al. (1958) in which it was recommended pile capacity load testing be performed 30 days after the installation to account for the dissipation of pore pressure as the cause of setup. Soderberg (1961) concluded the increase of pore pressure induced during pile installation is constant with depth. The time for the excess pore pressure to dissipate is proportional to the square of the horizontal pile dimension and is inversely proportional to the horizontal coefficient of consolidation of the soil. Soderberg proposed that the dissipation of excess pore pressures induced during pile installation could be predicted by utilizing Terzaghi’s
theory of consolidation (Soderberg, 1962). This theory presented a non-dimensional time
factor $T_h$,

$$T_h = \frac{4c_h t}{B^2}$$  \hspace{1cm} (1)

where

- $c_h =$ coefficient of horizontal consolidation,
- $t =$ time from the end of driving, and
- $B =$ pile width/diameter.

Many analytical and numerical models have utilized this theory to predict the increase of
pile capacity with time with great success during the time any induced excess pore
pressure is dissipating.

2.1.2.2 Contribution of Setup due to Soil Aging

Studies have shown that pore pressure dissipation in clays equalizes between 6 and
14 days after the pile has been installed (Karlsrud and Haugen, 1985). Figure 2
presents data retrieved from Bullock (1999) that clearly suggests that pile side shear
continues to increase well beyond the time at which the pore pressure equalizes to a
hydrostatic state.
Many other studies (Azzouz et al., 1990; Bullock, 1999; Komurka et al., 2003; Augustesen, 2006) verify the existence of this phenomenon. This post-dissipation increase in side shear has been attributed to a phenomenon called aging. Schmertmann (1991) defines aging as a time-dependent increase of soil friction resistance at a constant effective stress similar to the secondary compression after the primary consolidation has completed. The contribution of aging to pile setup can be significant, increasing the overall pile capacity up to 30% or more (Bullock 1999). While the mechanism of aging has yet to be clearly defined, Schmertmann suggests it can be attributed to particle interference, clay dispersion, thixotropy, and drained creep (secondary compression) resulting in increases in the frictional component of soil at a constant effective stress.

Komurka, et al. (2003) presented a three-phase setup path to explain the ultimate pile capacity as shown in Figure 3, which relates the ratio of the final and initial pile resistance ($Q_i/Q_r$) versus the time in log scale and provides a relatively general display of
the pile setup path. There are a number of factors involved in how this graph will behave; namely, soil type, depth of pile, depth of groundwater, type of pile. The ratio of resistances requires testing of the pile immediately after installation and also at various time intervals after the end-of-driving (EOD) until equilibrium is reached.

![Figure 3: Idealized Schematic of Pile Setup Phases (Komurka et al., 2003)](image)

The first two phases are said to be in direct relation to the dissipation of pore pressure developed during the pile installation located within the remolded soil zone surrounding the pile. The third phase presenting the aging effects has a slope much less than the linear rate of dissipation because, Komurka hypothesized that the aging effect in cohesive soils is very small and does not contribute a great deal to the ultimate capacity of the pile. However, research listed herein suggests that aging is a considerable contribution to setup in cohesive soils. This difference in hypotheses is largely due to the fact that the mechanism involved in the aging process of clays is not fully understood. It
should also be noted that there is a potential that aging could be occurring during the pore pressure dissipation phases.

2.1.3 Current Prediction Methods for Pile Setup

The design of friction piles is generally based on empirical formulas and depends greatly on the engineering judgment and experience of the engineer. Due to this uncertainty associated with the nature of the friction pile design, load testing is usually performed after pile installation. The results of the load tests are generally compared with the capacity of the piles at the end of the initial driving and then ultimate capacity is estimated (Skov and Denver, 1988). Likins (2010) highlighted two Florida projects where design engineers utilized the time for setup to occur in indicator piles. This reduction resulted in claims of cost savings in the millions of dollars for the projects. Due to time constraints during construction, indicator piles used to verify the design load of a pile foundation system are typically load tested within 14 days after they are installed, and often much sooner. Prediction methods are often limited to this type of short term testing when utilized. Many methods of prediction have been studied, yet many U.S. State agencies are not utilizing these methods (Budge 2009), presumably due to the lack of verification and reliability of the methods.

2.1.3.1 Empirical Relationships

Incorporating a prediction equation to determine setup, and thus ultimate capacity, into the design of the piles can greatly reduce project costs as the size of the piles, the embedment lengths, and the size of the installation crane could be reduced. Engineers and researchers have most widely used the Skov and Denver (1988) equation to predict the bearing capacity of piles with the contribution of pile setup at a specific time.
Presented as Equation 2, it is an empirical relationship that models setup linearly with respect to the log of time

$$\frac{Q(t)}{Q_o} = A \log_{10} \frac{t}{t_o} + 1. \quad (2)$$

The vertical bearing capacity $Q$ at some time $t$ after the EOD is related to an assumed initial or reference time $t_o$ and capacity $Q_o$. $A$ is a dimensionless setup factor used to characterize the pile and soil. Theoretically, time $t_o$ is said to be the point at which the relationship between the dissipation of excess pore pressure becomes linear with respect to the log of time. The values of both $A$ and $t_o$ have become the subject of much debate and study due to the considerable variation in site conditions and pile design options. These studies (Skov and Denver, 1988, Bullock, 1999, Axelsson, 2000) have generated varying values for $A$ using assumptions, back-calculations, or empirical relationships making it difficult for engineers to confidently utilize this equation. A brief summary of the calculated $A$ values are presented in Table 1. Using these conventional values for both $A$ and $t_o$ in Equation 1 has shown significant error when compared to measured pile bearing capacity (Wang et al., 2009).

**Table 1: A Summary of Pile Setup Factors and Reference Time (Wang et al., 2009)**

<table>
<thead>
<tr>
<th>Author</th>
<th>Soil conditions</th>
<th>$A$</th>
<th>$t_o$ (day)</th>
<th>Pile Type/Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skov and Denver (1988)</td>
<td>Sand</td>
<td>0.5</td>
<td>0.2</td>
<td>Concrete piles/Alborg, Denmark</td>
</tr>
<tr>
<td></td>
<td>Clay</td>
<td>0.2</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>Svinkin et al. (1994)</td>
<td>Clayey and Sandy soils</td>
<td>0.36~1.07</td>
<td>1 or 2</td>
<td>Pre-stressed concrete piles and H-piles/Ohio</td>
</tr>
<tr>
<td>Axelsson (1998)</td>
<td>Non-cohesive soils</td>
<td>0.2~0.8</td>
<td>N/A</td>
<td>Concrete piles/Sweden</td>
</tr>
</tbody>
</table>
There are a number of other empirical formulas that have been developed that are of similar nature to the Denver and Skov equation. Table 2 presents four such variations of the empirical equations of setup prediction with time as presented by Wang et al. (2009).

<table>
<thead>
<tr>
<th>Author</th>
<th>Equation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Camp and Parmar (1999)</td>
<td>Stiff, highly plastic sandy clay or sandy silt (Cooper Marl)</td>
<td>Concrete piles, H-piles/Charleston, South Carolina</td>
</tr>
<tr>
<td>Bullock et al. (2005)</td>
<td>Dense fine sand and soft to medium stiff silty clay</td>
<td>Concrete piles/North Florida</td>
</tr>
<tr>
<td>Yang and Liang (2006)</td>
<td>Clayey soils</td>
<td>Pipe, HP, concrete, timber, etc./various</td>
</tr>
<tr>
<td>Huang (1988)</td>
<td>$Q_t = Q_{EOD} + 0.236[1 \log(t) (Q_{max} - Q_{EOD})]$</td>
<td>$Q_t$ = capacity at time t $Q_{EOD}$ = capacity at EOD $Q_{max}$ = Maximum capacity</td>
</tr>
<tr>
<td>Guang-Yu (1988)</td>
<td>$Q_{14} = Q_{EOD}(0.375 S_t + 1)$</td>
<td>$Q_{14}$ = capacity after 14 days $S_t$ = Sensitivity of soil</td>
</tr>
<tr>
<td>Svinkin (1996)</td>
<td>$Q_t = 1.4Q_{EOD} t^{0.1}$</td>
<td>Upper Bound</td>
</tr>
<tr>
<td></td>
<td>$Q_t = 1.025Q_{EOD} t^{0.1}$</td>
<td>Lower Bound</td>
</tr>
<tr>
<td>Svinkin and Skov (2000)</td>
<td>$\frac{Q_u(t)}{Q_{EOD}} - 1 = B[\log(t) + 1]$</td>
<td>$B$ = similar to A in Denver and Skov</td>
</tr>
</tbody>
</table>
Komurka et al. (2003) summarized the equations presented in Table 2. Huang (1988) presents a formula to predict the increased pile capacity with time in the soft soils of Shanghai using H-piles. There is an assumption that the value of the maximum pile capacity can be obtained. Svinkin (1996) presented a prediction formula for setup in sands and Svinkin and Skov (2000) presented a variation of the Denver and Skov equation using a consistent $t_0$ of 0.1. Guang-Yu (1988) made an attempt to include a variation with the characteristic of the soil by comparing the increase in capacity with the sensitivity of the soil. However, the relationship is only valid for 14 days after pile installation. As with the widely used Denver and Skov prediction method, these are all empirically based and seem to be specific to the region or soil type related specifically to the study.

Tables 1 and 2 illustrate the variation inherent in these types of prediction models. The predictions made with these types of empirical relationships are the total capacities. There is no general agreement regarding the appropriate time period after the installation when setup should be investigated, No provision made for considering the differing soil characteristics that alter the rate and amount of setup, and there is the inherent nature of the log/linear relationship that does not allow for the prediction of an ultimate capacity.

A growth-rate-based prediction model was developed by Wang (2010a, 2010b) based on the restrike data of 95 production piles, and restrike and load testing data of 9 tested piles from a south Louisiana highway project. The model utilizes a reference time and initial frictional component, such as unit skin friction, total frictional resistance or total side shear capacity, to calculate the same frictional component some time after the reference time. The rate-based method is an attempt to determine the ultimate capacity,
as the pile side shear must equalize at some time after installation. This ultimate capacity is a major deficiency of the previously mentioned empirical methods. The rate-based method is also empirical in nature and lacks the long-term setup data to accurately define aging as a setup effect.

2.1.3.2 Exploratory Field Test Relationships

The improvements of technology with the exploratory field testing techniques have improved tremendously over the past three decades. Utilizing an alternative version of the standard penetration testing devices, such as the Torque test or the Uplift test, can provide relationships that allow setup behavior of soils to be determined. Bullock (1999) found the SPT-Torque test to provide a good fitting relationship with the pile setup data in cohesive soils. Rausche et al. (1996) provided a ratio of resistance values between 10 minutes and 70 minutes from SPT Uplift tests to correlate with setup and when compared with SPT-Torque tests showed good agreement.

Another more advanced exploratory testing device is the electric Cone Penetrometer with Pore-water pressure measurement (CPTu), often called the Piezocone. The penetrometer is hydraulically driven into the soil and provides nearly continuous data of tip bearing, side shear, pore pressure, inclination, and temperature (Komurka, 2003). Because the device provides both side shear and pore pressure readings, opportunity exists to perform staged testing to investigate the increase in side shear as the pore pressure dissipates. The results of the CPTu tests are a function of the penetration rate due to the nature of the test. This requires a standardized penetration rate which provides an increase in pore pressure that may not correlate specifically with pile driving and subsequent dissipation. While there is a considerable amount of information that can be
obtained from this device, studies have shown that initial correlations between CPTu tests and pile setup are inconclusive (Bullock 1999). After an extensive survey and review, Budge (2009) recommended the State of Minnesota pursue utilizing the CPTu tests as a method of pile setup prediction. However, additional research is necessary.

2.1.3.3 Soil Characteristic Relationships

Lukas and Bushell (1989) performed a study on pile setup as it relates to the undrained shear strength and the sensitivity of the soil. Using *in-situ* shear vane tests, research showed an estimated adhesion factor can be determined at the time of driving and after some period of time to correlate with pile setup. This type of correlation is based on the total stress theory which neglects the effects of the effective stress increase with the dissipation of the excess pore pressure. While there may be a link between the undrained shear strength and the initial setup values of pile installation, long-term pile setup effects would appear to be better evaluated under drained conditions.

Azzouz and Morrison (1988) evaluated pore pressure and horizontal effective stress data from a Piezo-Lateral Stress (PLS) Cell installed within a model pile. The PLS Cell is equipped to provide the total horizontal stress, pore pressure, and shear stress acting on a cylindrical pile shaft. The results from the model pile tests provided the expected outcome of the dissipation of pore pressure and the stabilization of the horizontal effective stress, as indicated by the horizontal effective stress ratio, \( K \) (Lateral Earth Pressure Coefficient), shown in Figure 4.
Figure 4: Pore-Water Pressure, Horizontal Total and Effective Stresses During Consolidation from PLS Cell (Azzouz et al., 1990)

It was recommended that prediction for the horizontal stresses during this dissipation time be made by two models. The first model estimated the soil behavior during the installation of the pile by the Strain Path Method, which is a two-dimensional model where the strains in the soil are estimated by means of known velocities of an ideal fluid moving around a pile shape. Once the soil characteristics are established, the horizontal stresses and pore pressures are estimated by the cavity expansion method. Once the stresses become static, the frictional resistance is estimated by a method called the ρ-method. The ρ-method uses a skin friction ratio \((\tau/\sigma')\) which accounts for the friction angle between the pile and soil or internally within the soil, whichever is lower.

The frictional resistance value determined by the skin friction ratio in the ρ-method appears to provide a good basis for future investigation, however, the methods of
obtaining the data and the theories are not commonly utilized by design engineers. Additional correlation and significant research needs to be performed prior to incorporation in standard design procedures.

### 2.2 Laboratory Determination of Interface Shear Strength of Clay and Pile

Soil-structure interface systems are encountered in various Geotechnical engineering projects such as pile foundations, earth dams, retaining structures, and pipeline infrastructure. Researchers have used laboratory testing to study the frictional behavior between clay and solid material for many decades to aid in the estimation of parameters encountered within the actual structures as summarized by Zhang (2009). Potyondy (1961) presented early laboratory test results into the relationship between the soil internal friction angle and the interface friction angle and established that both cohesion and internal friction should be considered depending on the characteristics of the soil and the surface roughness of the solid material. Many studies followed to investigate specific aspects of the interface behavior of soils and construction materials. The majority of these studies involved non-cohesive soils due to the complexity and variety of cohesive soils. More recent studies have presented the behavior of the interface of clay with construction materials, such as steel or concrete, and presented the differences between the direct shear and the simple shear device as well as the effects of surface roughness, moisture content, drainage condition, and the stress history on the shearing strength (Skakir and Zhu, 2009; Subba Rao et al., 2000; Tsubakihara and Kishida, 1993).

Lupini et al. (1981) discussed the drained residual strength of clays with various clay fractions and defined three types of failure modes that depend on the type of soil
particles, the arrangement of the particles, and the surface interaction that the particles are adjacent to. The first is turbulent mode which is a behavior that is dominated by larger rotund particles that have high interparticle friction resulting in high residual strength. Sliding mode occurs in a lower interparticle frictional strength state where a strongly oriented platy surface develops. The developed shearing surface will then be less affected by subsequent stress histories. The third is a transitional mode which involves both turbulent and sliding behavior in different parts of the shearing surface (Lupini et al., 1981). While this study is specific to a shearing surface internally located within soil strata, it is believed by the author that the principles can be translated to the shearing surface at or close to the interface of soil and a structure.

Tsubakihara and Kishida (1993) presented a study comparing the frictional behavior of clay and steel between the standard direct shear device and the simple shear device. It was concluded that the simple shear device was more effective in presenting the peak interface strength in terms of effective stress than the shear box type due to the unreliable pore pressure data within the shear box as well as the difference in shear deformation of the clay specimen within the box. Also, the peak strength was not dependent on drainage condition or the consolidation pressure of the clay. In comparing the loading rates of the interface testing, the rates influenced the maximum friction resistance, yet had little influence on the residual friction. It should be noted that the residual friction was measured in a single shearing direction to 10 mm. Finally, the study confirmed previous Lupini et al. (1981) findings regarding sliding and turbulent shearing with a variation regarding the roughness of the surface of the solid material replacing the rotund versus layered/orientated platy shear surfaces.
Subba Rao et al. (2000) presented a study investigating the interface behavior of clay soils on solid steel plates in a direct shear type apparatus. The surface roughness of the steel plates tested was gradually increased to try to achieve a similar frictional response as would be obtained with a soil-soil interface. The soil samples used contained differing clay fractions and were made by applying a normal force on a slurry to achieve a saturated state. The samples were then subjected to three different OCR values and consolidated for 12 hours prior to shear testing. The results showed an increase in peak shear stress as the surface roughness increased. As the OCR value increased, the shear stress also increased, however, the ratio of the peak effective interface friction angle and the peak effective internal soil friction angle for each soil tested exhibited independence of the OCR as shown in Figure 5.
Figure 5: Increased Interface Friction Angle (a, c) with Constant Ratio of Interface and Soil-Soil Friction Angle (b, d) as OCR Increases (Subba Rao et al., 2000)

Shakir and Zhu (2009) performed research investigating the interface shear strength of clay of different moisture contents in contact with concrete with different degrees of surface roughness. Similar to Tsubakihara and Kishida (1993), tests were also performed using both the simple shear and the direct shear device to compare the results. Many of the conclusions drawn corroborated the results of previous research studying sliding versus deformation as the surface roughness increases and the difference in the maximum shear strength between the direct shear box and the simple shear device. Shakir and Zhu found that as the moisture content increased, the shearing mode became more turbulent than sliding. These findings support, to some extent, the reported findings.
from Eide et al. (1961) stating the dissipation of pore pressures at the wall of a pile created an adhering effect between clay and the concrete (or timber). This effect creates a shift in the location or type of shearing zone from sliding between the clay and the solid structure to a shearing zone some small distance within the clay, which the author states as displacement shearing. This effect within the Shakir and Zhu report is related to the high moisture content which most likely created a slight increase pore pressure, then dissipation occurs into the concrete aiding in the shifting of the shearing location (Eide et al., 1961).

### 2.3 Concluding Remarks

Standard design methods often neglect the effects of long-term pile setup which produce overly conservative pile capacity predictions. Indicator piles are often installed on the site to be tested after a short period of time to verify design prior to the remaining construction process. When the common empirical prediction methods are utilized in design, they still have limitations and require testing of indicator piles. Research has shown that cohesive soils produce an increase in side shear friction after the dissipation of the induced excess pore pressure, yet there is no mechanistic design prediction available to designers to account for setup.

From a mechanistic perspective, research has contributed to a relatively sophisticated understanding of the interface behavior of clays with solid surfaces. However, the laboratory determination of long-term residual frictional increase has not yet been thoroughly investigated.
CHAPTER 3

FRICTIONAL RESISTANCE PREDICTION MODEL

With the current state of long-term pile foundation design relying on empirical relationships with input variables based on load tests within days of pile installation, there is a need to develop a design methodology that incorporates longer-term changes in soil characteristics after pile installation.

3.1 Model Development

Documentation has shown the significant contribution of horizontal effective stresses leading to an increase in shear resistance at the pile wall occurring during the dissipation of excess pore water pressure. Axelsson (2000) concluded that the initial horizontal stress plays a role in the amount of setup that occurs with time. However, the increases in horizontal effective stresses presented in the study were related to the relaxation of the arching effect in non-cohesive soils. Cohesive soils lack the structure to develop the extensive arching necessary to provide the relaxation effects. Studies have also shown an increase in pile capacity with time at a constant horizontal effective stress (Karlsrud and Haugen, 1985). Once a hydrostatic state occurs within the soil body surrounding the pile, the increase of shear resistance at the pile-soil interface is caused by aging within the soil. With the assumption that aging is the sole contributor to the increase in side shear after the dissipation of induced excess pore pressure, a relationship
is developed to predict the long-term and potentially the ultimate frictional resistance of a driven friction pile within a clayey soil. This relationship will be based on the assumptions that the horizontal stresses become constant after the excess pore pressure has dissipated and that the mechanism of aging can be quantified by measurable soil properties.

3.1.1 Constitutive Side Shear Resistance Relationships

Total side shear pile capacity, $Q_s$, is defined as the frictional resistance times the surface area of the pile as Equation 3,

$$Q_s = \sum f_s p \Delta L,$$

where $f_s$ is the unit frictional resistance, $p$ is the perimeter of the pile, and $\Delta L$ is the incremental pile length over which $p$ and $f_s$ are taken to be constant.

Schmertmann (1991) indicated that the soil aging-strength-gain effect results from the increased basic soil friction and not from the increase in cohesion. Laboratory testing performed on a cohesive soil without residual effects after one day and then five days produces results indicating not just a constant cohesion, but an overall reduction in the cohesion, as illustrated by Figure 6.
When interpreting the effects from the installation of a pile which result in the remolded or residual behavior of a cohesive soil, it would be advisable to assume no cohesion effects during the aging process of a driven displacement pile.

Based on previous research, the frictional resistance per unit wall area will be determined on the basis of the effective stress parameters of the clay in a remolded state, i.e. $c' = 0$ (Das 2007). Following the $\beta$-method, at any depth of a driven pile, the unit frictional resistance can be written as:

$$ f_s = \beta \sigma'_v, $$

where

$$ \beta = K \tan \phi'_R, $$

with $\sigma'_v$ representing the vertical effective stress and $\phi'_R$ is representing the drained friction angle of remolded clay (residual) (Meyerhoff 1976). Horizontal stresses are typically estimated by utilizing the vertical stress with a lateral earth pressure coefficient.
multiplier, $K$. Substituting Equation 5 into 4 produces the common equation for frictional resistance of a pile

$$f_s = K(\sigma_v - u)\tan\phi'_R = \sigma'_R\tan\phi'_R.$$  

(6)

where $u$ is the total pore water pressure. Meyerhoff (1976) stated that the lateral earth pressure coefficient in clay can be taken at rest, whether soft or stiff, however, there can be a wide variation in the value of $K$ and this should only be considered an estimate when accurate values are not available. The lateral earth pressure coefficient taken at rest is estimated as

$$K = (1 - \sin \phi'_R)\sqrt{OCR}.$$  

(7)

By substituting the equivalent value for the lateral earth pressure coefficient, Equation 7, into Equation 6, the variables to calculate the unit frictional resistance along a pile wall become fundamentally evident as Equation 8:

$$f_s = (1 - \sin \phi'_R)\sqrt{OCR}(\sigma_v - u)\tan\phi'_R.$$  

(8)

Schmertmann (1991) provided an initial look into the increase of the mobilized friction angle with time. Figure 7 provides experimental results comparing the friction angle with the associated axial strain percentage after one day of constant stress, and then again after five days of constant stress. During this five day comparison, the change in friction angle was considerably larger and more dramatic at lower strains.
Similarly, the stress history of clay typically plays an important role in the interparticle behavior. Schmertmann (1991) found that the Over-Consolidation Ratio (OCR) will dramatically increase the mobilized frictional component of a clayey soil while maintaining a near constant cohesion as indicated by Figure 8. There is not a time component to this experimentation, but the effect is fundamentally apparent.

Figure 7: Increased Friction Component with Time (Schmertmann, 1991)

Figure 8: Increased Friction Component from OCR (Schmertmann, 1991)
Looking into the behavior of the disturbed soil surrounding the pile once the pore water pressure has become hydrostatic and based on the assumption that the horizontal effective stresses are now in steady-state condition, the remolded friction angle and the OCR are the only parameters providing a contribution to the increase in frictional resistance. A time element can now be added into the equation to account for the aging effect. Therefore, the equation can be written as Equation 9,

\[ f_s(t) = (1 - \sin \phi'_R(t))\sqrt{OCR}(\sigma_v - u(t))\tan\phi'_R(t). \]  \hspace{1cm} (9)

The frictional resistance at a specific time after the pile has been installed is dependent upon the residual friction angle of the clay, the horizontal effective stress, and pore water pressure all at that specific time, based on Equation 9. It will be assumed that the OCR will remain constant for the duration of the investigation. The time dependency on the pore water pressure is crucial in determining the time after pile installation at which the steady-state conditions of the horizontal effective stress occurs.

The evidence indicates that frictional side shear increases over time and is directly related to an apparent increase in residual friction angle as time passes caused by aging. Upon first investigation, Equation 9 appears to be a reasonable relationship, because common practice suggests the equations used in its development are accurate. However, if the unit frictional side shear were to be calculated using Equation 9, there is a mathematical deficiency in the hypothesis that an increase in the friction angle increases the frictional resistance. A friction angle of 38 degrees produces a maximum value of frictional resistance, suggesting that this equation has limitations. As time passes after the dissipation of pore water pressure, the effective stress becomes a constant. It can be assumed that the OCR is a constant as time passes. In a hydrostatic state with only soil
aging occurring, the friction angle is the only contributor to the increase in friction resistance with time. Meyerhoff’s presentation of the lateral earth pressure coefficient equation presented as Equation 7 explains that estimating $K$ using the square root of OCR often underestimates the actual value from field data (Meyerhoff 1976).

Further investigation presented by Mayne and Kulhawy (1982) provides a relationship between OCR and the lateral earth pressure coefficient, $K$, based on a statistical analysis of 189 data points. The research reveals a varying relationship between $K$ and the friction angle during loading and unloading. Equation 10 shows this relationship,

$$K = (1 - \sin \phi_R')OCR^\sin \phi_R',$$

which seems to be more consistent with the trends shown in research.

Figure 9 provides the graphical representation of the relationship between frictional resistance and the friction angle with the two methods described. The normal range encountered of residual friction angle of soil is between 10 and 50 degrees, so the frictional side shear begins to decrease when using the square root of the OCR. It is also encouraging that the frictional resistance begins to have a reduced slope as the friction angle increases, suggesting a potential equilibrium state or maximum value as the friction angle increases.
Implementing the Mayne and Kulhawy relationship into the frictional resistance equation produces Equation 11, which will be utilized throughout the remainder of the study,

\[ f_s(t) = (1 - \sin \phi_R(t))(OCR^{\sin \phi_R})(\sigma_y - u(t))\tan \phi_R(t). \]  

(11)

3.1.2 Friction Angle Prediction Development

It would appear intuitively apparent that the friction angle would increase in disturbed and then remolded clay with time due to the altering of the soil particle arrangement when compressive forces are applied. Schmertmann (1981) hypothesized this:

* A clay can and will slowly readjust its fabric under drained conditions, such as during long periods of time at constant stress. The more easily dispersed (moved) particles or aggregates of particles yield by particle-
to-particle slippages to those particles or aggregates of particles with more rigidity and which probably also have more strength and more resistance to dispersion. Such slippage and consequent yielding produces secondary compression in one-dimensional or isotropic consolidation, or creep if the soil can develop shear strain. With time the soil becomes stronger and stiffer as a result of the yield-transfer of applied shear to those stiffer and stronger aggregates.

From the statement, remolded soil will gain strength under constant stress over time, but the increase in frictional behavior under a constant stress is not defined. Creep is defined as the time-dependent permanent deformation that occurs under stress. The deformation occurring within a soil block surrounding an installed friction pile once the initial stresses have become equalized will be very difficult to measure. Also, as previously stated, Schmertmann (1991) showed the most dramatic increase in friction angle is in the low strains, which is most likely due to creep effect. The concept of developing a relationship based on obtainable soil information using creep theory is relevant and useful here. Due to the secondary compressive nature of the soil aging process, a simplified creep model was used to predict the remolded friction angle of the soil at a specific time.

The framework of the model begins with the two expected elements of time and an initial remolded friction angle. Time will be normalized by an initial time which will be the time at which the pore water pressure has dissipated. The relationship between the remolded friction angle and time will be logarithmic, which follows the same pattern of long-term models in soils. The initial remolded friction angle is assumed to be the
standard remolded friction angle of the clay at a hydrostatic state, which remains consistent with the assumptions previously stated. Equation 12 provides the beginning model,

$$\phi_R(t) = \log \left( \frac{t}{t_0} \right) + \phi'_{R_0}. \quad (12)$$

As previously mentioned in the Axelsson study (2000), it is evident that the amount of horizontal effective stress plays a role in determining the increase in friction angle achieved. This value, which is typically normalized by the atmospheric pressure of 100kPa, will be added to the model as an influencer to the amount of increase and will be quantified by an exponential coefficient. This exponential value will be assumed to be dictated by the material properties of the soil and is to be quantified through further analysis.

Schmertmann (1991) showed that shear strength and friction angle of soils will increase more dramatically as OCR increases. Enhancing Equation 12 with the additional components of OCR and horizontal stress produces Equation 13 which will be analyzed to calculate the remolded friction angle of clay after the dissipation of pore water pressure based on the initial remolded friction angle,

$$\phi'_{R}(t) = a_0 OCR a_1 \left( \frac{\sigma'_h}{p_a} \right)^{a_2} \log \left( \frac{t}{t_0} \right) + \phi'_{R_0}. \quad (13)$$

Where $\phi'_{R}(t)$ is the time dependent remolded friction angle of the clay after the dissipation of pore water pressure, $\sigma'_h$ is the steady-state horizontal effective stress of the clay, $p_a$ is the atmospheric pressure to normalize the stress ($\sim 100 \text{ N/m}^3$), $t_0$ is the time at which hydrostatic conditions occur within the pore water, $\phi'_{R_0}$ is the initial remolded friction angle at $t_0$, and $a_0, a_1, a_2$ are constants.
Similar to the relationship between the capacity ratio and the time ratio with the Skov and Denver method, this model uses an initial time to begin the analysis, along with a corresponding friction angle. The difference here is that \( t_0 \) is clearly defined as the time when the pore pressure stabilized after dissipation, whereas in the Skov and Denver model, \( t_0 \) is a highly debatable value (assumed to be 24 hours in some cases). Unlike those statistics-based models directly for pile capacity prediction, Equation 13 can be viewed as a constitutive description of friction angle change during soil aging. Additional coefficients \( a_0, a_1, \) and \( a_2 \) are believed to be factors that take into account material properties such as permeability, consolidation coefficients, etc. and will be determined using experimental data.

It should be noted at this point that the remolded friction angles included in this model are assumed to be the friction angle within the clay. However, there is a need to investigate the friction angle between the pile and the clay to determine if the critical failure occurs along the slip surface between the pile and the soil or within the soil mass as aging occurs. This concept will be further developed in later sections of this work.

3.1.3 Hydrostatic Pore Pressure Determination

The time required for the pore pressure to dissipate within a soil mass has been extensively investigated. The standard model follows Terzaghi's one-dimensional consolidation theory where the excess pore pressure out to one pile diameter from the pile can be written as (Wood 2004),

\[
u(t) = \frac{4}{\pi} \sin \left( \frac{\pi D}{2} \right) e^{-\pi^2 \left( \frac{t}{T} \right)} u_o + u_z,
\]

where \( D \) represents the pile diameter, \( u_o \) is the maximum pore pressure at EOD, and \( u_z \) is the hydrostatic pore pressure calculated from depth and soil properties. The time factor \( T \)
is previously presented as Equation 1 and dependent upon the coefficient of consolidation.

It should be noted that the pore water pressure calculated with this model reflects the soil within one pile diameter of the pile wall. To reiterate previously mentioned research by Eide et al. (1961), if the pile material is porous such as concrete or timber, then there is a tendency for the pore pressure to dissipate into the pile material reducing the pressure within the soil at a small distance from the pile wall. This rapid dissipation could reduce the accuracy of the pore pressure model presented, but more importantly, it could change both the amount of frictional resistance as well as the location of the shearing zone.

3.2 Model Verification

The developed model presented as Equation 13 requires field or laboratory data to verify, refute, or enhance its viability and reliability. Ideally, the information would be provided by a large database of pile setup research in various types of soils that includes unit side shear, pore pressure, and horizontal stress at various depths from continuous analysis along a time period of a year or longer from the end of driving. The database would also include complete soil boring characteristic data (Atterberg limits, moisture content, natural density, consolidation history and coefficients) for the surrounding strata before and after pile installation. Such a database does not exist due to the fact that very little research providing all of this information has been performed. To the author’s knowledge, no research has been conducted that combines the previously mentioned pile testing procedures with a detailed examination of the soil characteristics at the pile-soil
interface with varying depth. The reason for this is the extreme cost of such an extensive and lengthy project.

3.2.1 Long-Term Field Pile Test Data

An investigation of the increase of friction angle during pile setup is most reliable if the data can be obtained from full-scale pile research. The majority of instrumented pile research has been used to enhance existing prediction models where only the capacity data is required (Titi et al., 1999; Camp and Parmar, 1999). When pore pressure measurements are included, the time of the pile testing is only investigated until an equilibrium state has been reached, which can be in as little as 1 day and up to 14 days (Titi et al., 1999; Long et al., 1999). The most comprehensive research study found by the author is presented in Bullock’s Ph.D. dissertation (1999) with results from a five-year long study into the setup effects of five fully instrumented piles installed and tested in north Florida in various types of soils. Bullock presents a number of appendices that include a large amount of data that prove to be extremely unique and helpful to provide further understanding of the mechanism of aging as pile setup occurs. This information is critical in the evaluation of the friction angle model presented herein.

3.2.1.1 Bullock Research Summary and Conclusions

Bullock (1999) conducted a test pile program for nearly five years, in which instrumented prestressed concrete piles were driven into coastal plain soils at four bridge construction sites in northern Florida. This program presented the most extensive pile setup data collected to date, with regard to long-term increased side shear. The primary soils in which test piles were driven include soft to medium clays, stiff silty clays, and dense fine to medium sands. An Osterberg-cell was cast into the tip of each pile for axial
load testing. Strain gauges were installed at soil boundaries. Total stress cells and pore pressure cells were installed at 18 pile segments centered in one pile face between adjacent strain gauge elevations. Each pile load test series included three to six static tests using the embedded Osterberg-cell at the pile bottom to perform multiple staged shear testing of the entire pile, with up to 1,727 days in total setup time. In the long-term staged tests, shear strains, total horizontal earth pressure, and pore pressure values were measured at different segments of each individual pile. Shear force and average shear stress acting on the pile wall were then calculated over time in repeated tests, in an effort to investigate the time effect on the side shear setup.

Bullock found that all pile segments, regardless of the type of soil, show setup continuing long after the dissipation of pore pressures as shown in Figure 10. The results presented in Figure 10 are the total pile capacity with time as indicated initially by the estimates from dynamic measurements during restrikes of the pile analyzed by CAPWAP, and then from the results of the static axial load testing utilizing the Osterberg-cell installed at the pile base.
Figure 10: Increase in Pile Side Shear Capacity with Time (Bullock 1999)

The long-term setup data presented by Bullock in this research using the Osterberg-cell requires the complete mobilization of the pile, which causes slight disturbances in the soil adjacent to the pile. Thus, the data provided after subsequent staged testing might not be entirely characteristic of actual aging effects at the interface of the pile. However, this research appears to provide a unique dataset. It should also be noted that the piles located at the Seabreeze site (SBZ) and the Aucilla site (AUC) were installed in predominantly clay soils and produced the most dramatic side shear increase. The research also concluded that the unit side shear increase appeared to be independent of the horizontal effective stresses once the pore water pressure had equalized as shown in Figure 11.
Bullock integrated the results from the investigation into the existing predictive models (Skov and Denver) to generate and verify appropriate setup factors. While individual setup factors were calculated for each pile segment, Bullock concluded that for conservative design, a setup factor of 0.2 can be used for all soils in the North Florida region when predicting setup.

While Bullock’s research did include a brief study on the use of site investigative tools, such as STP, CTPu, and DMT testing, the testing was only performed at one pile.
site. The results from the testing were used to compare with stresses and shear forces of the pile load testing and did not include an investigation into the mechanism causing the soil aging. Hence, Bullock’s research did not include significant subsurface parameters such as OCR, consolidation coefficients, plasticity indices of the soils surrounding the piles. Boring logs within close proximity to the pile locations with limited information were retrieved.

3.2.1.2 Long-Term Field Testing Results

From the load testing data files provided by Bullock (1999), the author was able to assemble the information to compare the time after pile installation at which the pore pressure and the horizontal effective stress became static. Similar to Figure 2, Figure 12 provides a comparative look at the stabilized pore pressure and horizontal effective stress with the increased unit side shear at that specific location. All six pile segments adjacent to clay show a similar trend. Table 3 provides the time for pore pressure dissipation and the skin resistance increase results for the six pile segments surrounded by clay.
Figure 12: Instrumented Pile Data, Aucilla Bridge Pile, at 14 meters Below Grade (Data from Bullock 1999)

Table 3: Pile Segment Skin Resistance Increase due to Aging (data from Bullock, 1999)

<table>
<thead>
<tr>
<th>Pile Segment</th>
<th>Depth Below Grade (meters)</th>
<th>Stabilized Pore Pressure (days)</th>
<th>Skin Friction at Stabilized Pore Pressure (kPa)</th>
<th>Skin Friction at Final Load Test (kPa)</th>
<th>Aging Time (days)</th>
<th>Percent Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aucilla</td>
<td>14</td>
<td>16</td>
<td>43</td>
<td>77</td>
<td>1710</td>
<td>79%</td>
</tr>
<tr>
<td>Aucilla</td>
<td>17.5</td>
<td>16</td>
<td>114</td>
<td>136</td>
<td>1710</td>
<td>19%</td>
</tr>
<tr>
<td>Seabreeze</td>
<td>17.8</td>
<td>4</td>
<td>30</td>
<td>86</td>
<td>1053</td>
<td>186%</td>
</tr>
<tr>
<td>Seabreeze</td>
<td>21.2</td>
<td>17</td>
<td>91</td>
<td>115</td>
<td>1053</td>
<td>26%</td>
</tr>
<tr>
<td>Vilano W</td>
<td>12.3</td>
<td>18</td>
<td>18</td>
<td>27</td>
<td>139</td>
<td>50%</td>
</tr>
<tr>
<td>Vilano W</td>
<td>15</td>
<td>18</td>
<td>36</td>
<td>58</td>
<td>139</td>
<td>61%</td>
</tr>
</tbody>
</table>

The pore pressures became hydrostatic on or before 18 days and the increase of side shear resistance ranges from 19% to 186% after the effective stresses have stabilized.
To determine the remolded effective friction angle of the soil, Equation 4 can be changed to directly include the horizontal effective stress as provided by the testing data producing Equation 16,

\[ f_s = K(\sigma_v - u)tan\phi'_R = \sigma'_h tan\phi'_R, \]

\[ \phi'_R = \tan^{-1} \frac{\sigma'_h}{f_s}. \]

Now utilizing Equation 16, the time-dependent remolded friction angle can be determined for each measurement of frictional resistance provided by the dynamic load testing of each pile. Table 4 provides the results of back-calculated friction angle with time after the stabilization of pore pressure.

**Table 4: Back-Calculated Friction Angle Over Time (Data from Bullock, 1999)**

<table>
<thead>
<tr>
<th></th>
<th>Aucilla</th>
<th>Seabreeze</th>
<th>Vilano West</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time (days)</td>
<td>Time (days)</td>
<td>Time (days)</td>
</tr>
<tr>
<td></td>
<td>14.0 m</td>
<td>17.5 m</td>
<td>17.8 m</td>
</tr>
<tr>
<td>16</td>
<td>23.3°</td>
<td>42.2°</td>
<td>4</td>
</tr>
<tr>
<td>41</td>
<td>29.2°</td>
<td>38.5°</td>
<td>18</td>
</tr>
<tr>
<td>65</td>
<td>24.6°</td>
<td>40.9°</td>
<td>70</td>
</tr>
<tr>
<td>265</td>
<td>32.9°</td>
<td>43.6°</td>
<td>293</td>
</tr>
<tr>
<td>1726</td>
<td>23.3°</td>
<td>42.2°</td>
<td>1057</td>
</tr>
</tbody>
</table>

The results given in Table 4 show an increase in both the friction angle of the clay and the skin friction, which indicates a direct relationship between the friction angle of the clay and the frictional resistance. So, this assumption in generating the basis for the model has merit.

### 3.2.1.3 Estimating the Over-Consolidation Ratio (OCR)

Research has shown that the OCR plays a role in the amount of frictional angle change with time (Schmertmann, 1991). The final variable that can be estimated directly...
from the data provided by Bullock (1999) in Equation 9 is the OCR value at each pile segment depth. Because oedometer testing was not performed on the soil from the pile test sites, the OCR for each soil type being investigated along the pile must be estimated. Estimation of the OCR at various sites and depths requires the results from common subsurface investigation devices.

A significant amount of effort was used in an attempt to accurately determine the OCR of the clay at each pile segment being investigated. There have been a number of studies that relate the results from subsurface testing, such as the Cone Penetration Test (CPT), the Dilatometer Test (DMT), and the Standard Penetration Test (SPT), to an estimate of the OCR. Kulhawy et al. (1990) provided a compilation of these techniques to estimate many soil characteristics including OCR. Abu-Fasakh (2004) reported a comparison of CPT techniques to estimate in-situ soil properties including OCR and then developed a new technique to estimate OCR from CPT data that is reported to be more accurate for the Louisiana clays encountered in the study.

Each site in the Bullock study was tested using the CPT and the SPT devices and the data was provided. The Vilano West site was more thoroughly investigated using the CPT, SPT, as well as the DMT devices in an attempt to utilize the results from the testing to estimate the pile setup. Bullock found the results from SPT showed the best potential as a possible precursor test to estimate pile setup. However, research has shown that the DMT data can provide more accurate correlations with soil properties due to lowered angular interaction of the testing device with the soil creating a less disturbed testing surface (Marchetti 1980). The author utilized these correlations between the test data
from the SPT, CPT, and DMT as provided by Bullock to estimate the OCR for each pile segment in this investigation. The following correlation equations were utilized.

Table 5: Estimating OCR using Subsurface Exploration Test Data

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Method</th>
<th>Equation</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT</td>
<td>(Mayne and Kemper 1988)</td>
<td>$OCR = 0.193 \left( \frac{N_{60}}{\sigma'_o} \right)^{0.689}$</td>
<td>$\sigma'<em>o$ = effective vertical stress in MN/m², $N</em>{60}$ = Standard Penetration Number</td>
</tr>
<tr>
<td>CPT₁</td>
<td>(Schmertmann, 1978)</td>
<td>$OCR = \left( \frac{S}{S_n} \right)^{1.13+0.04(S/S_n)}$</td>
<td>$S = (s_u/\sigma'<em>{vo})$, based on cone factor $N</em>{kt}$, $S_n = (s_u/\sigma'<em>{vo})</em>{NC} = 0.11 + 0.0037 I_p$</td>
</tr>
<tr>
<td>CPT₂</td>
<td>(Mayne and Kemper 1988)</td>
<td>$OCR = 0.37 \left( \frac{q_c - \sigma'_o}{\sigma'_o} \right)^{1.01}$</td>
<td>$\sigma_o$ and $\sigma'_o$ = total and effective stress, $q_c$ = cone resistance</td>
</tr>
<tr>
<td>CPT₃</td>
<td>(Abu-Farsakh 2004)</td>
<td>$OCR = 0.152 \left( \frac{q_t - \sigma_{vo}}{\sigma_{vo}} \right)$</td>
<td>$q_t$ = cone resistance, $\sigma_{vo}$ and $\sigma'_{vo}$ = total and effective stress</td>
</tr>
<tr>
<td>DMT</td>
<td>(Marchetti 1980)</td>
<td>$OCR = (0.5K_D)^{1.56}$</td>
<td>$K_D$ = Horizontal stress index</td>
</tr>
</tbody>
</table>

Using the data provided by Bullock from the SPT, CPT, and the DMT results to estimate the OCR produced a wide range of values. Table 6 provides the calculated data, which indicates that these relationships are estimations and may depend on other factors not included in the presented equations.
Table 6: Estimated OCR using Subsurface Exploration Test Data from Bullock (1999)

<table>
<thead>
<tr>
<th></th>
<th>SPT</th>
<th>CPT₁</th>
<th>CPT₂</th>
<th>CPT₃</th>
<th>DMT</th>
</tr>
</thead>
<tbody>
<tr>
<td>AUC 14m</td>
<td>2.6</td>
<td>23.6</td>
<td>17.0</td>
<td>6.7</td>
<td>-</td>
</tr>
<tr>
<td>AUC 17.5m</td>
<td>4.7</td>
<td>56.6</td>
<td>24.2</td>
<td>9.5</td>
<td>-</td>
</tr>
<tr>
<td>SBZ 17.8m</td>
<td>2.3</td>
<td>2.8</td>
<td>4.5</td>
<td>1.8</td>
<td>-</td>
</tr>
<tr>
<td>SBZ 21.2m</td>
<td>4.4</td>
<td>1433.1</td>
<td>54.0</td>
<td>21.1</td>
<td>-</td>
</tr>
<tr>
<td>VLW 12.3m</td>
<td>1.1</td>
<td>0.4</td>
<td>0.6</td>
<td>0.3</td>
<td>1.95</td>
</tr>
<tr>
<td>VLW 15m</td>
<td>1.0</td>
<td>0.6</td>
<td>1.0</td>
<td>0.5</td>
<td>1.85</td>
</tr>
</tbody>
</table>

The OCR values calculated from the CPT data in most cases appear to be overestimated when compared to SPT or the DMT results. Any results producing values lower than 1.0 will be taken as 1.0. The extreme value of the Seabreeze pile at a depth of 21.2 meters is a result of a very large cone resistance value that was also encountered by the Standard Penetration Test. Comparing the SPT and the CPT data, there appears to be a thin layer of hardened sands or gravels encountered near that depth. However, the nature of the CPT estimation equations relies heavily on the Cone resistance to calculate the OCR, but is most likely not indicative of the actual OCR of the soils at that depth. Typically the OCR value will be larger within the upper 10 to 15 feet of soil and then trend toward a normal consolidation behavior as depth increases. Plotting the OCR values calculated from the SPT data for each pile segment in the Bullock study reveals reasonable and expected trending.

Based on the values presented in Table 6 and the trending plots in Figure 13, along with the depths at which the pile segments for this study were investigated, it was decided to use the results from the SPT tests and the DMT tests for OCR for the remainder of the research.
3.2.2 Time-Dependent Friction Angle Determination

As previously shown, Bullock’s research provides the justification for the components of the model to predict the side shear based on the increased friction angle. Although there may be a question regarding the accuracy of the dynamic CAPWAP analysis, studies have shown a relatively good comparison to staqmamic or static load
testing as stated by Wang (2010b) and Fellenius (1998). The data will also be used to verify the validity of the model for use to estimate increased frictional resistance with time.

Now that the data has been evaluated to indicate that the friction angle change with time is an apparent mechanism behind the aging process in remolded clays, there are still many differences in the characteristics of clays that must be included within the model. Unfortunately, additional differences in soil properties for these six pile segments cannot be evaluated due to the lack of published subsurface investigative information. Thus, the model presented as Equation 13 includes coefficients $a_0$, $a_1$, and $a_2$ that must be determined for verification.

A detailed analysis was performed to determine the coefficients ($a_0$, $a_1$, $a_2$) using the horizontal stresses, the calculated friction angles, and the estimated OCRs from the data collected for each pile segment. The appropriate values were inserted into Equation 13 to calculate a “predicted” value for the time-dependent residual friction angle with an initial guess of 1.0 for each of the coefficients. The friction angles were only calculated for the values of $t$ after the pore pressure had stabilized. Then each value of $t$ is normalized by $t_o$, the time the pore pressure becomes stable, which was previously presented as Equation 13,

$$
\phi'_R(t) = a_0 \text{OCR}^{a_1} \left( \frac{\sigma_h}{\sigma_v} \right)^{a_2} \log \left( \frac{t}{t_o} \right) + \phi'_R.
$$

Then, using the least-squares method by minimizing the residuals (Equation 17), a basic iterative solver was implemented to determine the material coefficients for each pile segment,

$$
Res = \sum \left( \phi^m_R - \phi^p_R \right)^2.
$$
Those predicted friction angles were then compared to the actual measured friction angles and plotted to show the $R^2 = 0.956$, as shown in Figure 14.

![Predicted Residual Friction Angle using $a_0$, $a_1$, $a_2$ Calculated for Each Individual Pile Segment](image)

Figure 14: Predicted Residual Friction Angle using $a_0$, $a_1$, $a_2$ Calculated for Each Individual Pile Segment

Further analysis was performed in an attempt to develop these coefficients into constants which would allow the verification and incorporation into practice to be greatly simplified. However, averaging the values from the six pile segments for each of the coefficients created a scatter-plot that cannot be utilized. This is expected, considering the differences in material type. The pile segments for the individual piles were then combined due to the similarity of the descriptions provided by the boring logs. The next analytical step is to average the individual coefficients at each pile. This produced results
for the predicted residual friction angle to be much lower when compared to the measured friction angle, as shown in Figure 15.

![Figure 15: Predicted Residual Friction Angle using $a_0$, $a_1$, $a_2$ Calculated from the Average Values of Each Individual Pile Segment](image)

The third consideration for determining the coefficients was to utilize the solver with just one set of values for each pile, thus combining the iterative step to produce one best fit set of $a_0$, $a_1$, and $a_2$ values. This produces the best results based on the $R^2$ value of 0.957, seen in Figure 16. The values for all calculated coefficients are displayed in Table 7.
Figure 16: Predicted Residual Friction Angle using \( a_0, a_1, a_2 \) Determined for Each Pile

Table 7: Deterministic Results of Coefficients for Each Pile Location Based on Equation 13

<table>
<thead>
<tr>
<th></th>
<th>( a_0 )</th>
<th>( a_1 )</th>
<th>( a_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aucilla</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14m segment</td>
<td>6.5</td>
<td>-0.7</td>
<td>-4.5</td>
</tr>
<tr>
<td>17.5m segment</td>
<td>-2.1</td>
<td>2.4</td>
<td>-23.9</td>
</tr>
<tr>
<td>Averaged</td>
<td>2.2</td>
<td>0.8</td>
<td>-14.2</td>
</tr>
<tr>
<td>Combined</td>
<td>5.8</td>
<td>-2</td>
<td>-38.4</td>
</tr>
<tr>
<td>Vilano West</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.3m segment</td>
<td>2.7</td>
<td>2.6</td>
<td>0.05</td>
</tr>
<tr>
<td>15m segment</td>
<td>4.2</td>
<td>1.5</td>
<td>2.3</td>
</tr>
<tr>
<td>Averaged</td>
<td>3.5</td>
<td>2.1</td>
<td>1.2</td>
</tr>
<tr>
<td>Combined</td>
<td>3.9</td>
<td>2.1</td>
<td>0.04</td>
</tr>
<tr>
<td>Seabreeze</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17.8m segment</td>
<td>2.7</td>
<td>3.4</td>
<td>-2</td>
</tr>
<tr>
<td>21.2m segment</td>
<td>2.6</td>
<td>1.1</td>
<td>1.6</td>
</tr>
<tr>
<td>Averaged</td>
<td>3.2</td>
<td>3.2</td>
<td>-3.3</td>
</tr>
<tr>
<td>Combined</td>
<td>9.2</td>
<td>-7.9</td>
<td>10.5</td>
</tr>
</tbody>
</table>
The coefficients displayed are widely varying, especially with the $a_2$ coefficient associated with the horizontal effective stress. Upon further evaluation, when the OCR value is varied, the results show a decrease and then a subsequent convergence of remolded friction angle with time as shown in Figure 17. These results contradict what has been presented in literature which states that the increase in OCR will produce an increase in the amount of friction angle.

![Figure 17: Predicted Residual Friction Angle with Varied OCR, AUC 14m Depth](image)

The large range of values presented for the material properties as well as the trend of increased OCR actually decreasing the friction angle leads to a change in the model to remove the element related to the horizontal effective stress, thus eliminating the $a_2$ coefficient. The removal of the horizontal effective stress element seems logical considering the lack of evidence indicating the effect on the frictional resistance in a
steady-state condition. The model will now be evaluated as shown in Equation 18,

$$\phi'_R(t) = a_0 \text{OCR}^{a_1} \log\left(\frac{t}{t_o}\right) + \phi'_{R_0}. \quad (18)$$

The determinations of the coefficients for Equation 15 with the removal of the initial horizontal effective stress reveal the expected relationship between the increases in OCR and an increasing friction angle. The same pile segment, as shown in Figure 17, is again plotted using Equation 18 as the OCR is increased, as shown in Figure 18. The trend is similar to what is expected based on the current literature.

![Figure 18: Predicted Residual Friction Angle with Varied OCR, Aucilla 14m Depth](image)

The determined values for the coefficients are also more consistent within close range to the other segments of the same pile, which would be presumably similar material. The coefficients using Equation 18 are presented in Table 8.
Table 8: Deterministic Results of Coefficients for Each Pile Location Based on Equation 18

<table>
<thead>
<tr>
<th>Location</th>
<th>Segment</th>
<th>$a_o$</th>
<th>$a_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aucilla</td>
<td>14m</td>
<td>1.2</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>17.5m</td>
<td>3.8</td>
<td>0.014</td>
</tr>
<tr>
<td></td>
<td>Combined</td>
<td>3.0</td>
<td>1.7</td>
</tr>
<tr>
<td>Vilano West</td>
<td>12.3m</td>
<td>6.0</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>15m</td>
<td>2.4</td>
<td>2.8</td>
</tr>
<tr>
<td></td>
<td>Combined</td>
<td>5.8</td>
<td>1.4</td>
</tr>
<tr>
<td>Seabreeze</td>
<td>17.8m</td>
<td>1.3</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>21.2m</td>
<td>1.3</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>Combined</td>
<td>0.31</td>
<td>3.4</td>
</tr>
</tbody>
</table>

Comparing the calculated friction angles based on the material factors listed in Table 8 to the measured values from the field data reveals a similar comparison to the previous equation, as shown in Figures 19 and 20.

![Graph](image)

Figure 19: Predicted Residual Friction Angle using $a_o$ and $a_f$ Calculated for Each Individual Pile Segment
Figure 20: Predicted Residual Friction Angle using $a_\phi$ and $a_I$ Calculated for Each Pile

Based on the similarity of the results of the coefficients $a_\phi$ and $a_I$ within each pile and the fact that the combined values for each pile produce very similar results to the measured values, it can be assumed that the $a_\phi$ and $a_I$ coefficients are material factors. Further research should be performed on various soil types to provide validation of this assumption.

### 3.2.3 Pore Pressure Dissipation Determination

The data provided by Bullock (1999) show that the pore pressures reduced to a steady state with time after installation. An attempt to predict the time at which the dissipation was completed is not necessary. However, a brief investigation into verifying the existing prediction of the steady state pore pressure equations, as presented in Equation 14, was performed. The time factor, $T$, required to solve the time-dependent pore pressure equation includes knowledge of the horizontal consolidation coefficient, which is not known. With the data that is provided, the consolidation coefficient can be
back-calculated using Taylor series expansion. When this is performed, reasonable $c_h$ values are found when compared to published values of similar material type. However, these are values after the pile has been installed and the pore pressure has dissipated through heavily disturbed and remolded clay. It would seem reasonable that the consolidation coefficient would be significantly different after the pile has been installed due to the change in the soil fabric. Further research may be necessary to compare the post pile installation consolidation coefficient and then subsequent pore pressure relationship with the in situ consolidation coefficient.

### 3.2.4 Comparison between the Results from the Models and the Field Measurements

The prediction model developed herein to determine the remolded friction angle with time appears to provide a close approximation to the actual remolded friction angle of the clay once the effective stresses and pore pressures have stabilized. However, a more robust analysis to determine the accuracy of the friction angle model is to compare the results from the calculated unit frictional resistance based on the developed models to the actual side shear values determined from dynamic pile testing. After all, the frictional resistance is the value required for standard design for displacement friction piles in clay. Also, the process for predicting the ultimate capacity of a pile will include these steps, so the methodology is being developed.

This analysis requires the use of models that include mechanistic properties and responsive soil behavior. Calculating the unit frictional resistance of the pile segments based on the results from the friction angle requires the previously developed Equation
11, where knowledge of the amount of effective stress is a contributor.

\[ f_s(t) = (1 - \sin \phi'_R(t)) \cdot OCR \cdot \sin \phi'_R(\sigma_v - u(t)) \cdot \tan \phi'_R(t). \] (11)

Once the unit frictional resistance is determined for each pile segment at the times indicated by the research, they are used to calculate the unit side shear resistances using Equation 3. These values are then directly compared to the values presented from the load testing of the piles at each specific time and segment. The following Figures 21-26 are the comparison plots for each pile segment with the time after the pore pressure has dissipated. The abscissa values are time plotted in a logarithmic scale and the ordinate values are side shear, \( f_s \), plotted in arithmetic scale. The values represented on the plots are a) the measured side shear from research, b) the calculated side shear using the back calculated friction angle from research, c) the predicted side shear using the developed prediction model for determining the friction angle, and d) the predicted side shear using the Denver and Skov prediction method with the \( A_o \) and \( t_o \) values for each pile segment calculated by using the results from Bullock (1999).
Figure 21: Frictional Side Shear Resistance Comparisons, Aucilla at 14m Depth

Figure 22: Frictional Side Shear Resistance Comparisons, Aucilla at 17.5m Depth
Figure 23: Frictional Side Shear Resistance Comparisons, Vilano West at 17.8m Depth

Figure 24: Frictional Side Shear Resistance Comparisons, Vilano West at 21.2m Depth
Figure 25: Frictional Side Shear Resistance Comparisons, Seabreeze at 12.3m Depth

Figure 26: Frictional Side Shear Resistance Comparisons, Seabreeze at 15m Depth

The trend of the results from the prediction model in Figures 21-26 show a close comparison with the frictional resistance values observed from the field load testing. The
purpose of showing the comparison between the frictional resistance calculated by the predicted friction angle and the frictional resistance calculated using the back-calculated friction angle with the measured frictional side shear is to investigate the potential errors involved in constitutive equations between field measurements and friction angle.

The results from the Denver and Skov prediction method show a mostly inconsistent comparison with the measured side shear resistance, with most cases showing an extremely conservative trend. The setup factor, $A_o$, used in the calculations for the Denver and Skov results are the specific values back-calculated for each pile segment. When using the single recommended value of 0.2, as presented by Bullock (1999), the results are even more conservative, as shown in Figure 29.

An overall comparison between the predicted frictional resistance and the observed frictional side shear from the field testing is presented in Figure 27 with an $R^2$ value of 0.922 when compared to the linear regression equation, which is very similar to the 45 degree line.
Figure 27: Frictional Resistance Comparison, Predicted vs. Measured

The same comparison using the Denver and Skov method is presented in Figures 28 and 29. Figure 28 is the comparison with the setup factor specific to each pile segment and Figure 29 is the comparison using the overall recommended setup factor of 0.2 for all segments.
Figure 28: Frictional Resistance Comparison, Denver and Skov using Individual Setup Factors vs. Measured

Figure 29: Frictional Resistance Comparison, Denver and Skov using Setup Factor = 0.2 vs. Measured
The results from the prediction model compare very well with both the calculated values for the skin friction using the calculated time-dependent frictional angle as well as the direct measured values for the skin friction. Not only do the results produce a well defined linear regression equation with an impressively high $R^2$ value, the linear regression is very close to the 45 degree line to show a high level of accuracy when compared to the actual field data. In every case, the Skov and Denver model proved to be inconsistent and mostly conservative. When the recommended value of 0.2 is used for the setup factor using the Denver and Skov method, the results are even more conservative; however, the linear regression for this data does produce a fair $R^2$ value of 0.77. The values are quite inconsistent with the 45 degree line which shows a requirement for curve fitting to improve accuracy.

### 3.3 Summary and Conclusions

Based on previously published research and the configuration of constitutive equations, a mechanistically-determined prediction model for pile setup is established and is related specifically to the remolded friction angle increase with time. This frictional resistance prediction model is based upon the determination of the remolded friction angle at specific times and accounts for lateral earth pressure and over-consolidation ratio once the induced excess pore water pressure has stabilized.

The model was then further developed using field test data from instrumented piles presented in a Ph.D. thesis in Florida (Bullock, 1999). The study includes data to determine the hydrostatic stresses after pile installation, frictional resistance values over a long period of time after installation, and a small amount of in situ soil data prior to pile installation. The OCR was determined using rough correlated estimates based on
subsurface investigations. Material factor coefficients had to be produced using iterative back-analysis due to the lack of material properties provided by the field data publication.

The results of the prediction model developed prove to be quite accurate when compared to measured restrike pile test results which produce a linear regression equation with an $R^2$ value of 0.922. When comparing the measured results from pile load testing to the most widely used method of setup prediction, the new deterministic model appears to be much more accurate. The model requires knowledge of the friction angle of the soil at the time when the pore pressure becomes stable. Currently, this value must be measured or assumed.

Further experimentation and modeling is required to aid in the determination of the soil friction angle over a period of time after the disturbance of the soil fabric once the pile has been installed. Also, it would appear that the material coefficients presented in the model require a methodology that is soil and site specific and that can be developed from subsurface investigation prior to pile installation. Overall, the model developed herein is the framework for a new method to predict the ultimate frictional resistance of a pile using preconstruction site investigative testing results, whether in the laboratory or the field or both. The following chapter attempts to address the experimental remolded friction angle increase over time.
CHAPTER 4

EXPERIMENTAL INVESTIGATION OF THE LONG-TERM RESIDUAL INTERFACE FRICTION ANGLE INCREASE

The model presented in the previous chapter requires the determination of the friction angle between the soil and pile at various times throughout the aging process to successfully predict the unit frictional resistances over time. This includes the friction angle at the approximate time when the excess pore water pressure has fully dissipated. The field data used to verify the approach to the model indicates an increase in friction angle between the soil and a structural material under constant effective stress as time passes. It is not feasible to install and test fully instrumented piles months in advance of most construction projects to aid in the determination of the aging effects on each particular site. As with many geotechnical engineering problems, a laboratory analysis of a site obtained soil specimen can aid in the determination of in-situ characteristics which are utilized in design of foundation systems. Long-term laboratory testing research is not currently available to show this friction angle increase with time. However, the ability to simulate and determine the effect of aging on soil/structure interface would be a significant contribution to the prediction of pile setup. The results from laboratory testing can aid in the verification of the research used to develop the prediction model in the previous chapter. This type of experimental testing can also be used to develop the material parameters for specific soil types within the deterministic model. A process is
developed to determine the time-dependent frictional behavior between clay and concrete in a laboratory to provide designers with a method to utilize models to predict long-term pile frictional resistance.

4.1 Selection of Testing Apparatus

Investigating the frictional relationship between soil and a construction material has been studied using various types of apparatus and with many types of material. However, there has been very little investigation conducted into the long-term frictional behavior of clays and their interaction with structures. Laboratory friction testing is typically classified into either a direct shear type (such as shear box, simple shear, or torsional shear) or indirect shear type (such as Triaxial shear). The direct shear type tests are capable of the direct measurement of both the normal and shear stresses at the interface, but lack a controlled moisture environment in which the specimen can be tested. The indirect shear test can place the specimen in very specific conditions to better simulate in-situ environments, yet require that many assumptions be made to determine an indirect measurement of the interface shear stress. The research described in the previous chapter requires the ability to vary the preconsolidation pressure in order to alter the OCR, the ability to implement different types of construction materials (steel, concrete, timber, etc) to interact with the soil, and the ability to apply stresses over a long period of time prior to the acquisition of the shearing results. For the purpose of this research, the ability to directly measure of the interface shear stress is considered to be more important than the ability to control the moisture condition of the soil test sample. Plus, there are techniques that can be performed to simulate the appropriate pore water pressures within the specimens that are described. For the above reasons, the direct shear
type apparatus was chosen as the most appropriate geotechnical laboratory equipment for this research program.

The direct shear device permits the placement of a soil specimen directly onto the surface of a construction material, such as steel or concrete. The most common of the shear tests is the shear box test, as illustrated in Figure 30. In this test, the soil specimen is placed within a container in direct contact with the interface material being investigated. A force orthogonal to the interface plane can be applied through a porous plate to simulate various stress states at the interface. The failure plane is predetermined to be the interface surface and the stress and displacement during the shearing is captured as indicated in Figure 30.

![Figure 30: Shear Box Device](image-url)
The shear box test is simple to setup and easily allows the residual effects to be determined, as the soil specimen can be moved back and forth with measurements of force and displacement in both directions.

The simple shear device replaces the conventional box surrounding the soil with small rings that offer laterally independent displacement, as illustrated in Figure 31.

![Figure 31: Simple Shear Device](image)

The use of rings allows the measurement of both the sliding displacement (movement between the clay and the concrete) and deformation displacement (movement within the clay) to be determined from the total shear displacement. Also, the critical failure plane can be more easily located, rather than a predetermined failure plane along the interface surface as with the shear box device. However, due to the nature of the disturbance of the sample during failure, residual shear measurements are much more difficult to determine.
A number of studies have investigated the comparative results between the shear box and the simple shear device to show the peak shear strength of soil to be typically higher in the shear box due to the predetermined shear failure plane (Shakir and Zhu, 2009; Subba Rao et al., 2000; Tsubakihara and Kishida, 1993). The predetermined shear plane has very little relevance to the interface testing because the focus of research is specifically related to this predetermined shear surface between the soil and the concrete. There is also criticism that the relationship between stress and strain is non-homogeneous within the shear box as compared to the simple shear device leading to errors in stress determination. Miller et al. (2006) reported on the results of finite element modeling that showed that very little error in stress determination occurred due to the difference in the stress-strain state of the shear box test compared to a simple shear test. Also, this problem is only relevant from a continuum mechanics perspective and does not contribute to significant error in the measurement of the stresses on the interface surface.

The nature of this research requires the residual frictional behavior of soil specimens interacting with a construction material after time dependent consolidation. While the simple shear device will allow a thorough investigation into the behavior of the specimen displacement, the residual interfacial friction angle is the main focus of this research. A more sophisticated shear box apparatus has been used in other research to allow the measurement of pore pressure within the sample during testing, however the goal of the research enclosed is to develop a methodology that can be implemented within standard testing laboratories that do not possess research capable equipment. A conventional shear box apparatus is most applicable and will be utilized for this experimentation.
4.2 Testing Apparatus

The standard configuration of a shear box type device includes a shear box that is separated through the center to allow a soil specimen to be sheared along a plane orthogonal to the applied normal force, as shown in Figure 32.

![Figure 32: Conventional Shear Box for Direct Shear Testing of Soil](image)

For interface testing, the bottom half of the shear box will be replaced with a construction material that the soil specimen will interact with, as seen in Figure 33 with a small section of concrete.
Figure 33: Top Half of Shear Box on Concrete

Once the soil sample has been placed within the top half of the shear box, the box and concrete together are submerged into the water bath of the ELE direct shear machine and attached by two locking bolts shown in Figures 34 and 35.

Figure 34: Specimen Placed in the Direct Shear Machine
Figure 35: Direct Shear Machine

The machine is designed to apply continuous strain-controlled horizontal movement to the lower half of the shear box containing the concrete specimen. Horizontal displacement is controlled using a microprocessor-controlled drive system with strain rate capabilities between 0.00001 mm and 9.99999 mm per minute. The top half of the shear box, which includes the soil specimen, is held stationary thus creating the frictional response along the interface, as shown in Figure 36.
Figure 36: Schematic of Horizontal Load Application to Specimen

The normal force is applied by a hanging weight lever system with a 10:1 lever ratio. The reactive force produced by the interface friction is captured by a 2,000 pound capacity S-type load cell. The vertical movement of the specimen during consolidation and shearing as well as the tangential movement between the specimen and the concrete is captured by two displacement Linear Voltage Differential Transformer (LVDT) transducers. The load cell and the transducers are connected to an ELE Autonomous Data Acquisition Unit (ADU) for data acquisition. The data is compiled and analyzed using the commercially available software ELE Datasystems 7 (DS7).

In an effort to provide the most time efficient testing results, the samples were transferred from the direct shear machine to 1-dimensional consolidation devices during the long-term consolidation. This allowed the direct shear machines to be utilized for testing short-term consolidated samples during the long consolidation wait periods of other samples. The consolidation devices were manufactured by Geotest Instrument Corp. and offer the same lever ratio as the direct shear devices to allow similar
consolidation conditions. The vertical deformation was recorded manually from the
digital indicators on each device, as shown in Figure 37.

![1-Dimensional Consolidation Devices with Interface Samples Submerged](image)

**Figure 37: 1-Dimensional Consolidation Devices with Interface Samples Submerged**

4.3 Soil and Material Tested

4.3.1 Construction Material - Concrete

Of the common pile materials; steel, timber, and concrete, concrete was chosen as
the construction material for this testing program. One of the primary reasons for this
selection was that field data utilized in the analysis of the developed model included
square-shaped concrete piles. Also, concrete was readily available to the author and
samples could be fabricated to specific standards.

The sizes of the concrete samples were made to match the lower half of the shear
box, as described in Section 4.2, which were 130 mm x 130 mm square by 21 mm thick.
The samples were made in a wooden mold that was sanded using 240 grit sand paper on an electronic sanding machine to provide the smoothest surface possible. Roughness measurements were not obtained, as piles are not typically subjected to roughness requirements. However, in written correspondence with a pile manufacturer in southern Louisiana, square-shaped concrete piles are manufactured in steel molds on three sides with the top or fourth side being hand-troweled (Price III 2009). It is feasible to assume that the surface roughness of a well sanded wooden mold would be comparable to the surface roughness experienced from a precast concrete pile.

The concrete mixture contained a simple two parts Portland cement, one part fine aggregate, and one part coarse aggregate. Due to the relatively thin height of the required concrete sample, the mixture contained a class A pea-gravel for the coarse aggregate. Strength parameters were not obtained or deemed relevant to the research, as the applied load was extremely small compared with the strength of concrete. Threaded inserts were attached to the mold prior to pouring the concrete shown in Figure 38. These inserts were used to receive the clamping screws to attach the top half of the shear box to the concrete during the consolidation phase of the experimentation. The location of the inserts was dictated by the predrilled holes on the top half of the shear boxes.
4.3.2 Clay Samples Tested

The soil utilized for the experimentation in this research was provided by the Louisiana Department of Transportation and Development (La DOTD) Materials Lab in Baton Rouge. The DOTD provided all remaining soil specimens from two exploratory bore holes from a highway project in Iberia Parish and one bore hole from a bridge project in St. Bernard Parish, both located in southeastern Louisiana. The southern Louisiana subsurface soils consist mainly of soft saturated clays and silty clays with occasional lenses of silt and sand (Titi et al., 1999). The Louisiana “gray clays”, as they are referred to, are predominant throughout the Mississippi Delta region and remain quite consistent in nature and properties, with natural moisture contents ranging from 40% to 75%, liquid limits between 60% and 85% and PI’s in the middle 50’s. The material used in the experimentation was from samples 17-19 on the boring log presented in Appendix
A and has a calculated specific gravity of 2.68 and a clay fraction of 99% with an unconsolidated-undrained shear strength ranging between 4 and 10 tons per square foot under loads consistent with depths sampled.

Care was taken to preserve the consistency of the samples to prevent further disturbances. However, because these samples were stored specimens from previously tested samples, slight disturbances to the structure of the soil can be assumed. The nature of the experimentation requires a residual state of the material, so this disturbance was not judged to have a significant bearing on the accuracy of the results. This will be further discussed in the section of this chapter regarding the testing procedure.

### 4.4 Testing Procedure

The experimental testing required for this research is time consuming, due to the nature of the aging process. The maximum aging time selected was for a period of 90 days. Other intermediate time-intervals for shear testing were determined to be 1 day, 7 days, 14 days, and 30 days. It was anticipated that over a 90-day testing period with the intermediate time intervals selected, that a clear increasing trend attributable to soil aging would be observed in the measured friction angle.

#### 4.4.1 Soil Preparation

Soil specimens were trimmed to fit into the upper portion of the shear box using a manufacturer supplied cutting ring and porous stones for a spacer to achieve the appropriate height. On average, the specimens were approximately 10 mm in height and 63.5 mm diameter. The weight of each specimen was recorded and varied depending on the moisture content of the soil. Once the measurements were recorded, the specimens were placed within the upper half of the shear box and then attached to the concrete
plates using two stainless steel screws passing through the shear box and threaded into
the inserts embedded within the concrete plates. This is the initial moment when the soil
and the concrete create an interface.

The testing of the interface shear should attempt to mimic the soil condition that
would have been encountered had piles been driven in the location where the soil samples
were obtained. In this case, the soil samples were obtained beneath the water table, thus
to be representative of *in-situ* soil conditions, test samples should be saturated. The soil
was relatively well sealed for long-term storage wrapped in multiple layers of plastic, yet
some moisture had left the material prior to testing. Ideally, the shear box would be
equipped to fully saturate the samples. However, the direct shear devices utilized do not
allow a controlled saturation process. As an alternative, the samples were submerged in
water under a small stress to allow the material to absorb as much moisture as possible
prior to the aging and shear tests. The samples were placed into either a consolidation
device or the direct shear device with a small amount of load on the lever arm, equaling
135 gm. This weight would equate to approximately 4 kPa of normal stress applied to
the sample. The reason for the small load is to keep the lever arm in contact with the
sample for deformation measurements while water is added to the water bath. As the soil
absorbed water, vertical deformation was monitored. Absorption was considered
complete when the vertical deformation equalized, which required up to 72 hours for
some samples. The moisture content of the specimens should be similar to the moisture
content determined during the initial testing of the boring log samples. A comparison of
all moisture content values will be discussed later in this chapter.
Once the specimen deformation reached equilibrium under the small submerged loading, the pre-consolidation pressure was applied to each sample. A value of 43.5 kPa was used as the baseline for normal stress for all samples tested during this experimentation. This pressure was selected because of the convenience of the size and quantity of the existing weights in the laboratory. This pressure would simulate a depth of approximately 5.6 meters of fully saturated overburden pressure of a homogeneous soil with a density of 1778 kg/m$^3$. OCR values for the soil samples were chosen to be 1, 3, and 6 based on the results presented by Schmertmann (1991) where lab testing was performed to compare the frictional behavior between a similar soil having differing OCRs of 1 and 4, as previously shown in Figure 8. Thus, specimens that were slated to have an OCR of 1 would have a pre-consolidation pressure equal to the consolidation pressure of 43.5 kPa. Specimens slated to have an OCR of 3 and 6 would have a pre-consolidation pressure of 130.5 kPa and 261 kPa, respectively. Vertical deformation was measured and recorded once the pre-consolidation pressure was applied to the specimens. This pressure was applied to the specimens until vertical deformation equalized, which typically required 24 to 36 hours. The pre-consolidation pressure was not removed prior to 24 hours, even when vertical deformation had already achieved equilibrium.

4.4.2 Interface Shearing

Due to the many variations of interface situations encountered between soils and construction materials, experimental testing has been largely developed on a case-by-case basis. An attempt was made to develop a procedure for this particular research that followed the existing ASTM D3080 standard for direct shear testing of soils. Deviations from the standard procedure were made as necessary to acquire specific aging data.
It was important that the laboratory procedure developed for this time-dependent residual friction angle determination simulated the installation of a driven pile. Pile installation creates a heavily disturbed fabric within the structure of the surrounding soils. Accordingly, a decision was made to shear the soil specimen along the concrete plate immediately after the OCR pre-consolidation pressure stage was complete to simulate pile installation. This would generate the residual shearing data required for comparison with the aging shear results. Once the vertical deformation reached an equilibrium state, the sample was then installed into the direct shear apparatus and a normal pressure of 43.5 kPa was applied. For specimens with an OCR of 3 and 6, the samples would be allowed to rebound until measured deformation values were static.

The determination of the rate of horizontal displacement during shearing would follow the same procedure as indicated by the ASTM D3080 standard for direct shear of testing soil. The vertical deformation results from the specimen with an OCR of 1 were plotted for the purposes of determining the time for 90% primary consolidation, t_{90}. The t_{90} value is then used, according the ASTM D3080 (ASTM International 2004), to calculate the rate of shearing. The rate of shearing was determined to be 0.26 mm/min. An initial shearing rate to of 0.52 mm/min, double the calculated rate, was chosen as an attempt to simulate the installation of a pile. It is believed that a faster rate would increase the pore pressure within the soil due to the disturbances generated during shearing. The direct shear apparatus utilized in the experimentation contains a feature to return the water bath to the precise location as before the shearing occurred. This allows residual testing to be performed with little effort in relocating the sample to the exact location on the concrete plate. The horizontal movement of the water bath with the top
half of the shear box remaining stationary and measuring the resistance force occurring along the interface occurred until a constant force value was obtained. Then the sample was moved back to the original location on the concrete plate and the test movement was started again. This forward and reverse movement of the soil along the concrete was performed until the residual forces recorded were constant. As expected the initial shearing force was higher than the residual forces. A typical shear stress versus horizontal displacement relationship during this residual testing is shown in Figure 39.

![Figure 39: Residual Shear Stress Versus Displacement, Sample SA3B](image)

After the residual shear testing was completed for each sample, the predetermined consolidation pressure was applied to the sample for specified periods of time-increments to create an aging effect between the soil and the concrete. As previously stated, the experimentation requires similar materials to have differing OCR and consolidation times in order to produce the expected variation of the friction angle at the interface. This
requires a large amount of material if each test was to be performed individually. In an attempt to be more time-efficient, a testing program was established to utilize the same soil samples for multiple timed tests. The original testing program was organized with the same soil sample to be placed on the same concrete sample for each time test from one day up to 30 days. All of this testing could be accomplished within the time frame of the 90-day consolidation with just six soil samples, thus producing all test results within 90 days after the first residual shearing was performed, as shown in Table 9.

**Table 9: Interface Age Testing Program**

<table>
<thead>
<tr>
<th>Material A</th>
<th>OCR = 1</th>
<th>OCR = 3</th>
<th>OCR = 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>t = 1 d</td>
<td>Specimen A1</td>
<td>S A2</td>
<td>S A3</td>
</tr>
<tr>
<td>t = 7 d</td>
<td>S A1B</td>
<td>S A2B</td>
<td>S A3B</td>
</tr>
<tr>
<td>t = 14 d</td>
<td>S A1C</td>
<td>S A2C</td>
<td>S A3C</td>
</tr>
<tr>
<td>t = 30 d</td>
<td>S A1D</td>
<td>S A2D</td>
<td>S A3D</td>
</tr>
<tr>
<td>t = 90 d</td>
<td>S A4</td>
<td>S A5</td>
<td>S A6</td>
</tr>
<tr>
<td>Total time:</td>
<td>90 days</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

At the completion of the consolidation time specified for each sample, an additional shear test was performed. This shear test was a standard direct shear with no additional back and forth motion as previously described. The assumption for performing only one shear test was that the time interval during the consolidation after the residual testing would simulate the time of aging of an in-place driven pile. Thus, creating a similar effect as with a pile that has been load tested after aging has occurred. Then, the increased frictional behavior, or more specifically the residual interface friction angle, of the surrounding soils can be determined and compared with the previously
obtained residual friction angle. The rate of horizontal displacement for the time-
dependent shearing test was the previously determined 0.26 mm/min.

After the shearing test, the specimen was removed from the concrete using a
horizontal sliding motion, as specified by the ASTM D3080 standard to visual observe
the interface failure conditions. The soil specimen was then removed from the shear box
and the moisture content was obtained.

A simplified step-by-step procedure of the interface shear testing program is as follows:

1. Obtain an undisturbed soil specimen that is approximately 10 mm high and 63.5
   mm in diameter; or the measurements of the top half of the shear box used.
   Measure and record the diameter, height, and weight of the sample.
2. Obtain the moisture content of the trimmings from the specimen.
3. Install an undisturbed soil specimen into the top half of the shear box and then
   onto the concrete plate, tightening the screws of the shear box to the concrete.
4. Install the sample (concrete and soil specimen) into a consolidation machine (or
direct shear machine), apply a small preload to the lever arm (something less than
200 g), install a deformation measurement device, and take an initial reading.
5. Submerge the sample and monitor the deformation until equilibrium; at least 24
   hours.
6. Apply a predetermined normal load to the sample to be the pre-consolidation
   pressure depending on the OCR specified for the specimen. This could be equal
to the vertical effective stress of the depth where the soil was obtained multiplied
by the OCR required for that sample. Monitor the deformation until equilibrium or at least 24 hours.

7. Apply the predetermined stress load to the sample. Monitor the deformation until equilibrium.

8. Install the sample into the direct shear machine and shear the specimen on the concrete at twice the shearing speed calculated in accordance with the ASTM standards from the consolidation results from the specimen with an OCR equal to one. Perform residual shearing of the soil on the concrete until the force measurement has equalized and a constant residual force measurement is obtained.

9. Return the specimen to the original location on the concrete plate and reinstall the locking screws.

10. Reapply the predetermined stress load to the sample for the specified period of time, monitoring the deformation of the sample throughout the time.

   a. Consolidation times will be 1 day, 7 days, 14 days, 30 days, 90 days.

11. Shear the specimen on the concrete at the shearing speed calculated in accordance with the ASTM standards from the consolidation results from the specimen with an OCR equal to one.

12. After the final shearing test is completed, remove the sample in accordance with the ASTM D3080 to visually inspect the interface failure zone. Obtain the moisture content of the soil specimen.

4.4.3 Internal Soil Aging Shear

The research regarding the laboratory frictional behavior between soils and construction materials presented in the literature review shows two different failure
mechanisms occurring at the interface depending on the roughness, soil moisture, or other unknown reasons (Shakir and Zhu., 2009; Subba Rao et al., 2000; Tsubakihara and Kishida, 1993). The interface shearing procedure described in this chapter only includes the direct shearing of the interface with concrete. Using the same soils, the author performed a similar procedure without the concrete and using the interface within the soil sample, very similar to the standard direct/residual shearing tests described by the ASTM D3080 standard. Once the results are obtained from this testing, the results can be compared with the interface testing with the concrete to determine if the internal friction angle can be related to the interface friction angle and contribute to a better understanding of the failure mechanism.

The procedure for the internal soil shear testing is very similar to the procedure described in the previous section. The main differences will be the exclusion of the concrete plate and the use of the entire shear box with half of the height of the soil specimen in the upper half and the half of the height of the soil in the lower half of the shear box. This creates a shearing plane within the center of the soil specimen. It should be noted that the consolidation of the specimen in this arrangement will be slightly different than the previous section due to the specimen having twice the height. Also, there is double drainage for the specimen, as there are brass porous stones installed on both the top and the bottom of the specimen. The drainage should not alter the effects of the testing, as the concrete in the previous section acts as a porous medium to allow drainage. The shearing rate calculated from the consolidation results, in accordance with the ASTM procedure, was 0.84 mm/min. Due to time constraints, the last shearing tests
was performed after 60 days of consolidation after the residual shearing, rather than the 90 days performed with the interface testing.

A simplified step-by-step procedure of the internal soil shear testing program is as follows:

1. Obtain an undisturbed soil specimen that is approximately 20 mm high and 63.5 mm in diameter. Measure and record the diameter, height, and weight of the sample.

2. Obtain the moisture content of the trimmings from the specimen.

3. Install an undisturbed soil specimen into the assembled shear box.

4. Install the shear box into a consolidation machine (or direct shear machine), apply a small preload to the lever arm (something less than 200 g), install a deformation measurement device, and take an initial reading.

5. Submerge the sample and monitor the deformation until equilibrium; at least 24 hours.

6. Apply a predetermined normal load to the sample to be the pre-consolidation pressure depending on the OCR specified for the specimen. This could be equal to the vertical effective stress at the depth the soil was obtained multiplied by the OCR required for that sample. Monitor the deformation until equilibrium or at least 24 hours.

7. Apply the predetermined stress load to the sample. Monitor the deformation until equilibrium.

8. Install the sample into the direct shear machine and shear at twice the shearing speed calculated in accordance with the ASTM standards from the consolidation
results from the specimen with an OCR equal to one. Perform residual shearing of the soil until the force measurement has equalized and a constant residual force measurement is obtained.

9. Return the specimen to the original location and reinstall the locking screws.

10. Reapply the predetermined stress load to the sample for the specified period of time, monitoring the deformation of the sample throughout the time.

   a. Consolidation times will be 1 day, 7 days, 14 days, 30 days, 90 days.

11. Shear the specimen at the shearing speed calculated in accordance with the ASTM standards from the consolidation results from the specimen with an OCR equal to one.

12. After the final shearing test is completed, remove the sample in accordance with ASTM D3080 to visually inspect the failure zone. Obtain the moisture content of the soil specimen.

4.5 Experimental Results

The data from the consolidation and shear testing was collected by commercially available software designed to follow the ASTM standard procedures. Modifications to the testing procedure did not alter the acquisition of the data, other than the addition of manual recording of vertical displacement during long-term consolidation. The reports from the experimental testing are presented in Appendix B.

4.5.1 Swelling and Consolidation Results

As previously stated, the soil samples provided by the Louisiana DOTD were sealed to prevent moisture loss, but it was apparent through visual inspection that some moisture loss had occurred from the time the samples were collected. During the initial
submerging stage of each sample, the soil absorbed moisture under very small loading in
an attempt to simulate the in situ conditions. The trimmings collected from the samples
averaged 35.6% moisture content which is lower than the boring logs indicate, where the
average moisture content of samples C17 through C20 was 54%. The specimens were
able to absorb water until the readings stabilized, which required approximately 7 days.
The average final moisture content of all samples tested was 47.6%, which is a 12%
increase during the submerged time. The specimens increased in vertical height an
average of 3.5% during this 7-day moisture conditioning period. All values listed include
both the specimens used for interface testing and internal soil shear testing.

The samples were then subjected to the pre-consolidation pressures as indicated
by the testing schedule in Table 9. The samples subjected to the consolidation pressure
with an OCR equaling one had an overall increase in sample height of 0.92%, indicating
the consolidation pressure did not provide enough load to remove the displacement
gained during the swelling. The samples with OCR of three and six had an expected
decrease in vertical height of 5.4% and 7.7%, respectively.

The consolidation plot generated from the specimen with an OCR of one was
used for the determination of the loading rate of shearing, according to the ASTM D3080
standard. A consolidation plot from sample SA1C is presented in Figure 40, and is
typical of the vertical displacement versus time plots of the other specimen exposed to the
same consolidation pressure.
4.5.2 Residual Shear Strength Test Results

The residual strength gain effects from long-term soil structure changes have been shown to occur without an increase in the cohesion of the soil, as visually shown in Figures 6 and 8. Based on this information, the direct shear Mohr-Coulomb failure criteria can be simplified to solve for the residual friction angle as stated in Equation 19 (Das 2007),

\[ \phi'_R = \tan^{-1} \left( \frac{\tau_R}{\sigma'_R} \right) \]  

(19)

where \( \tau_R \) is the residual shear stress and \( \sigma'_R \) is the effective normal stress. The equation presented allows for quick determination of the friction angle from each test without the need to utilize additional soil samples to increase the normal stress and plot the complete Mohr-Coulomb failure envelope.
The shear tests were all performed using the same effective normal stress of 43.5 kPa. The residual stresses were first tested and calculated with consistent values regardless of the OCR resulting in an average value of 21 kPa with a standard deviation of 2.4 kPa. The residual strength value of 20.8 kPa corresponds to a friction angle of 25.1 degrees. The Plasticity Index as indicated by the boring log in Appendix A is approximately 52. Holtz and Kovacs (1981) show a comprehensive compilation of 3 studies that compare Plasticity Index with the associated drained friction angle by Triaxial test. The results from the residual friction angle testing herein fall within the 1% standard deviation of that correlation.

4.5.2.1 Interface Residual Shear Testing

Shear tests were then performed for each sample after the specified aging period. The shear resistance between the clay and the concrete increased over time from the residual shear stress for all OCR values tested as seen in Figures 41-43.
Figure 41: Shear Stress Versus Shear Strain for All Samples with OCR = 1

Figure 42: Shear Stress Versus Shear Strain for All Samples with OCR = 3
The data presented in Figures 41-43 provide a unique laboratory depiction of an increase in shear strength occurring between a structural material and clay. As the time passes during these lab tests, the samples remained under constant environmental conditions. So the only feasible explanation to the increase in shear strength over time is the rearrangement of the micro-structural elements that were disturbed during the initial shear testing, which is consistent with the hypothesis presented by Schmertmann (1981).

Another observation of the interface aging test data is the influence of the stress history of the soil on the amount of shear strength. Figure 45 shows that as the OCR on the soil increases, so does the shear stress. The trend was generally similar for all time intervals tested, as Figure 44 shows the increase after 90 days of aging. Again, this evidence is in agreement with the previously stated research that provided the basis for the prediction model developed in Chapter 3.
Figure 44: Shear Stress Versus Shear Strain at 90 Days of Aging at Various OCR Values

The shear stress presented in the Figures 41 – 43 can then be used to calculate the friction angles for each time interval for the three OCR test programs. Table 10 presents the results from all shear testing and the calculated residual friction angles corresponding to each long-term shear test.
Table 10: Increased Friction Angle by Direct Shear Age Testing

<table>
<thead>
<tr>
<th>Sample</th>
<th>Initial Residual Friction angle</th>
<th>Time Consolidating (days)</th>
<th>Time-Dependent Friction Angle</th>
<th>Friction Angle Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>OCR = 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA1 A</td>
<td>25.3</td>
<td>1</td>
<td>26.1</td>
<td>0.8</td>
</tr>
<tr>
<td>SA1 B</td>
<td>26.1</td>
<td>7</td>
<td>27.9</td>
<td>1.8</td>
</tr>
<tr>
<td>SA1 C</td>
<td>21.8</td>
<td>14</td>
<td>27.9</td>
<td>6.1</td>
</tr>
<tr>
<td>SA1 D</td>
<td>24.2</td>
<td>30</td>
<td>38.2</td>
<td>14.0</td>
</tr>
<tr>
<td>SA4</td>
<td>26.1</td>
<td>90</td>
<td>44.0</td>
<td>17.9</td>
</tr>
<tr>
<td><strong>OCR = 3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA2 A</td>
<td>21.7</td>
<td>1</td>
<td>26.1</td>
<td>0.0</td>
</tr>
<tr>
<td>SA2 B</td>
<td>14.3</td>
<td>7</td>
<td>23.7</td>
<td>5.9</td>
</tr>
<tr>
<td>SA2 C</td>
<td>19.9</td>
<td>14</td>
<td>32.2</td>
<td>8.1</td>
</tr>
<tr>
<td>SA2 D</td>
<td>19.9</td>
<td>30</td>
<td>37.4</td>
<td>13.3</td>
</tr>
<tr>
<td>SA5</td>
<td>21.3</td>
<td>90</td>
<td>45.3</td>
<td>19.7</td>
</tr>
<tr>
<td><strong>OCR = 6</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA3 A</td>
<td>21.4</td>
<td>1</td>
<td>27.9</td>
<td>2.2</td>
</tr>
<tr>
<td>SA3 B</td>
<td>23.5</td>
<td>7</td>
<td>33.0</td>
<td>5.1</td>
</tr>
<tr>
<td>SA3 C</td>
<td>23.1</td>
<td>14</td>
<td>35.8</td>
<td>8.3</td>
</tr>
<tr>
<td>SA3 D</td>
<td>21.4</td>
<td>30</td>
<td>40.0</td>
<td>14.3</td>
</tr>
<tr>
<td>SA6</td>
<td>23.1</td>
<td>90</td>
<td>48.7</td>
<td>21.3</td>
</tr>
</tbody>
</table>

Plotting these results in Figure 45 shows an overall increase of friction angle that is dependent on the amount of OCR applied to the sample after 90 days of consolidation. This is consistent with the previous study by Schmertmann (1991).
Figure 45: Friction Angle Increase Over Time with Varying OCR

Plotting the friction angle after the consolidation has been completed shows the value of the friction angle is also increased with a higher OCR, as shown in Figure 46.
The visual observation along the interface after the shearing testing was performed produced an interesting occurrence. The shear testing at 1, 7, and 14 days showed a sliding failure, meaning the soil slide directly at the concrete indicating the critical failure plane to be along the interface. The shear tests at 30 and 90 days showed approximately 1.5 to 2.5 mm of soil attached to the concrete at the interface location indicating a critical failure plane within the soil and not at the interface, as shown in Figure 47. Based on this observation, it would seem reasonable to state that the shear strength at the interface is larger than the shear strength of the surrounding soil.
4.5.2.2 Internal Residual Shear Testing

Internal residual shear testing was performed according to the previously described testing program, with some limitations. First, as mentioned, the longest aging test was 60 days rather than the 90 days performed on the interface testing due to time constraints. Also, the amount of material was limited, which resulted in only one series of tests being performed. Since the OCR results were so definitive from the interface testing, and previous research by Schmertmann (1991) has shown an increase in friction angle due to aging, yet very short-term, the internal residual shear test series was performed using an OCR of one. The aging effects on the internal soil friction angle were similar to the interface friction angle increase up to 10 days, yet the increase was much less than the interface friction angle increases thereafter, as seen in Figure 48.
Figure 48: Friction Angle Increase Over Time with Varying OCR and Internal Soil Friction Angle

The calculated friction angle within the soil was interestingly close at the 60-day time consolidation as the plots of the interface friction angle in Figure 49. The cause for this could be that the lines connecting the points for the interface friction angle are not representative of the actual trend the friction angles follow between 30 days and 90 days of consolidating. There is potential for the slope of the trend to be larger between the 30 and 90 days with a more gradual slope at a later time than what is presented.
4.5.3 Conclusions

A testing method was developed to experimentally demonstrate and determine the increased friction angle that is generated under a constant stress over time between a clay and concrete, as well as within clay. The results from the testing program indicate that there is an increase in the residual shear strength between clay and concrete as time passes. The results also show a larger increase in the frictional behavior when the soil has been subjected to a larger stress history prior to shearing.

Similar results apply to the internal shear strength of soil, yet there appears to be a limit in time when the shear strength either becomes static or is slightly reduced. Due to the nature of handling undisturbed soil samples, this type of testing can be subjected to disturbances that cannot be visibly noticed, but can affect the results. Also, it would
seem appropriate to perform additional internal soil testing with increased OCR to view the trends generated and compare to the interface testing results. Unfortunately, with the limitations of the soil amount as well as time, there are very few data points to draw a definitive conclusion.

The results from the testing do lead to the following provisional conclusions:

1. The testing procedure developed to determine the increased friction angle at the interface between clay and concrete appears to provide an effective method in quantifying soil aging.

2. The results of the testing program show an increase in frictional behavior at the interface between clay and concrete over time.

3. The larger the pre-consolidation pressure applied to a soil, the larger the increase of friction angle between clay and concrete, as the soil ages.

4. There is tentative data from the testing program that suggests the critical failure plane between a soil and concrete moves from the interface (sliding) to a location within the soil at some distance from the interface (deformation), sometime between 5 and 14 days after the pore pressure has dissipated.
CHAPTER 5

PILE CAPACITY PREDICTION METHOD BY COMBINING EXPERIMENTAL PROCESS WITH PREDICTION MODEL

The ability to successfully predict the frictional resistance a pile will obtain some time after installation can allow for much more accurate and cost-effective designs of deep foundation systems. The equation developed in Chapter 3 of this dissertation is designed to be used as a prediction model to determine the time-dependent residual friction angle, yet the equation requires two variables, namely $a_0$ and $a_l$, which had to be back-calculated. Chapter 4 provides a laboratory method to determine the amount of residual interface friction angle increase during aging which can be utilized to determine the variables developed in the model. Once the material parameters $a_0$ and $a_l$ are obtained from laboratory experimentation, this deterministic model can be employed to predict pile setup to be implemented in pile foundation design. A method is presented combining the research described in the previous two chapters to utilize the information gathered from the laboratory testing along with the developed prediction model to determine the time-dependent pile capacity.
5.1 Combining the Laboratory Aging Process with the Prediction Model

The results acquired from the laboratory testing can now be introduced into the model to aid in the further development of the method by which pile setup due to aging can be predicted prior to construction. As shown in Chapter 4, the time-dependent interface testing results between clay and concrete show an increased trend in the residual friction angle depending on the stress history of the soil. Similar to the implementation of the field testing data for the time-dependent friction angle, the results from the laboratory testing can be easily integrated into the prediction model previously developed as Equation 13.

The data from each sample of the laboratory friction testing from Table 10 is placed into Equation 13 and then grouped by OCR value, since only one soil type was evaluated in the lab. The $a_I$ value should be the same for all equations because it is specific to the OCR and the same soil type is used for all samples. So, the set of equations is combined to solve for the $a_I$ and $a_o$ values. Then the $a_I$ value found is used in each grouping of equations separated by OCR to determine if the resultant $a_o$ value is similar to the combined value. If the $a_o$ values are consistent between each of the set of equations, then they can be considered material factors, since the material is the same and the only difference being the stress history. This would agree well with the previously stated conclusions that the initial residual friction angle increases with a larger OCR for the same soil.

Once again, a basic iterative solver was implemented using the least squares method by minimizing the residuals (Equation 17) to determine the $a_o$ and $a_I$ variables for of the entire set of equations. The same iterative analysis was used to also determine...
$a_o$ and $a_l$ values by grouping the equations by OCR, to compare the overall values with the individual values. The values determined were very similar as shown in Table 11.

Table 11: Deterministic Results of Equation 13 Variables $a_o$ and $a_l$

<table>
<thead>
<tr>
<th>OCR</th>
<th>$a_o$</th>
<th>$a_l$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.05</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>6.39</td>
<td>0.135</td>
</tr>
<tr>
<td>6</td>
<td>7.04</td>
<td>0.135</td>
</tr>
<tr>
<td>Combined</td>
<td>6.82</td>
<td>0.135</td>
</tr>
</tbody>
</table>

The $a_l$ value for the OCR of one is not significant due to the nature of the exponent. Comparing the calculated friction angles based on the variables listed in Table 11 to the measured values from the laboratory data reveals a favorable comparison, as seen in Figure 50.

![Figure 50: Comparison of Predicted Residual Friction Angle using $a_o$ and $a_l$ Calculated Individually for Each OCR Group](image-url)
The values of the $a_0$ and $a_1$ variables between the different OCR groupings are very similar, suggesting that these values are specific to the soil. Unfortunately, there is only one type of soil tested to provide this conclusion, but the preliminary discovery is encouraging. Based on this discovery, it can be assumed that these values are soil characteristic coefficients and can quite possibly be correlated to one or more soil property, such as Plasticity Index, clay fraction, clay mineral, etc. More research should be performed to generate a more conclusive correlation, but the framework is developed.

Combining the test data from each OCR allows for the determination of one set of $a_0$ and $a_1$ variables which generates one prediction equation for this specific soil type, thus simplifying the design procedure. Equation 21 would be the friction angle prediction equation for the soil type tested with the newly developed experimental procedure,

$$
\phi'(t) = 6.82 \ OCR^{0.135} \log\left(\frac{t}{t_0}\right) + \phi'_{R_0}.
$$

(20)

Equation 21 was used to solve for the predicted friction angle values at the same time intervals as the laboratory testing. A comparison of the predicted values versus the measured values is provided in Figure 51.
The $R^2$ values from the trend-line generated from Figures 50 and 51 show a relatively close correlation between the predicted values using back-calculated coefficients and the measured values. The correlations between the individual coefficients and the combined coefficients are very similar, which is very positive since it is ideal to have one prediction equation for each soil type encountered. To summarize these findings, the $a_0$ value is a multiplicative term associated with the time variation and the $a_1$ value is the exponent that adjusts the intensity of OCR for calculating long-term friction angle. It can be concluded that these values appear to be soil specific and can be considered soil parameters, where $a_0$ might be called the time factor and $a_1$ the consolidation factor.
5.2 Predicting Pile Capacity using the Friction Angle Prediction Model

The main objective of the research is to develop a methodology to implement more accurate long-term frictional behavior based on soil properties into the design for driven piles. Using the information obtained through the laboratory testing program developed herein and the resulting enhanced prediction equation for the time-dependent friction angle, Equation 20, also developed within this research, a preliminary method for pile design can be presented.

5.2.1 Proposed Pile Capacity Calculation Procedure Utilizing Setup

The following steps provide a method for pile capacity determination based on experimental soil aging test data. The calculated time-dependent pile capacity can then be implemented into the commonly utilized $\beta$-method to design the pile dimensions.

1. During preliminary site investigation, obtain undisturbed soil samples at standard depth intervals and when soil strata changes.

2. Specimens obtained from clay samples should be tested in the laboratory with standard property tests, as well as consolidation and OCR determination.

3. Specimens obtained from the clay samples are tested according to the procedure presented in Chapter 4. Long-term testing can be performed up to 30 days for faster results.

4. The results from the long-term friction angle testing are then used to determine the soil coefficients in Equation 13.

5. Determine the frictional resistance for the pile segment according to Equation 11 with the time-dependent friction angle calculated from the enhanced
Equation 13 at some time after the dissipation of pore water pressure. Possibly recommend a time 50 to 100 days after the dissipation of pore water pressure.

6. Implement some safety factor and then calculate the dimensions of the pile using the pile capacity equation with the sum of all segments of soil the pile will encounter. End bearing should also be added if applicable.

5.2.2 Pile Capacity Determination based on Proposed Calculation Procedure

An example of a pile design using these steps is now shown using the experimental data presented in Chapters 4 and 5. The pile capacity process will have the following assumptions:

- The material within a proposed segment is homogeneous and 10 meters thick
- The proposed segment begins 5 meters below the surface and the water table is at the surface
- The proposed segment has an average determined OCR of 6
- Any excess pore pressure has been dissipated and pore pressure is hydrostatic
- Friction angle of the clay at the completion of the pore pressure dissipation is equal to the initial residual friction angle from laboratory tests
- Any pile setup obtained during the dissipation of pore water pressure is included in the initial equation
- The common material for the region is a square concrete pile, so laboratory tests are performed with concrete

The laboratory testing previously performed according to Chapter 4 is used to develop Equation 20 for the material tested. The vertical stress and pore pressure is
calculated based on the assumptions and are equal to 209 kPa and 98 kPa, respectively. Now utilizing Equation 20 and Equation 11, the time-dependent friction angles and frictional resistance values are calculated and presented in Table 12.

**Table 12: Time-Dependent Increased Frictional Resistance Using Soil Aging**

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Friction Angle (degrees)</th>
<th>Frictional Resistance (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27.9</td>
<td>1324</td>
</tr>
<tr>
<td>14</td>
<td>37.9</td>
<td>1832</td>
</tr>
<tr>
<td>100</td>
<td>45.3</td>
<td>2121</td>
</tr>
<tr>
<td>365</td>
<td>50.2</td>
<td>2237</td>
</tr>
</tbody>
</table>

Based on the proposed method developed in this research, the frictional resistance of a pile after a year installed is more than double from the initial resistance taken at day one. Even more significant, the 14-day frictional side shear value shows an increase of nearly 38%. Even if designers were to neglect the increase of side shear beyond the 14-day prediction to remain conservative, the design would still provide a significant benefit to the reduction of the pile size and then overall cost to a project. However, it should be noted that these values are site specific and derived based on many assumptions.

Fourteen days is typically the required amount of time that indicator piles are load tested to verify the design on the specific project in the state of Louisiana. Utilizing this proposed procedure for design and then subsequent implementation can easily be verified without any additional steps in the construction process. Utilizing the proposed method of setup with aging test data within the pile design can be completed prior to construction with potentially smaller piles that can be installed in less time.
5.2.3 Comparison of the Proposed Method to Existing Setup Prediction Methods

Implementing one of the previously mentioned empirical methods, such as the Skov and Denver method, requires previous knowledge of the soil interaction with piles to provide the setup factor, $A$, or, for a more accurate setup prediction, preconstruction pile installation and testing to produce the setup factor. The latter requires a significant amount of time and cost, because the installed pile requires testing and then a potential design change based on the results of the test data. While the former can be accurate, research presented herein has shown inconsistencies with this method.

A comparison with the proposed method will be made to the Skov and Denver method using a setup factor calculated from the results of the laboratory testing with the assumption that the frictional resistance values were gathered by indicator pile load testing. The previously mentioned assumptions will be used. Based on the 1-day and the 14-day results, a setup factor $A$ can be calculated using Equation 1,

$$\frac{1832}{1324} = A \log_{10} \frac{14}{1} + 1.$$

The calculated setup factor, $A$, equals 0.335. The setup factors presented in Table 1 show Skov and Denver as well as Bullock suggesting a value of 0.2 to be used for clay. The proposed method is compared to Skov and Denver prediction method using both setup factors. The side shear comparison will be for a segment 10 meters thick starting at five meters below the surface with homogeneous clay identical to the soil used for the laboratory testing presented in Chapter 4. The pile is concrete and has a perimeter of 1.83 meters. Figure 52 provides the results from the calculations from the two methods one year after installation.
Figure 52: Segment Side Resistance Comparisons between the Aging Process Proposed and the Skov and Denver Method

The Skov and Denver method using the calculated setup factor of 0.335 shows slightly higher results than the research method proposed. This indicates the design would be quite similar, yet the method proposed in this report for setup due to aging would provide a significant cost savings than the Skov and Denver method that requires indicator pile testing prior to construction. The same empirical method using the published setup factor of 0.2 shows a much more conservative design, potentially requiring larger piles with further embedment depth. The trend of both Skov and Denver prediction equations is continuously increasing, while the slope of the prediction method developed in this report is reducing as time passes. The significance of the deterministic method presented herein is that it incorporates the pile setup mechanism of soil aging
within the prediction model, unlike those methods that are completely empirical, such as the Skov and Denver method.

5.3 Summary and Conclusions

The experimental process presented in Chapter 4 appears to provide evidence that the properties of the soil can be linked to the amount of frictional resistance increase when in contact with concrete at a constant normal stress over time. With the data provided in the laboratory procedure, an enhanced prediction equation based on the model presented in Chapter 3 can be developed for a specific soil type. The model can then be used to predict the amount of side shear resistance due to aging effects a pile will be subjected to over time after the dissipation of pore water pressure.

The method was presented to determine the increased side shear with time using the experimental results obtained during the laboratory research portion. The method presented results similar to an idealized empirical setup method. Additional large-scale pile testing should be performed to correlate with the method proposed for further verification. However, based on the results presented from the example herein, this proposed method of determining the side shear resistance for pile design with contribution from aging effects appears to be a feasible method when clay soils are encountered. Implementing this design can potentially lead to significant cost savings on projects requiring pile foundation systems.
CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The following conclusions are derived from the results of the presented research:

1. A mechanistically-determined prediction model for pile setup in clay due to aging is established which relates specifically to the remolded friction angle increase with time after the dissipation of excess pore water pressure induced during installation and the OCR of the soil.

2. The model requires the determination of two coefficients $a_0$ and $a_1$, the time factor and the consolidation factor, respectively, which appear to be directly related to the properties of the soil. These two factors can be measured by performing simple and regular laboratory experiments.

3. Testing data from full-scale instrumented piles published in previous research was used to compare the predicted values of capacity obtained from the model with the measured capacity and proved to be quite accurate producing linear regression equation with an $R^2$ value of 0.922.

4. The predicted capacity using the new model was much more accurate than the prediction obtained from the conventional empirical method of setup prediction.

5. A testing method was developed to experimentally simulate the increased friction angle that is generated under a constant stress over time between a clay and
concrete using conventional direct shear apparatus. A detailed analysis to find the
the two coefficients of $a_0$ and $a_1$ was formulated.

6. The results from the testing program of a group of samples of one type of clay
from south Louisiana indicate that there is an increase in the residual shear
strength between clay and concrete as time passes. The results also show a larger
increase in the frictional behavior when the soil has been subjected to a larger
stress history prior to shearing.

7. The testing procedure developed to determine the increased friction angle at the
interface between clay and concrete appears to provide an effective method in
quantifying soil aging.

8. Combining the prediction model as well as the laboratory procedure developed, a
method to predict the frictional resistance of a pile due to aging prior to
installation of on-site indicator piles was presented.

9. The long-term setup prediction method presented results similar to an idealized
empirical setup method. Based on the results presented from the example herein,
this proposed method of determining the side shear resistance for pile design with
contribution from aging effects appears to be a feasible method when clay soils
are encountered. Implementing this design can potentially lead to significant cost
savings on projects requiring pile foundation systems.

10. The conventional estimation for the lateral earth pressure coefficient, $K$, used to
determine the horizontal earth pressure in over-consolidated soil

$$ K = (1 - \sin \phi'_c) \sqrt{OCR} $$
was shown to produce inaccurate estimates as the friction angle increases. The results produced using the following equation proved more accurate using this specific data,

\[ K = (1 - \sin \phi'_K)O\!C\!R^{\sin \phi'_K}. \]

6.2 Recommendations

The conclusions presented in this report leads to additional routes of research that should be followed to further enhance the methods previously described.

1. Ideally, the most effective work that can be performed is to install fully-instrumented piles into various soil types exhibiting setup behavior. These piles should be load tested at time intervals after installation to at least a year. Also, laboratory testing of the same soil at various depths of the pile should be performed to complement the methods proposed herein.

2. Additional experimental testing with differing soil properties, such as PI, Clay Fraction, Clay mineral should be performed to potentially correlate to increased frictional behavior with time.

3. Additional long-term experimental testing to include the full range of internal shear testing with differing OCR values with differing soil types to enhance any correlations presented.

4. Study the effects of the pore pressure dissipation and the relationship with the coefficient of consolidation or the hydraulic conductivity. Determine the amount of setup that occurs and the time at which aging is the sole contributor.
5. Based on the results from the other recommendations listed, develop complete method to determine an efficient, effective prediction of pile setup using common subsurface sampling techniques and standard lab testing devices.

6. A study into the micro- or nano- particle rearrangement within the clay fabric during the aging process could lead to a better understanding or a correlation between the shape of clay mineral and the increase in shear behavior.
APPENDIX A

SUBSURFACE EXPLORATORY BORING LOG FOR EXPERIMENTAL TEST SOIL
<table>
<thead>
<tr>
<th>SOIL TYPE AND COLOR</th>
<th>MODEL NUMBER</th>
<th>LENGTH</th>
<th>MATERIAL</th>
<th>SPT NQR</th>
<th>DIA.</th>
<th>HORIZONTAL</th>
<th>VERTICAL</th>
<th>SAMPLE LOCATION</th>
<th>DEPTH</th>
<th>RATED ENERGY</th>
<th>POINT LOAD</th>
<th>ELEVATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gr SI</td>
<td>420</td>
<td>19</td>
<td>1</td>
<td>13.35</td>
<td>8.9</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Lean CI</td>
<td>185</td>
<td>46</td>
<td>23</td>
<td>3.01</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>10.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Sasa CI</td>
<td>188</td>
<td>20</td>
<td>7</td>
<td>4.05</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>10.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Lean CI</td>
<td>107</td>
<td>37</td>
<td>13</td>
<td>2.05</td>
<td>6.1</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>11.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Sasa</td>
<td>107</td>
<td>71</td>
<td>N P</td>
<td>1.05</td>
<td>7.1</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>11.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Lean CI</td>
<td>144</td>
<td>59</td>
<td>19</td>
<td>2.01</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>11.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Sasa</td>
<td>113</td>
<td>40</td>
<td>N P</td>
<td>1.60</td>
<td>7.1</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>11.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Sasa w/Sa</td>
<td>N P</td>
<td>N P</td>
<td>N P</td>
<td>N P</td>
<td>50</td>
<td>61.0</td>
<td>50</td>
<td></td>
<td>11.0</td>
<td></td>
<td></td>
<td>50.0</td>
</tr>
<tr>
<td>Gr Sasa w/Sa</td>
<td>N P</td>
<td>N P</td>
<td>N P</td>
<td>N P</td>
<td>50</td>
<td>61.0</td>
<td>50</td>
<td></td>
<td>11.0</td>
<td></td>
<td></td>
<td>50.0</td>
</tr>
<tr>
<td>Gr Sasa w/Sa</td>
<td>N P</td>
<td>N P</td>
<td>N P</td>
<td>N P</td>
<td>50</td>
<td>61.0</td>
<td>50</td>
<td></td>
<td>11.0</td>
<td></td>
<td></td>
<td>50.0</td>
</tr>
<tr>
<td>Gr PadCI</td>
<td>111</td>
<td>42</td>
<td>35</td>
<td>1.44</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>03</td>
<td>47</td>
<td>140</td>
<td>1.44</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>112</td>
<td>50</td>
<td>71</td>
<td>5.12</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>106</td>
<td>93</td>
<td>91</td>
<td>1.00</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>110</td>
<td>42</td>
<td>31</td>
<td>3.64</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>97</td>
<td>70</td>
<td>56</td>
<td>3.15</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>107</td>
<td>82</td>
<td>55</td>
<td>10.07</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>109</td>
<td>86</td>
<td>55</td>
<td>5.69</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Lean CI w/Sa</td>
<td>103</td>
<td>57</td>
<td>24</td>
<td>2.69</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Lean CI</td>
<td>123</td>
<td>41</td>
<td>21</td>
<td>16.15</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Eclastic Cl</td>
<td>111</td>
<td>42</td>
<td>23</td>
<td>5.77</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Sasa</td>
<td>N P</td>
<td>12</td>
<td>45</td>
<td>10.45</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gr Sasa</td>
<td>N P</td>
<td>12</td>
<td>45</td>
<td>10.45</td>
<td>10.65</td>
<td>M S</td>
<td>M S</td>
<td></td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DATE TAKEN:** 11/3/2009

**TYPE OF METER:**

**LOCATION:** P.J. Oil East River

**DATE TAKEN:** 11/3/2009

**SITE NO:** 004545001441
APPENDIX B

EXPERIMENTAL TEST REPORTS
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Project</th>
<th>Lab Ref</th>
<th>Job</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Research</td>
<td></td>
<td>C18</td>
<td>SAI B</td>
</tr>
</tbody>
</table>

**Test Details**

- **Standard**: ASTM D3000-03 / AASHTO T296-12
- **Sample Type**: Thin walled push in sample
- **Lab. Temperature**: 70-90°F
- **Sample Description**: Residual shear
- **Variations from procedure**: None

**Specimen Details**

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>A</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000inh</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4250 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td></td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>117.43 lb/ft³</td>
<td>Degree of Saturation 50.29%</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>86.28 lb/ft³</td>
<td>Initial Voids Ratio 118.73%</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>127.35 lb/ft³</td>
<td>Final Water Content 45.34%</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>87.02 lb/ft³</td>
<td>Dry Mass 0.1025 lb</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td></td>
</tr>
<tr>
<td>Comments</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

**Deformation vs Square Root Time**

![Deformation vs Square Root Time Graph](image-url)
Shear Stress vs Displacement

Change in Specimen Thickness vs Displacement

Rate of Horizontal Displacement

Stage 1: 0.020000 in/min
Stage 2: 0.020000 in/min
Stage 3: 0.020000 in/min
Stage 4: 0.020000 in/min
Stage 5: 0.020000 in/min
# Shear Strength by Direct Shear

(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td>Job C18</td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Simple SA1 B</td>
</tr>
</tbody>
</table>

## Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.44 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>4.72 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0086 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>3.15 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0129 in</td>
</tr>
</tbody>
</table>

Tested By

and Date

Checked By

and Date

Approved By

and Date
Shear Strength by Direct Shear
(Small Shear Box)

Client: Research
Project: Borehole
Lab Ref: Job C18
Sample: SA1 B

Test Details
- Standard: ASTM D3080 03 / AASHTO T236-92
- Sample Type: Thin walled push in sample
- Lab Temperature: 70.0 deg F
- Sample Description: 1 day shear
- Variations from procedure: None

Specimen Details
- Specimen Reference: B
- Description:
  - Depth within Sample: 0.0000m
  - Initial Height: 0.4250 in
  - Structure Preparation:
    - Initial Wet Unit Weight: 117.43 lb/ft³
    - Initial Dry Unit Weight: 86.28 lb/ft³
  - Initial Water Content:
  - Degree of Saturation: 50.29 %
  - Initial Voids Ratio: 118.737
  - Final Wet Unit Weight: 126.27 lb/ft³
  - Final Dry Unit Weight: 86.28 lb/ft³
  - Final Water Content: 46.34 %
  - Dry Mass: 0.1025 lb
  - Tested Dry or Submerged: Submerged
  - Comments: Calculated from initial and dry unit weights of whole specimen

Deformation vs Square Root Time

<table>
<thead>
<tr>
<th>Time Square Root Mins</th>
<th>Deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.00002</td>
</tr>
<tr>
<td>10</td>
<td>0.00004</td>
</tr>
<tr>
<td>15</td>
<td>0.00006</td>
</tr>
<tr>
<td>20</td>
<td>0.00008</td>
</tr>
<tr>
<td>25</td>
<td>0.00010</td>
</tr>
<tr>
<td>30</td>
<td>0.00012</td>
</tr>
<tr>
<td>35</td>
<td>0.00014</td>
</tr>
<tr>
<td>40</td>
<td>0.00016</td>
</tr>
</tbody>
</table>
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Borehole</td>
<td>Job Sample</td>
<td>C18 SA1 B</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
# Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA1B</td>
</tr>
</tbody>
</table>

## Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D2080 03 &amp; AASHTO T296 92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled puck in sample</td>
</tr>
<tr>
<td>Particle Specific</td>
<td>165 ±3</td>
</tr>
<tr>
<td>Gravity</td>
<td>Single or Multi Stage</td>
</tr>
<tr>
<td>Location</td>
<td>Multi Stage Stages</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>7 day consol than 7 day shear</td>
</tr>
<tr>
<td>Variations from</td>
<td>None</td>
</tr>
<tr>
<td>procedure</td>
<td></td>
</tr>
</tbody>
</table>

## Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>C</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4250 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td></td>
<td>Initial Water Content</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>117.43 lbf ft</td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>86.28 lbf/ft³</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>125.66 lbf/ft³</td>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>85.87 lbf/ft³</td>
<td>Dry Mass</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td></td>
</tr>
<tr>
<td>Comments</td>
<td>None</td>
<td>Calculated from initial and dry weights to whole specimen</td>
</tr>
</tbody>
</table>

## Deformation vs Square Root Time

![Deformation vs Square Root Time Graph]

- **Time Square Root Mins**: 0, 2, 4, 6, 8, 10, 12, 14
- **Deformation**: 0.0003, 0.00018, 0.00043, 0.0004, 0.00053

---

Page 1 of 3
(Small Shear Box)
Shear Strength by Direct Shear
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA1B</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
### Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA1 B</td>
</tr>
</tbody>
</table>

#### Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080 03 / AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>7 day shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

#### Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>Depth within Sample</td>
<td>3 000 ft</td>
</tr>
<tr>
<td>Initial Weight</td>
<td>3.4250 in</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td></td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>117.43 lb ft</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>56.28 lb ft</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>126.21 lb ft</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>56.25 lb ft</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
</tr>
</tbody>
</table>

| Initial Water Content | 36.1% (trimmings 32.6%) |
| Degree of Saturation | 50.29% |
| Initial Voids Ratio | 118.737 |
| Final Water Content | 46.34% |
| Dry Mass | 0.1025 lb |

#### Deformation vs Square Root Time

```
0 10 20 30 40 50 60 70
0.0001 0.0002 0.0003 0.0004 0.0005 0.0006 0.0007 0.0008 0.0009 0.001
```

* Calculated from initial and dry weights of whole specimen
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA1 B</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

| Client | Research
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td>C18</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D4354-93 : AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg.F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>Initial moisture then residual then 14 day shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Reference</td>
</tr>
<tr>
<td>Depth within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
</tr>
<tr>
<td>Structure / Preparation</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
</tr>
<tr>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
</tr>
<tr>
<td>Dry Mass</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

Time Square Root Mins

Deformation in
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client Project</th>
<th>Research</th>
<th>Lab Ref Job</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole</td>
<td></td>
<td>C18</td>
<td>SA1 C</td>
</tr>
</tbody>
</table>

### Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>634 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>310 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0363 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>2.58 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0009 in</td>
</tr>
</tbody>
</table>

Tested By

Checked By

Approved By

and Date

and Date

and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA1C</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080-03 AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>14 day shear specimen B for SA1C</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

Particle Specific Gravity | 165.43

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
</tr>
<tr>
<td>Orientation within Sample</td>
<td></td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4070 in</td>
</tr>
<tr>
<td>Area</td>
<td>4.83174 in²</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td></td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>111.20 lb/ft³</td>
</tr>
<tr>
<td>Degree of Saturation</td>
<td>61.66 %</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>72.96 lb/ft³</td>
</tr>
<tr>
<td>Initial Voids Ratio</td>
<td>140.605</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>111.87 lb/ft³</td>
</tr>
<tr>
<td>Final Water Content</td>
<td>52.41 %</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>73.40 lb/ft³</td>
</tr>
<tr>
<td>Dry Mass</td>
<td>0.0830 lb</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
</tr>
<tr>
<td>Comments</td>
<td></td>
</tr>
</tbody>
</table>

*Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

![Graph showing deformation vs square root time]
Shear Strength by Direct Shear  
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA1 C</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement: 0.001206 mm/min
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>5A1C</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>Particle Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM D3080-03 / AASHTO T236-92</td>
<td>165 43</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin walled push in sample</td>
<td>Multi Stage</td>
</tr>
<tr>
<td></td>
<td>5 Stages</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lab. Temperature</th>
<th>Sample Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>70.0 deg F</td>
<td>30 day shear Residual shear</td>
</tr>
</tbody>
</table>

Variations from procedure

None

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth within Sample</th>
<th>Orientation within Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 0000in</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Height</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 4050 in</td>
<td>4.90900 in²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Structure Preparation</th>
<th>Initial Water Content*</th>
</tr>
</thead>
<tbody>
<tr>
<td>103 03 lbf/ft³</td>
<td>61.2% (trimmings 48.4%)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Wet Unit Weight</th>
<th>Degree of Saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td>63 91 lbf/ft³</td>
<td>65.99%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Final Wet Unit Weight</th>
<th>Final Water Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>105 97 lbf/ft³</td>
<td>0.0735 lb</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Final Dry Unit Weight</th>
<th>Dry Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>63 84 lbf/ft³</td>
<td>61.2%</td>
</tr>
</tbody>
</table>

Tested Dry or Submerged

Submerged

Comments

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

Deformation ln

Time Square Root Mins

0.0014

0.0016

0.0018

0.002

0.0022

0.0024

0.0026

0.0028

0.003

0.0032
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA1D</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement

Stage 1: 0.00000 in/min  Stage 2: 0.00000 in/min  Stage 3: 0.00000 in/min  Stage 4: 0.00000 in/min  Stage 5: 0.00000 in/min
Shear Strength by Direct, Shear  
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA1 D</td>
</tr>
</tbody>
</table>

### Conditions at Failure

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>5.34 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>3.95 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0063 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>2.84 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0027 in</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SAI D</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>Particle Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM D3080-03 • AASHTO T236-92</td>
<td>165 43</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Lab. Temperature</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin walled push in sample</td>
<td>70 0 deg F</td>
<td>Single Stage</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>Variations from procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 day shear</td>
<td>None</td>
</tr>
</tbody>
</table>

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth within Sample</th>
<th>Orientation within Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 0000m</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Height</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 4050 in</td>
<td>4 83174 in²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Structure / Preparation</th>
<th>Initial Water Content*</th>
</tr>
</thead>
<tbody>
<tr>
<td>104 68 lbf ft³</td>
<td>61 2 % (trimmings 48 4 %)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Wet Unit Weight</th>
<th>Degree of Saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td>64 93 lbf ft³</td>
<td>64 06 %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Final Wet Unit Weight</th>
<th>Initial Voids Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>107 97 lbf ft³</td>
<td>158 122</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Final Dry Unit Weight</th>
<th>Dry Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>65 05 lbf ft³</td>
<td>0 0735 lb</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested Dry or Submerged</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Submerged</td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Borehole</td>
<td>Job</td>
</tr>
<tr>
<td>C18</td>
<td>SA1D</td>
<td></td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement | Stage 1 0.0102/60in/min

H E H, et al
# Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job C18</td>
<td>Sample SA1D</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By and Date
Checked By and Date
Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA4 OCR</td>
</tr>
</tbody>
</table>

**Test Details**

<table>
<thead>
<tr>
<th>Standard</th>
<th>Particle Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM D2808-03 ' AASHTO T236-92</td>
<td>165.43</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Lab. Temperature</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin walled push in sample</td>
<td>70.0 deg F</td>
<td>Multi Stage 4 Stages</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>Variations from procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual prior to 90 day</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**Specimen Details**

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Orientation within Sample</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth within Sample</th>
<th>0.0000 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Height</td>
<td>0.4250 ft</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Structure Preparation</th>
<th>Initial Water Content*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>4.8317 ft²</td>
</tr>
<tr>
<td>Degree of Saturation</td>
<td>51.63%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Wet Unit Weight</th>
<th>Initial Voids Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>104.17 lb/ft³</td>
<td>(trimmings 41.2%)</td>
</tr>
<tr>
<td>74.92 lb/ft³</td>
<td>136.899</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Final Water Content</th>
<th>Dry Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>51.12%</td>
<td>0.0890 lb</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested Dry or Submerged</th>
</tr>
</thead>
<tbody>
<tr>
<td>Submerged</td>
</tr>
</tbody>
</table>

*Calculated from initial and dry weights of whole specimen.

**Deformation vs Square Root Time**

![Graph](image)
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA4 OCR</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By
and Date

Checked By
and Date

Approved By
and Date
## Shear Strength by Direct Shear
*(Small Shear Box)*

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
<th>Job</th>
<th>C18</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample</td>
<td></td>
<td>SA4 OCR</td>
</tr>
</tbody>
</table>

### Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080-03 / AASHTO T238-92</th>
<th>Particle Specific Gravity</th>
<th>165.43</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
<td>Single or Multi Stage</td>
<td>Single Stage</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg F</td>
<td>Location</td>
<td></td>
</tr>
<tr>
<td>Sample Description</td>
<td>90 day shear Specimen Number B</td>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

### Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>B</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4250 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>Initial Water Content*</td>
<td>44.4% (trimmings: 41.2%)</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>108.17 lb/ft(^3)</td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>74.92 lb/ft(^3)</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>113.29 lb/ft(^3)</td>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>74.96 lb/ft(^3)</td>
<td>Dry Mass</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td>Comments</td>
</tr>
</tbody>
</table>

*Calculated from initial and dry weights of whole specimen

### Deformation vs Square Root Time

![Deformation vs Square Root Time Graph]

- **Time Square Root Mins**
  - 0 2 4 6 8 10 12 14 16 18 20
- **Deformation in**
  - 0.0006 0.0007 0.0008 0.0009 0.001 0.0011 0.0012 0.0013 0.0014 0.0015 0.0016

* Image data for the graph is not provided in the text.*
Shear Strength by Direct Shear
(Small Shear Box)

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement
Stage 1: 0.400 In/min
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA4 OCR</td>
</tr>
</tbody>
</table>

### Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.44 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>5.22 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.00/9 th</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>0.00 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0021 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested By</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>and Date</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Checked By</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>and Date</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Approved By</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>and Date</td>
<td></td>
</tr>
</tbody>
</table>
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td>Job C18</td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample SA2 B</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
</tr>
<tr>
<td>Sample Type</td>
</tr>
<tr>
<td>Particle Specific Gravity</td>
</tr>
<tr>
<td>Lab. Temperature</td>
</tr>
<tr>
<td>Sample Description</td>
</tr>
<tr>
<td>Variations from procedure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Reference</td>
</tr>
<tr>
<td>Depth within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
</tr>
<tr>
<td>Structure Preparation</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
</tr>
<tr>
<td>Comments</td>
</tr>
</tbody>
</table>

* Calculated from initial dry weight of whole specimen

Deformation vs Square Root Time

![Deformation vs Square Root Time graph](image-url)
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA2B</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement

St1 ε1 0.020000 in/min  St2 ε2 0.020000 in/min  St3 ε3 0.020000 in/min
St4 ε4 0.020000 in/min  St5 ε5 0.020000 in/min  St6 ε6 0.020000 in/min
# Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA2 B</td>
</tr>
</tbody>
</table>

## Conditions at Failure

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.44 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>6.69 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0222 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>3.15 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0337 in</td>
</tr>
</tbody>
</table>

| Tested By and Date         |             |
| Checkel By and Date        |             |
| Approved By and Date       |             |
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>Sample</td>
</tr>
<tr>
<td>Borehole</td>
<td>C18</td>
<td>SA2 B</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3380 03 / AASHTO T235 92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thir walled push in sample</td>
</tr>
<tr>
<td>Lab Temperature</td>
<td>70 °C deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>1 day shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
<tr>
<td>Particle Specific Gravity</td>
<td>155.43</td>
</tr>
<tr>
<td>Single Stage Location</td>
<td>Single Stage</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Reference</td>
</tr>
<tr>
<td>Depth within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
</tr>
<tr>
<td>Structure Preparation</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
</tr>
<tr>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Water Content</td>
</tr>
<tr>
<td>Dry Mass</td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA2 B</td>
</tr>
</tbody>
</table>

Shear Stress vs Displacement

Change in Specimen Thickness vs Displacement

Rate of Horizontal Displacement: 0.010280 mil/min
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SA2 B</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By:  

Checked By:  

Approved By:  

Date:  

Date:  

Date:  

Date:
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client Project</th>
<th>Research Job Ref</th>
<th>SA2B Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Test Details**

| Standard Sample Type | Particle Specific Gravity Single or Multi Stage Location Variations from procedure |
|----------------------|------------------------------------|-------------------------------------|-------------------------------------|
| ASTM D3080-03, AASHTO T236-92 Thin walled push in sample 165 43 Multi Stage 6 Stages | 165 43 Multi Stage 6 Stages | None |

**Lab. Temperature Sample Description**

<table>
<thead>
<tr>
<th>70 0 deg F</th>
<th>Residual prior to 7 day shear</th>
</tr>
</thead>
</table>

**Test Details**

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>Area</td>
</tr>
<tr>
<td>Structure / Preparation</td>
<td>Initial Water Content</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>Dry Mass</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
</tr>
<tr>
<td>Comments</td>
<td></td>
</tr>
</tbody>
</table>

*Calculated from initial and dry weights of whole mass

**Deformation vs Square Root Time**

Deformation in

<table>
<thead>
<tr>
<th>Time Square Root Mins</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
</tr>
<tr>
<td>0.0032</td>
</tr>
</tbody>
</table>

Deformation in
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA2 B</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement

Stage 1: 0.020000 in/min  Stage 2: 0.020000 in/min  Stage 3: 0.020000 in/min  Stage 4: 0.020000 in/min  Stage 5: 0.020000 in/min  Stage 6: 0.020000 in/min
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Lab Ref</td>
</tr>
<tr>
<td>Borehole</td>
<td>Job C18</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By and Date
Checked By and Date
Approved By and Date
# Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080 03 AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>7 day shear</td>
</tr>
<tr>
<td>Variations from Procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

## Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td></td>
<td>Initial Water Content'</td>
</tr>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4130 in</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td></td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>113.48 lb ft³</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>71.90 lb ft³</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>112.61 lb ft³</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>73.89 lb ft³</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
</tr>
</tbody>
</table>

### Comments

- Calculated from initial and dry weight of whole sample.

### Deformation vs Square Root Time

- **Deformation in:**
  - 0.011
  - 0.012
  - 0.013
  - 0.014
  - 0.015
  - 0.016
  - 0.017
  - 0.018
  - 0.019
  - 0.02

- **Time Square Root Mins:**
  - 0
  - 5
  - 10
  - 15
  - 20
  - 25
  - 30
  - 35
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>S42 B</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement:
- Sample 1: 0.01005 in/min
- Sample 2: 0.02000 in/min
- Sample 3: 0.03000 in/min
- Sample 4: 0.02000 in/min
- Sample 5: 0.03000 in/min
- Sample 6: 0.01000 in/min
# Shear Strength by Direct Shear
## (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA2 R</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>644 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>2.83 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.014 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>1.84 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0021 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested By and Date</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Checked By and Date</td>
<td></td>
</tr>
<tr>
<td>Approved By and Date</td>
<td></td>
</tr>
</tbody>
</table>

159
Shear Strength by Direct Shear (Small Shear Box)

Client: Research  
Project: Ikirehofr  
Borehole: Sample SA2C

<table>
<thead>
<tr>
<th>Test Details</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
<td>ASTM D3080-03 / AASHTO T236-92</td>
<td>Particle Specific Gravity</td>
</tr>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
<td>Single or Multi Stage</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70 0 deg F</td>
<td>Location</td>
</tr>
<tr>
<td>Sample Description</td>
<td>Residual Slag for 14 day shear</td>
<td>Variations from procedure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Details</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Reference</td>
<td>A</td>
<td>Description</td>
</tr>
<tr>
<td>Depth within Sample</td>
<td>0.0000m</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4090 in</td>
<td>Area 4.83174 in²</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>Initial Water Content*</td>
<td>50.65% (trimmings 41.9%)</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>106 90 lbf/ft³</td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>72 60 lbf/ft³</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>113 53 lbf/ft³</td>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>74 49 lbf/ft³</td>
<td>Dry Mass</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td>Comments</td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

Time Square Root Mins

0 10 20 30 40 50 60

0.0089 0.0099 0.0109 0.0119 0.0129 0.0139 0.0149
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Project: Job C18
Borehole: Sample SA2C

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement:
Stage 1: 0.02000 in/min
Stage 2: 0.02000 in/min
Stage 3: 0.02000 in/min
Stage 4: 0.02000 in/min
Stage 5: 0.02000 in/min
Stage 6: 0.02000 in/min
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Project</th>
<th>Borehole</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
</table>

Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.44  psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>5.24  psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0074 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>2.88  psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0085 in</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
<th>Job</th>
<th>C18</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample</td>
<td></td>
<td>SA2 C</td>
</tr>
</tbody>
</table>

**Test Details**

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3380-03 / AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>14 day shear</td>
</tr>
<tr>
<td>Variations from</td>
<td>None</td>
</tr>
<tr>
<td>procedure</td>
<td></td>
</tr>
</tbody>
</table>

**Particle Specific Gravity**

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3380-03 / AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>14 day shear</td>
</tr>
<tr>
<td>Variations from</td>
<td>None</td>
</tr>
<tr>
<td>procedure</td>
<td></td>
</tr>
</tbody>
</table>

**Test Details**

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3380-03 / AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>14 day shear</td>
</tr>
<tr>
<td>Variations from</td>
<td>None</td>
</tr>
<tr>
<td>procedure</td>
<td></td>
</tr>
</tbody>
</table>

**Specimen Details**

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>B</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4090 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure / Preparation</td>
<td></td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>108.90 lbf/ft³</td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>72.60 lbf/ft³</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>112.27 lbf/ft³</td>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>73.68 lbf/ft³</td>
<td>Dry Mass</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td>Calculated from initial and dry weights of whole specimen</td>
</tr>
<tr>
<td>Comments</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Deformation vs Square Root Time**

<table>
<thead>
<tr>
<th>Time Square Root Min</th>
<th>Deformation in</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0037</td>
</tr>
<tr>
<td>0.5</td>
<td>0.0032</td>
</tr>
<tr>
<td>1</td>
<td>0.0047</td>
</tr>
<tr>
<td>1.5</td>
<td>0.0052</td>
</tr>
<tr>
<td>2</td>
<td>0.0057</td>
</tr>
<tr>
<td>2.5</td>
<td>0.0055</td>
</tr>
<tr>
<td>3</td>
<td>0.0053</td>
</tr>
</tbody>
</table>
(Small Shear Box)
Shear Strength by Direct Shear

<table>
<thead>
<tr>
<th>Sample</th>
<th>Lab Ref</th>
<th>Project</th>
<th>Research</th>
</tr>
</thead>
<tbody>
<tr>
<td>30/1</td>
<td>C14</td>
<td>200</td>
<td>2000</td>
</tr>
</tbody>
</table>

Karl of Horizontal Implementation

Change in Thickness

Relative Lateral Displacement %

Shear Stress vs Displacement
Shear Strength by Direct Shear  
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA2 C</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>Sample</td>
</tr>
<tr>
<td>Borehole</td>
<td>C18</td>
<td>SA2 D</td>
</tr>
</tbody>
</table>

**Test Details**

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3039-03 / AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lb. Temperature</td>
<td>70°F deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>30 day residual shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

**Specimen Details**

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth within Sample</th>
<th>Orientation within Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0000 in</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Height</th>
<th>Area</th>
<th>Initial Water Content*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4150 in</td>
<td></td>
<td>433174 in²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Structure Preparation</th>
<th>Initial Dry Unit Weight</th>
<th>Degree of Saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>66.45 lb/ft³</td>
<td>62.28 %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Wet Unit Weight</th>
<th>Initial Voids Ratio</th>
<th>Final Water Content</th>
<th>Dry Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>105.07 lb/ft³</td>
<td>154.524</td>
<td>62.75 %</td>
<td>0.3765 lb</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Final Wet Unit Weight</th>
<th>Final Dry Unit Weight</th>
<th>Tested Dry or Submerged</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>106.61 lb/ft³</td>
<td>65.51 lb/ft³</td>
<td>Submerged</td>
<td></td>
</tr>
</tbody>
</table>

*Calculated from initial and dry weights for wet specimen

**Deformation vs Square Root Time**

<table>
<thead>
<tr>
<th>Time Square Root Mins</th>
<th>Deformation in</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-0.0001</td>
</tr>
<tr>
<td>5</td>
<td>0.0005</td>
</tr>
<tr>
<td>10</td>
<td>0.001</td>
</tr>
<tr>
<td>15</td>
<td>0.0015</td>
</tr>
<tr>
<td>20</td>
<td>0.002</td>
</tr>
<tr>
<td>25</td>
<td>0.0025</td>
</tr>
<tr>
<td>30</td>
<td>0.003</td>
</tr>
<tr>
<td>35</td>
<td>0.0035</td>
</tr>
</tbody>
</table>
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA? D</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

![Graph showing shear stress vs displacement]

Change in Specimen Thickness Vs Displacement

![Graph showing change in specimen thickness vs displacement]

<table>
<thead>
<tr>
<th>Rate of Horizontal Displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1: 0.020000 mm/min</td>
</tr>
<tr>
<td>Stage 2: 0.020000 mm/min</td>
</tr>
<tr>
<td>Stage 3: 0.020000 mm/min</td>
</tr>
</tbody>
</table>

0 10 20 30 40 50 60 70 80 90 100

-0.0003 0.0008 0.00012 0.00018

0 0.0002 0.0003 0.0004 0.0005

0 10 20 30 40 50 60 70 80 90 100

0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0

0 10 20 30 40 50 60 70 80 90 100

0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0

0 10 20 30 40 50 60 70 80 90 100

0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0
## Shear Strength by Direct Shear
### (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borhole</td>
<td>Sample</td>
<td>SA2 D</td>
</tr>
</tbody>
</table>

### Conditions a: Failure

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.44 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>5.24 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0120 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>2.88 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>-0.0019 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested By and Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Checked By and Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Approved By and Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>
Shear Strength by Direct Shear  
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
<th>Job</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>C13</td>
<td>SA2D</td>
</tr>
</tbody>
</table>

**Test Details**

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080-03, AASHTO T20G-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>30 day shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

**Specimen Details**

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>B</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000in</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4120 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td></td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>105.07 lbf ft3</td>
<td>Degree of Saturation 62.28%</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>66.43 lbf ft3</td>
<td>Initial Voids Ratio 154.524</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>109.50 lbf ft3</td>
<td>Final Water Content 62.75%</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>67.29 lbf ft3</td>
<td>Dry Mass 0.0765 lb</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td></td>
</tr>
<tr>
<td>Comments</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Calculated from initial and dry weights of whole specimen

**Deformation vs Square Root Time**

![Deformation vs Square Root Time Graph]

**Deformation in**

<table>
<thead>
<tr>
<th>Time Square Root Mine</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
</tr>
<tr>
<td>0.0005</td>
</tr>
<tr>
<td>0.001</td>
</tr>
<tr>
<td>0.0015</td>
</tr>
<tr>
<td>0.002</td>
</tr>
<tr>
<td>0.0025</td>
</tr>
<tr>
<td>0.003</td>
</tr>
<tr>
<td>0.0035</td>
</tr>
<tr>
<td>0.004</td>
</tr>
<tr>
<td>0.0045</td>
</tr>
<tr>
<td>0.005</td>
</tr>
</tbody>
</table>
# Shear Strength by Direct Shear

(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Borehole</td>
<td>Job Sample</td>
<td>C18 SA2 D</td>
</tr>
</tbody>
</table>

## Conditions at Failure

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.12 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>4.93 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>3.0085 ft</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>3.00 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>3.6056 ft</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear  
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SAS OCR</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>Particle Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM D3080-03 / AASHTO T296-92</td>
<td>165.43</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin walled push in sample</td>
<td>Multi Stage : 4 Stages</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lab. Temperature</th>
<th>Sample Description</th>
<th>Variations from procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>70.0 deg.F</td>
<td>Residual prior to 90 day shear</td>
<td>None</td>
</tr>
</tbody>
</table>

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
<th>A</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>Orientation within Sample</td>
<td>0.0000m</td>
<td>4.83174 in²</td>
</tr>
<tr>
<td>Initial Height</td>
<td>Area</td>
<td>0.4100 in</td>
<td></td>
</tr>
<tr>
<td>Structure / Preparation</td>
<td>Initial Water Content*</td>
<td>114.31 lbf/ft³</td>
<td></td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>Degree of Saturation</td>
<td>77.66 lbf/ft³</td>
<td></td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>Initial Voids Ratio</td>
<td>116.91 lbf/ft³</td>
<td></td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>Final Water Content</td>
<td>77.65 lbf/ft³</td>
<td></td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>Dry Mass</td>
<td>0.0890 lb</td>
<td></td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Comments</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

![Graph showing deformation vs square root time](image)
Shear Strength by Direct Shear
(Small Shear Box)

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement
Stage 1: 0.020000 in/min
Stage 2: 0.020000 in/min
Stage 3: 0.020000 in/min
Stage 4: 0.020000 in/min
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
<td></td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA5 OCR</td>
<td></td>
</tr>
</tbody>
</table>

Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.4 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>4.98 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0446 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>0.19 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0013 in</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
</tr>
<tr>
<td>Sample Type</td>
</tr>
<tr>
<td>Lab Temperature</td>
</tr>
<tr>
<td>Sample Description</td>
</tr>
<tr>
<td>Variations from procedure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Reference</td>
</tr>
<tr>
<td>Depth within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
</tr>
<tr>
<td>Structure Preparation</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
</tr>
</tbody>
</table>

*Calculated from initial and dry weights of whole specimen

**Deformation vs Square Root Time**

![Graph showing deformation vs square root time]
Shear Strength by Direct Shear
(Small Shear Box)

Client

Project

Borehole

Research

Lab Ref

C18

Sample

SA5 OCR

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement | Stage 1 0.01026 in/min

Page 2 of 3
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA3 OCR</td>
</tr>
</tbody>
</table>

Conditions at Failure

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.44 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>6.51 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0032 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>0.00 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0103 in</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

Client: research
Project: Job: C18
Borehole: Sample: SA3 B

<table>
<thead>
<tr>
<th>Test Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard: ASTM D3080-93 / AASHTO T236-92</td>
</tr>
<tr>
<td>Sample Type: Thin walled push in sample</td>
</tr>
<tr>
<td>Particle Specific Gravity: 165.43</td>
</tr>
<tr>
<td>Lab. Temperature: 70.0 deg F</td>
</tr>
<tr>
<td>Sample Description: 1 Day Residual</td>
</tr>
<tr>
<td>Variations from procedure: None</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Reference: A</td>
</tr>
<tr>
<td>Description:</td>
</tr>
<tr>
<td>Depth within Sample: 9.0000 in</td>
</tr>
<tr>
<td>Orientation within Sample: A90900 in2</td>
</tr>
<tr>
<td>Initial Height: 0.4130 in</td>
</tr>
<tr>
<td>Area: 4.90900 in2</td>
</tr>
<tr>
<td>Initial Wet Unit Weight: 107.45 lb ft3</td>
</tr>
<tr>
<td>Initial Dry Unit Weight: 57.55 lb ft3</td>
</tr>
<tr>
<td>Final Wet Unit Weight: 79.10 lb ft3</td>
</tr>
<tr>
<td>Final Dry Unit Weight: 52.87 lb ft3</td>
</tr>
<tr>
<td>Tested Dry or Submerged: Submerged</td>
</tr>
<tr>
<td>Initial Water Content*: 88%</td>
</tr>
<tr>
<td>Degree of Saturation: 80.32%</td>
</tr>
<tr>
<td>Initial Voids Ratio: 178.514</td>
</tr>
<tr>
<td>Final Water Content: 49.63%</td>
</tr>
<tr>
<td>Dry Mass: 0.0675 lb</td>
</tr>
<tr>
<td>Initial Water Content: 88%</td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

Deformation in in

Time Square Root Mins

Page 1 of 3
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement

Stage 1: 0.020000 in/min
Stage 2: 0.020000 in/min
Stage 3: 0.020000 in/min
Stage 4: 0.020000 in/min
Stage 5: 0.020000 in/min
Stage 6: 0.020000 in/min
Stage 7: 0.020000 in/min
Stage 8: 0.020000 in/min
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA3 B</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested By and Date</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Checked By and Date</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Approved By and Date</th>
</tr>
</thead>
</table>
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample SA3 B</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080-93 / AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 dog F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>1 day shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td>Orientation within Sample, Area, Initial Water Content</td>
</tr>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4130 in</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>109.15 lbf/ft³</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>58.47 lbf/ft³</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>20.16 lbf/ft³</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>10.25 lbf/ft³</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
</tr>
<tr>
<td>Dry Mass</td>
<td>0.0675 lb</td>
</tr>
</tbody>
</table>

Deformation vs Square Root Time

Deformation in

<table>
<thead>
<tr>
<th>Time (Square Root Min)</th>
<th>0</th>
<th>0.0002</th>
<th>0.0022</th>
<th>0.0042</th>
<th>0.0062</th>
<th>0.0082</th>
<th>0.0102</th>
<th>0.0122</th>
</tr>
</thead>
</table>

- * Calculated from initial and dry weights of whole specimen.
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA3 B</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement: 0.01026056 mm/min
# Shear Strength by Direct Shear

*(Small Shear Box)*

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td>Job C18</td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample SA3 B</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By

and Date

Checked By

and Date

Approved By

and Date
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
<th>Lab Ref</th>
<th>Job</th>
<th>C18</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample</td>
<td>SA3 B</td>
<td></td>
</tr>
</tbody>
</table>

**Test Details**

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080 03, AASHTO T236-92</th>
<th>Particle Specific Gravity</th>
<th>165 43</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push-in sample</td>
<td>Single or Multi Stage Location</td>
<td>Multi Stage 5 Stages</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70 0 deg F</td>
<td>Sample Description</td>
<td>7 Day Fesidual</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Specimen Details**

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>C</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
<td>Orientation within Sample Area</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4130 in</td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td></td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>109 15 lb/ft³</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>58 47 lb/ft³</td>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>93 28 lb/ft³</td>
<td>Dry Mass</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>52 34 lb/ft³</td>
<td>Tested Dry or Submerged</td>
</tr>
<tr>
<td>Comments</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

**Deformation vs Square Root Time**

Deformation vs Square Root Time

<table>
<thead>
<tr>
<th>Time Square Root mins</th>
<th>0</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformation in in</td>
<td>0.0003</td>
<td>0.0012</td>
<td>0.0025</td>
<td>0.0038</td>
<td>0.0052</td>
<td>0.0068</td>
<td>0.0085</td>
<td>0.0103</td>
<td>0.0123</td>
</tr>
</tbody>
</table>

Page 1 of 3
Shear Stress vs Displacement

Change in Specimen Thickness vs Displacement

Rate of Horizontal Displacement

<table>
<thead>
<tr>
<th>Stage</th>
<th>Rate of Horizontal Displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1</td>
<td>0.020000 m/m/min</td>
</tr>
<tr>
<td>Stage 2</td>
<td>0.020000 m/m/min</td>
</tr>
<tr>
<td>Stage 3</td>
<td>0.020000 m/m/min</td>
</tr>
<tr>
<td>Stage 4</td>
<td>0.050000 m/m/min</td>
</tr>
<tr>
<td>Stage 5</td>
<td>0.050000 m/m/min</td>
</tr>
</tbody>
</table>

Page 2 of 3
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
</tr>
<tr>
<td></td>
<td>SA3 B</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Normal Stress</strong></td>
</tr>
<tr>
<td><strong>Peak Strength</strong></td>
</tr>
<tr>
<td><strong>Horizontal Deformation</strong></td>
</tr>
<tr>
<td><strong>Residual Stress</strong></td>
</tr>
<tr>
<td><strong>Vertical Deformation</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested By and Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Checked By and Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Approved By and Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>
# Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample SA3 B</td>
</tr>
</tbody>
</table>

## Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080 03 / AASHTO T236 92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>7d shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

## Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4130 in</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td></td>
<td>107.43 lb/ft³</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>57.55 lb/ft³</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>86.38 lb/ft³</td>
</tr>
<tr>
<td>Initial Water Content</td>
<td>107.43 lb/ft³</td>
</tr>
<tr>
<td>Degree of Saturation</td>
<td>80.32 %</td>
</tr>
<tr>
<td>Initial Voids Ratio</td>
<td>178.514</td>
</tr>
<tr>
<td>Final Water Content</td>
<td>49.63 %</td>
</tr>
<tr>
<td>Dry Mass</td>
<td>0.0675 lb</td>
</tr>
</tbody>
</table>

**Comments:**

*Calculated from initial and dry weights of whole specimen*

## Deformation vs Square Root Time

![Deformation vs Square Root Time Graph](image-url)
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement | Stage 1 0.010250 in/min

Page 2 of 3
# Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
</tr>
<tr>
<td>Lab Ref</td>
<td>Job</td>
</tr>
<tr>
<td></td>
<td>C18</td>
</tr>
<tr>
<td></td>
<td>Sample</td>
</tr>
<tr>
<td></td>
<td>SA3 B</td>
</tr>
</tbody>
</table>

## Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.33 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>4.18 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0804 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>0.00 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0007 in</td>
</tr>
</tbody>
</table>

- Tested By and Date
- Checked By and Date
- Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
</tr>
<tr>
<td>Sample Type</td>
</tr>
<tr>
<td>Lab. Temperature</td>
</tr>
<tr>
<td>Sample Description</td>
</tr>
<tr>
<td>Variations from procedure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen Reference</td>
</tr>
<tr>
<td>Depth within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
</tr>
<tr>
<td>Structure Preparation</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
</tr>
<tr>
<td>Description</td>
</tr>
<tr>
<td>Area</td>
</tr>
<tr>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Water Content</td>
</tr>
<tr>
<td>Dry Mass</td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen.

Deformation vs Square Root Time

<table>
<thead>
<tr>
<th>Time Square Root Mins</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
</tr>
<tr>
<td>0.024</td>
</tr>
</tbody>
</table>

Page 1 of 3
# Shear Strength by Direct Shear

(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA3B</td>
</tr>
</tbody>
</table>

## Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.34 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>7.49 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0382 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>3.62 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0079 in</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
<th>Lab Rel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA3 D</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080-03 / AASHTO T236 92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70°F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>residual prior to 30 day shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>Depth within Sample</td>
<td>0 0000m</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.3950 in</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td></td>
<td>103.86 lb/ft³</td>
</tr>
<tr>
<td></td>
<td>63.74 lb/ft³</td>
</tr>
<tr>
<td></td>
<td>102.68 lb/ft³</td>
</tr>
<tr>
<td></td>
<td>61.94 lb/ft³</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
</tr>
<tr>
<td>Comments</td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

```
Time Square Root Mins

0  5  10  15  20  25  30  35  40

0.00090 0.00099 0.00108 0.00117 0.00126 0.00135 0.00144 0.00153 0.00162

Deformation in

0.00001 0.00002 0.00003 0.00004 0.00005 0.00006 0.00007 0.00008 0.00009
```

Page 1 of 3
Change in Specimen Thickness vs. Displacement

Relative Level Displacement %

Shear Stress vs. Displacement

<table>
<thead>
<tr>
<th>Sample</th>
<th>Portion</th>
<th>Project</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3D</td>
<td>Job</td>
<td>Research</td>
<td>Lab Rack</td>
</tr>
</tbody>
</table>

(Small Shear Box)
Shear Strength by Direct Shear
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job C18</td>
<td>Borehole Sample SA3 D</td>
</tr>
</tbody>
</table>

Conditions at Failure

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.34 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>6.37 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0265 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>3.10 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0041 in</td>
</tr>
</tbody>
</table>

Tested By and Date
Checked By and Date
Approved By and Date
# Shear Strength by Direct Shear

(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample SA3 D</td>
</tr>
</tbody>
</table>

## Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080-03 / AASHTC T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Particle Specific Gravity</td>
<td>Single or Multi Stage</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>30 day shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

## Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>A</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000in</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.3660 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>4.83174 in²</td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>105.52 lbf/ft³</td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>64.76 lbf/ft³</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>108.60 lbf/ft³</td>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>65.53 lbf/ft³</td>
<td>Dry Mass</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weight of whole specimen

## Deformation vs Square Root Time

![Graph showing deformation vs square root time](image_url)
Shear Strength by Direct Shear  
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample SAA D</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By
and Date

Checked By
and Date

Approved By
and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA6 OCR</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3083-03 / AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Thin walled push in sample</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>Residual prior to 90 day shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>A</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000 in</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4160 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure - Preparation</td>
<td>Initial Water Content*</td>
<td>16.3 % (trammings 41.2 %)</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>111.37 lb/ft^3</td>
<td>Degree of Saturation 56.88 %</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>76.11 lb/ft^3</td>
<td>Initial Voids Ratio 134.74</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>112.28 lb/ft^3</td>
<td>Final Water Content 50.85 %</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>74.43 lb/ft^3</td>
<td>Dry Mass 0.0885 lb</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td>Comments</td>
</tr>
</tbody>
</table>

* *Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

[Graph showing deformation vs square root time]
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Juh</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA6 OCR</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
<tr>
<td>8.44 psi</td>
</tr>
<tr>
<td>7.98 psi</td>
</tr>
<tr>
<td>0.0120 in</td>
</tr>
<tr>
<td>3.67 psi</td>
</tr>
<tr>
<td>0.0035 in</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear  
(Small Shear Box)

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Client</td>
<td>Research</td>
<td>Lab Ref</td>
</tr>
<tr>
<td>Project</td>
<td>Job</td>
<td>Sample</td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>SA6 OCR</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>Sample Type</th>
<th>Particle Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM D3066-03 / AASHTO 1236-92</td>
<td>Thin walled push in sample</td>
<td>165 43</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>Lab. Temperature Location</td>
<td>Lab. Temperature Location</td>
</tr>
<tr>
<td>700 deg f</td>
<td>Single Stage</td>
<td></td>
</tr>
<tr>
<td>Sample Description</td>
<td>90 day shear specimen no B</td>
<td></td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
<td></td>
</tr>
</tbody>
</table>

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000 m</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.4160 in</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>Initial Water Content</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>111.37 lb/ft³</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>76.11 lb/ft³</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>114.54 lb/ft³</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>75.93 lb/ft³</td>
</tr>
<tr>
<td>Tested Dry or Submergec</td>
<td>Submergec</td>
</tr>
<tr>
<td>Comments</td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

Time Square Root Mins

Deformation

0.0001
0.0002
0.0003
0.0004
0.0005
0.0006
0.0007
0.0008
0.0009
0.0010

0
0.5
1
1.5
2
2.5
3

Graph showing deformation vs. square root of time with data points plotted at various time intervals.
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SAb CCR</td>
</tr>
</tbody>
</table>

Shear Stress vs Displacement

Change in Specimen Thickness vs Displacement

Rate of Horizontal Displacement: 1 in. / 100,000 in. 

Page 2 of 3
Shear Strength by Direct Shear  
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SA6 OCR</td>
</tr>
</tbody>
</table>

Conditions at Failure

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>644 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>734 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0015 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>0.00 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0006 in</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample SAE14</td>
</tr>
</tbody>
</table>

### Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080 0J; AASHTO T236 92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Core sample</td>
</tr>
<tr>
<td>Sample Description</td>
<td>Single or Multi Stage</td>
</tr>
<tr>
<td>Lab Temperature</td>
<td>70 0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>Swell then CONSOL then residual shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

### Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0.0000 ft</td>
</tr>
<tr>
<td>Initial Height</td>
<td>1 / 800 in</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>1.2 60 lbf ft3</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>78.43 lbf ft3</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>1.9 31 lbf ft3</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>79.81 lbf ft3</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>4 831/4 in2</td>
</tr>
<tr>
<td>Initial Water Content</td>
<td>13.6 % (trimmings 37.0 %)</td>
</tr>
<tr>
<td>Degree of Saturation</td>
<td>55.14 %</td>
</tr>
<tr>
<td>Initial Voids Ratio</td>
<td>130.722</td>
</tr>
<tr>
<td>Final Water Content</td>
<td>48.25 %</td>
</tr>
<tr>
<td>Dry Mass</td>
<td>0.1790 lb</td>
</tr>
</tbody>
</table>

*Calculated from initial and dry weights of whole specimen*

Deformation vs Square Root Time

![Graph of Deformation vs Square Root Time](image)
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Project</th>
<th>Job</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C18</td>
<td>SAE14</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

![Graph showing shear stress vs displacement with data points and a trend line.]

Change in Specimen Thickness Vs Displacement

![Graph showing change in specimen thickness vs displacement with data points and a trend line.]

Rate of Horizontal Displacement

<table>
<thead>
<tr>
<th>Stage 1</th>
<th>Stage 2</th>
<th>Stage 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.06598m/min</td>
<td>0.06598m/min</td>
<td>0.06598m/min</td>
</tr>
</tbody>
</table>

Relative Lateral Displacement %

0 10 20 30 40 50 60 70 80 90 100

Change in Thickness mm

-0.0016 -0.0006 0.0002 0.0004 0.0006 0.0008 0.0010 0.0012 0.0014 0.0016

0.0034 0.0036 0.0038 0.0040 0.0042 0.0044

Relative Lateral Displacement %

0 10 20 30 40 50 60 70 80 90 100

Shear Stress psi

-0.08 0.92 1.92 2.92 3.92 4.92 5.92

Change in Specimen Thickness mm
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SAE14</td>
</tr>
</tbody>
</table>

Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.42 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>6.24 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0260 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>3.15 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0092 in</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

**Client**
Pipjert

**Research**
Borehole Research Lab Jtef

**Job**
C18

**Sample**
SAE3C

### Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3056 03 AASHTO T936-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Core sample</td>
</tr>
<tr>
<td>Particle Specific Gravity</td>
<td>Single or Multi Stage</td>
</tr>
<tr>
<td>Location</td>
<td>Multi Stage 3 Stages</td>
</tr>
<tr>
<td>Lab Temperature</td>
<td>70 0 deg F</td>
</tr>
<tr>
<td>Sample Description</td>
<td>swell then consolidation residual shear</td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
</tr>
</tbody>
</table>

### Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>A</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth with sample</td>
<td>0.060 in</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0.780 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td></td>
<td>Initial Water Content* 42.6 % ( trapped 37.0 %)</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>108.5 lb/ft³</td>
<td>Degree of Saturation 52.91 %</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>7.06 lb/ft³</td>
<td>Initial Voids Ratio 133.075</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>119.0 lb/ft³</td>
<td>Final Water Content 50.89 %</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>8.6 lb/ft³</td>
<td>Dry Mass 0.1480 lb</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

### Deformation vs Square Root Time

<table>
<thead>
<tr>
<th>Time Square Root Mins</th>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformation in</td>
<td>0.0219</td>
<td>0.022</td>
<td>0.0221</td>
<td>0.0222</td>
<td>0.0223</td>
<td>0.0224</td>
<td>0.0225</td>
<td>0.0226</td>
</tr>
</tbody>
</table>
Shear Strength by Direct Shear
(Small Shear Box)

Client: Borehole
Project: Research
Lab Ref: Sample
Job: C15
Sample: SAE30

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement:
Stage 1: 0.065934 in/min
Stage 2: 0.065934 in/min
Stage 3: 0.032065 in/min
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td></td>
<td>Job C18</td>
</tr>
<tr>
<td>Borehole</td>
<td></td>
<td>Sample SAE30</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Conditions at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
</tr>
<tr>
<td>Peak Strength</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
</tr>
<tr>
<td>Residual Stress</td>
</tr>
<tr>
<td>Vertical Deformation</td>
</tr>
</tbody>
</table>

Tested By and Date

Checked By and Date

Approved By and Date
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Project</th>
<th>Lab Ref</th>
<th>Job</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Research</td>
<td>C18</td>
<td>SAF60</td>
<td></td>
</tr>
</tbody>
</table>

### Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3050-03</th>
<th>AASHTO T236-92</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Core sample</td>
<td>Single or Multi Stage</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>700 deg F</td>
<td>Multi Stage 2 Stages</td>
</tr>
<tr>
<td>Sample Description</td>
<td>swell then consol then residual shear</td>
<td></td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
<td></td>
</tr>
</tbody>
</table>

### Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth within Sample</th>
<th>Orientation within Sample Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>30000 in</td>
<td>4.83174 in²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Structure / Preparation</th>
<th>Initial Water Content*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>39.9 % (trimming 32.7 %)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Wet Unit Weight</th>
<th>Initial Dry Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>112.60 lb/ft³</td>
<td>90.50 lb/ft³</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Final Wet Unit Weight</th>
<th>Final Dry Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>117.67 lb/ft³</td>
<td>117.78 lb/ft³</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Degree of Saturation</th>
<th>Initial Voids Ratio</th>
<th>Final Water Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>51.82 %</td>
<td>127.345</td>
<td>43.87 %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dry Mass</th>
<th>Submerged</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1756 lb</td>
<td></td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

---

**Deformation vs Square Root Time**

![Graph showing deformation vs square root time]
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SAE69</td>
</tr>
</tbody>
</table>

Shear Stress Vs Displacement

Change in Specimen Thickness Vs Displacement

Rate of Horizontal Displacement

<table>
<thead>
<tr>
<th>Rate</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sl</td>
<td>1.0669</td>
</tr>
</tbody>
</table>
### Shear Strength by Direct Shear
*(Small Shear Box)*

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SAE60</td>
</tr>
</tbody>
</table>

#### Conditions at Failure

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>642 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>6.61 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0250 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>5.29 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0112 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested By</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Checked By</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Approved By</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Shear Strength by Direct Shear
(Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SAE60</td>
</tr>
</tbody>
</table>

Test Details

<table>
<thead>
<tr>
<th>Standard</th>
<th>ASTM D3080-03 · AASHTO T.24-92</th>
<th>Particle Specific Gravity</th>
<th>165.43</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Type</td>
<td>Core sample</td>
<td>Single or Multi Stage</td>
<td>Single Stage</td>
</tr>
<tr>
<td>Lab. Temperature</td>
<td>70.0 deg F</td>
<td>Location</td>
<td></td>
</tr>
<tr>
<td>Sample Description</td>
<td>60 day shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Variations from procedure</td>
<td>None</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Specimen Details

<table>
<thead>
<tr>
<th>Specimen Reference</th>
<th>B</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth within Sample</td>
<td>0 000 in</td>
<td>Orientation within Sample</td>
</tr>
<tr>
<td>Initial Height</td>
<td>0 780 in</td>
<td>Area</td>
</tr>
<tr>
<td>Structure Preparation</td>
<td></td>
<td>Initial Water Content*</td>
</tr>
<tr>
<td>Initial Wet Unit Weight</td>
<td>112 60 lb/ft³</td>
<td>Degree of Saturation</td>
</tr>
<tr>
<td>Initial Dry Unit Weight</td>
<td>80 50 lb/ft³</td>
<td>Initial Voids Ratio</td>
</tr>
<tr>
<td>Final Wet Unit Weight</td>
<td>116 30 lb/ft³</td>
<td>Final Water Content</td>
</tr>
<tr>
<td>Final Dry Unit Weight</td>
<td>80 84 lb/ft³</td>
<td>Dry Mass</td>
</tr>
<tr>
<td>Tested Dry or Submerged</td>
<td>Submerged</td>
<td>Comments</td>
</tr>
</tbody>
</table>

* Calculated from initial and dry weights of whole specimen

Deformation vs Square Root Time

![Graph showing deformation vs square root time]
Shear Stress vs Displacement

Change in Specimen Thickness vs Displacement

Rate of Horizontal Displacement: max 1.04659 m/min
Shear Strength by Direct Shear (Small Shear Box)

<table>
<thead>
<tr>
<th>Client</th>
<th>Research</th>
<th>Lab Ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Job</td>
<td>C18</td>
</tr>
<tr>
<td>Borehole</td>
<td>Sample</td>
<td>SAE60</td>
</tr>
</tbody>
</table>

### Conditions at Failure

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Stress</td>
<td>6.42 psi</td>
</tr>
<tr>
<td>Peak Strength</td>
<td>6.35 psi</td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0.0171 in</td>
</tr>
<tr>
<td>Residual Stress</td>
<td>0.00 psi</td>
</tr>
<tr>
<td>Vertical Deformation</td>
<td>0.0031 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tested By and Date</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Checked By and Date</td>
<td></td>
</tr>
<tr>
<td>Approved By and Date</td>
<td></td>
</tr>
</tbody>
</table>


REFERENCES


Titi, Hani H. and Wije Wathugala. *Numerical Procedure for Predicting Pile Capacity-Setup/Freeze.* Transportation Research Record #1663, 1999, 25-32.


