Cone Penetration in Cemented Deposits-A Field Study

Semih Arslan
Louisiana State University and Agricultural and Mechanical College

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CONE PENETRATION IN CEMENTED DEPOSITS-
A FIELD STUDY

A Thesis

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

in
The Department of Civil Engineering

by
Semih Arslan
B.S., Istanbul Technical University, 1989
August, 1993
MANUSCRIPT THESES

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This thesis is dedicated to my parents.

To My Mom and Dad

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ABSTRACT

Neglecting cementation results in an underestimation of slope stability and pile capacity in design and analysis of geotechnical engineering problems related to cemented deposits. Developing methods to identify and assess the engineering characteristics of cemented deposits is an important subject in geotechnical engineering.

This study is conducted to evaluate the reliability of the prediction schemes proposed by previous investigators through a field study of cone penetration in a cemented deposit and respective laboratory triaxial study on undisturbed cemented specimens. The effect of cementation on cone penetration resistance is investigated through the field study, and the effects of cementation on drained behavior of naturally and artificially cemented specimens are also investigated through the laboratory study. The repeatability and accuracy of the calibration chamber test results obtained by Puppala (1993) are investigated by conducting five cone penetration tests in the calibration chamber.

The research approach involves a field study of cone penetration resistance in Waterways Experiment Station (WES) on loess deposit and also a laboratory study of Isotropically Consolidated-Drained (CID) triaxial tests on naturally cemented specimens obtained from the same deposits at WES. Tip and sleeve resistances are predicted using the laboratory test results and Durgunoglu & Mitchell (1973) and Janbu & Senneset (1974) bearing capacity theories. The chart of plastification angle versus dilation angle and the chart of dilation angle versus K/K₀ proposed by Puppala (1993) are used in predictions. The predicted resistances are compared with measured resistances to evaluate the reliability of the theories and charts proposed by previous investigators for cemented deposits.
The investigation indicated that the predicted tip and sleeve resistances show varying degrees of agreement with the measured values. The prediction methods give excellent results in the second layer of the bluff (7-15 m). However, the predicted results show lower values in the first layer (0-7 m).
CHAPTER 1. INTRODUCTION

Cemented sands are found in many areas of the world. They cover large areas in the U.S. and many areas like the Middle East, Southeast Asia, and Africa. Large deposits of cemented sands exist along the California coast. Also, bluffs called "loess" deposits occur along the Mississippi Valley. These bluffs are typically cemented fine sands or cemented silty clay deposits. In this region, loess exists within an area that is 10 to 15 miles in width extending from southern Illinois southward along the eastern walls of the lower Mississippi River alluvial valley to near Bayou Sara in Louisiana (Krinitzsky, 1950). Plate 1.1 shows a steep bluff of cemented loess located in the US Army Corps of Engineers Waterways Experiment Station in Vicksburg, Mississippi. The present study is conducted on this bluff. Plate 1.2 is a photograph of a cemented loess bluff located in Natchez Trace Park, Mississippi.

The presence of cementing agents such as silica or silicious cement, calcium carbonate or calcareous cement, clay or argillaceous cement is the primary source of cementation in soil (Krynine and Judd, 1957). The welding between the particles at their contact points due to internal heat at the time of deposition also causes cementation (Lee, 1975).

The effect of cementation on the strength and deformation behavior of soil is very important in the design and analysis in geotechnical engineering systems constructed on or in cemented deposits. Cementation bonds which generally produce a cohesion intercept are generally neglected in design and analysis of geotechnical engineering systems in cemented deposits for practical purposes. However, neglecting the effect of cementation results in an underestimation of the strength of the soil deposits as well as an underestimation of the stability of the slopes (Rad and Clough, 1982; Poulos, 1980; Frydman et al., 1980). Large
Plate 1.1: A View of the Cemented Loess Bluff in Waterways Experiment Station in Vicksburg, Mississippi.
Plate 1.2: A View of the Cemented Loess Bluff Located in Natchez Trace Park, Mississippi.
deposits of cemented sands have the ability to stand in steep natural slopes due to the bonds of cementation between the sand particles. The slopes of these deposits are 60 degrees or steeper and the heights of the slopes can be up to 100 m (Clough et al., 1981). Slope failures are very common in these deposits as the result of earthquakes or heavy rains.

Another factor that makes cementation important in geotechnical engineering problems is that cementation is also used to stabilize weak sandy deposits. The addition of a small amount of cementing material such as portland cement improves the engineering properties of sands. This procedure is often used in engineering projects for improvement of subgrades and airport runways, stabilizing the slopes and embankments, and increasing the bearing capacity of the soil. Hence, it is very important to understand the behavior of naturally and artificially cemented deposits.

The objectives of this research are: (1) to evaluate the effect of cementation on cone penetration resistance through a field study in a cemented deposit; (2) to evaluate the effect of cementation on drained behavior of naturally and artificially cemented specimens through laboratory testing; (3) to provide a preliminary assessment of the reliability of the prediction schemes developed by the calibration chamber testing; and (4) to assess the repeatability of the calibration chamber test results conducted by Puppala (1993).

The effect of cementation on cone penetration resistance is investigated through a field study. The effects of cementation on the drained strength behavior of naturally and artificially cemented specimens are investigated through a laboratory triaxial test program. Tip and sleeve resistances are predicted by using Durgunoglu and Mitchell (1973), and Janbu and Senneset (1974) bearing capacity theories. They are compared with measured tip and sleeve resistances to evaluate the reliability of the prediction schemes. The influence of different
levels of cementation, relative density, and confining pressure on static drained strength parameters of artificially cemented and uncemented Monterey No. 0/30 Sand are also discussed. The repeatability and accuracy of the calibration chamber test results obtained by Puppala (1993) are investigated by conducting five cone penetration tests in the calibration chamber.

The scope of the research involves a field study of cone penetration resistance on loess deposits in Waterways Experiment Station (WES) using the Research Vehicle for Geotechnical Insitu-Testing (REVEGITS) and a laboratory study of Isotropically Consolidated-Drained (CID) triaxial tests on naturally cemented specimens obtained from the same deposits at WES. Five cone penetration tests in the calibration chamber were conducted to complement the previous calibration chamber study of Puppala (1993). Three tests were conducted to assess the performance of the newly purchased and calibrated triaxial equipment by comparing the results to those of Rad (1984). Artificially cemented specimens, 1 and 2 percent portland cement content, and relative density ranging from 55 to 88 percent, and similar but uncemented specimens were tested. They were cured for 7 and 14 days and tested at confining pressures of 100, 200, and 300 kPa.
CHAPTER 2. LITERATURE REVIEW

This chapter covers definition of cementation, factors causing cementation, locations for cemented deposits and the properties of cemented soils. Some of the studies conducted by previous investigators and their conclusions on the static behavior of cemented sands and cone penetration in calibration chamber are also summarized.

2.1 Cementation

Cementation is the term used to describe the bonds that cause coherence between soil particles which occur in many sand and silt deposits (El-Tahir 1985). Two factors cause this phenomenon:

1. "the welding between the particles at their contact points due to internal heat at the time of deposition or due to prolonged pressure at prominent points of contact between grains" (Lee 1975).

2. "the presence of cementing agents like silica or silicous cement, calcium carbonate or calcareous cement, clay or argillaceous cement "(Krynine and Judd 1957).

2.2 Loess Deposits

Krinitzsky (1950) presented a general picture of loess in the lower Mississippi Valley. His study indicated that "loess is recognized as unstratified, calcareous, slightly plastic, porous loam with an average grain size diameter ranging between 0.05 and 0.01 mm. It occurs within an area averaging 10 to 15 miles in width extending from southern Illinois southward along the eastern walls of the lower Mississippi River alluvial valley to near Bayou Sara in Louisiana". Wascher, and collaborators (1947) published a map of loess deposits in the lower
Mississippi Valley. Figure 2.1 shows the distribution of calcareous loess in the Mississippi Valley. Krinitzky (1947) sampled within the unweathered calcareous portions from the vertical sections of loess at Natchez and Greenwood, Mississippi. The samples were taken at three different levels, the base, mid-portion, and the top of the loess. The chemical analyses on these samples showed that there is a high percentage of calcium carbonate and these samples are very uniform. Krinitzky (1947) reported that "Carbonates in loess of the Lower Mississippi Valley are almost entirely secondary, chiefly in the form of precipitates around grass roots. These grass roots are inferred to be the result of precipitation of calcareous salts around roots by a process in which the roots absorb ground water but reject certain salts dissolved in the water. An increase in saturation of the CaCO$_3$ in the water near a root to the point of supersaturation can cause precipitation. Also a decrease in hydrogen ion concentration by the roots likewise may cause precipitation. The amount of carbon dioxide dissolved in ground water influences the pH. When roots take up CO$_2$, CaCO$_3$ is precipitated. Either of these conditions, or a combination of the two, is sufficient to explain the resulting concretionary deposits."

2.3 Static Behavior of Cemented Sands

Rad (1982) studied the strength and deformation characteristics of cemented sands by conducting 43 static strain controlled drained or undrained triaxial tests. Pure sands and artificially cemented sands with 1 or 2 percent portland cement, and relative densities of 25 to 80 percent were used. A curing period of 14 days was used for the cemented specimens. The results of the consolidated drained triaxial tests conducted on artificially cemented sand with one percent cement at relative density of 80 % are shown in Figure 2.2. The results of static drained triaxial tests on cemented and uncemented Monterey No.
Figure 2.1: The Distribution of Calcareous Loess in the Mississippi Valley
(Washer et al., 1947)
Figure 2.2: The Results of Drained Tests on 1% Cemented and 14 Days Cured Specimens (Rad, 1984)
0 Sand are provided in Table 2.1. The conclusions from Rad's (1984) study are summarized as follows:

1. As confining pressure increases, brittleness, volumetric expansion, and negative pore water pressure development decrease. However, strength and initial tangent modulus increase.

2. As relative density increases, brittleness, volumetric expansion, negative pore pressure development, friction angle, and cohesion intercept also increase.

3. As amount of cementation increases, the brittleness, volumetric expansion, negative pore pressures, and cohesion intercept increase. However, the friction angle is relatively unaffected.

Table 2.1: Results of Static Drained Triaxial Tests on Uncemented and Cemented Monterey No. 0 Sand Specimens Cured for 14 days (Rad, 1984)

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<td>D_r (%)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>31</td>
<td>45</td>
<td>77</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>50</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>50</td>
<td>80</td>
</tr>
<tr>
<td>Peak</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Strength</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \phi ) (degrees)</td>
<td>33</td>
<td>35</td>
<td>39</td>
</tr>
<tr>
<td>c' (kPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>9</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Residual</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \phi ) (degree)</td>
<td>33</td>
<td>34</td>
<td>35</td>
</tr>
<tr>
<td>c' (kPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>12</td>
<td>14</td>
</tr>
</tbody>
</table>

C.C.: Cement Content, D_r: Relative Density, \( \phi \): Friction Angle, c: Cohesion Intercept
Clough et. al., (1980) conducted a study on both naturally cemented sands and artificially cemented sands. A total of 137 laboratory tests were conducted on four naturally cemented sands found in the San Francisco Bay Area and artificially cemented sands prepared in the laboratory to simulate the natural soil. Natural soil samples were obtained by hand cutting large blocks of the material. The samples varied from very strong to very weak. The results of the tests show that the amount of cementing agent, sand density, confining pressure, and grain size distribution strongly affect the behavior of a cemented sand. They also concluded that many of the natural materials in some very steep and high bluffs contain a small amount of cementation. Some of the conclusions from this study are listed below.

1. "Introduction of a cementing agent into a sand produced a material with two components of strength- that due to the cement itself and that due to friction. The friction angle of a cemented sand is similar to that of uncemented sands".

2. "A weakly cemented sand shows a brittle failure mode at low confining pressures with a transition to ductile failure at higher confining pressures".

3. "Volumetric increases during shear occur at a faster rate and at a smaller strain for cemented sands than uncemented sands".

4. "The residual strength of a cemented sand is close to that of an uncemented sand".

2.4 Unconfined Compressive Strength, $q_{u}$

Rad and Clough (1985) proposed a classification of cemented sands based on unconfined compressive strength. The classification proposed in Table 2.2 is considered valid for all cemented soils irrespective of the cementing agent.
Table 2.2: The Classification of Cemented Sands by Rad and Clough. (1985)

<table>
<thead>
<tr>
<th>Classification</th>
<th>$q_u$(kPa)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY WEAKLY CEMENTED</td>
<td>&lt;100</td>
<td>cementation almost unapparent to touch</td>
</tr>
<tr>
<td>WEAKLY CEMENTED</td>
<td>100-300</td>
<td>breaks down under slight finger pressure; can be scratched with the finger tip</td>
</tr>
<tr>
<td>MODERATELY CEMENTED</td>
<td>300-1000</td>
<td>hardly breaks under finger pressure; can be easily scratched with the finger nail</td>
</tr>
<tr>
<td>STRONGLY CEMENTED</td>
<td>1000-3000</td>
<td>difficult to trim, can be hardly scratched with the finger nail</td>
</tr>
<tr>
<td>VERY STRONGLY CEMENTED</td>
<td>&gt;3000</td>
<td>very low strength soft rock</td>
</tr>
</tbody>
</table>

2.5 Cone Penetration Testing

The Beringen et al. (1982) study on both cone penetration testing and laboratory testing of calcareous marine sediments revealed that insitu-testing (cone penetration testing) can dramatically improve soil classification in marine deposits. The study also showed that cone penetration test results from cemented (carbonate) soils can be interpreted using the principles established for noncarbonated soils. Many examples are quoted in this study to show the importance of performing cone penetration testing when strength parameters are needed for design.
Rad and Tumay (1986) conducted a limited study of the effect of cementation on cone penetration resistance of sands. These tests were conducted in a rigid wall chamber. The tests were valid for very low confining pressures (<5 kPa) since they were conducted under no other external confinement. The major conclusions from this study were: "(1) cementation has a pronounced effect on cone penetration resistance of sand. Increasing the cement content increases the tip resistance and the sleeve friction, while decreasing the friction ratio. This behavior is similar to that of the relative density on cemented sands. This increase in tip resistance and sleeve friction is attributed to the increase in cohesion intercept in cemented sand; (2) the correlation between the internal friction angle and the cone penetration resistance of cemented sands depends strongly on the cement content. Specimens with similar friction angles but different cement contents show higher tip resistance and sleeve frictions and lower friction ratios; (3) the effect of cementation on cone penetration resistance of sand is similar to that of relative density. Utilizing the available correlation for uncemented sands to estimate the relative density or internal friction angle of naturally deposits possibly cemented sands can be possibly misleading. Generally existing correlations would suggest values of relative density and internal frictional angle higher than those actually available for the cemented sand".

Puppala (1993) conducted cone penetration tests using a large scale calibration chamber at Louisiana State University. Specimens having the diameter of 0.53 m and the height of 0.79 m were tested using a miniature quasi-static penetrometer of 1.27 cm in diameter. Monterey No. 0/30 Sand and ordinary portland cement were used. The following variables were used in the study: 0, 1 and 2 percent cement content; relative densities of 45-55, 65-75, and above 80%; a curing period of 7 days; and confining stresses of 100, 200, and 300 kPa. The following conclusions were given for this study: "(1) tests conducted with a
piezocone showed that there is no excess pore pressure developed during testing. This implied that the reduction in hydraulic conductivity due to cementation will not result in undrained conditions during cone penetration; (2) cementation increased tip resistance due to development of cohesion and friction resistance due to dilation; (3) two empirical approaches were suggested to interpret strength parameters of cemented sands. These approaches are based on the availability of obtaining block samples from the field. Predictions of present cone test results showed a good agreement with the measured strength parameters. Insitu data are still needed to validate these approaches; (4) in the theoretical approaches, two bearing capacity theories were used. Durgunoglu and Mitchell (1973) predictions showed a good agreement with measured values. The rigid plasticity assumption was used in the bearing capacity theories. Cemented and uncemented sands showed this rigid plastic behavior at and around peak stress, hence the theory which used peak strength parameters quite well predicted the measured resistances. (5) Janbu and Senneset (1974) predictions depend upon the plastification angle. Estimation of plastification angles are formalized by providing a correlation between the plastification angle and the dilation angle. This theory also rendered quite good comparisons. (6) bearing capacity theories and sleeve friction predictions are used in formulating a semi-empirical approach to predict cemented soil characteristics. This approach predicts cohesion and relative density based on the normalized cone tip resistance and friction ratio. Once the relative density is obtained, friction angle can be estimated from $D_r$-$\phi$ correlations".

It is necessary to conduct field tests to validate the findings of this study even though the effect of cementation on cone penetration testing can be predicted reasonably well with proposed theoretical models.
2. 6 Cone Penetration Testing Analysis

Bearing capacity theories, cavity expansion theories, strain path approach, and numerical methods have been used by many investigators in the penetration analysis in estimating tip resistance. In this study, two bearing capacity theories (Durgunoglu & Mitchell, 1973 and Janbu & Senneset 1974), are used to predict the cone tip resistance. In the following section, these theories and a method used in calculation of sleeve resistance are explained.

2. 6. 1 Bearing Capacity Theories

Durgunoglu and Mitchell (D & M) (1973) modified the Terzaghi (1943) bearing capacity equation. They considered the effect of symmetry, foundation shape and roughness and proposed the following equation.

\[ q_{uc} = c \cdot N_c \cdot \zeta_c + \gamma \cdot B \cdot N_{\gamma q} \cdot \zeta_{\gamma q} \quad (2.1) \]

where \( N_{\gamma q} \) is the bearing capacity factor for the friction-surcharge term and it depends upon the soil friction angle \( \phi \), base semi-apex angle \( \alpha \), base roughness \( \delta/\phi \) and relative depth of penetrometer base D/B. \( \zeta_c \), \( \zeta_{\gamma q} \) represent the corresponding shape factors and are calculated using the Brinch-Hansen (1961) parameters. The assumed failure mechanism is shown in Figure 2.3.

Janbu and Senneset (J & S) (1974) assumed plane strain conditions. The assumed failure mechanism is given in Figure 2.4. The failure surface fans out to different planes of plastification, \( \beta \), depending upon the dilational characteristics of the soil. For loose sands and normally consolidated clays, the values of plastification angle are assumed between 0 to 30 degrees, and for dense and over consolidated clays, the values are between 0 to - 40 degrees. They expressed the equation as follows:
Figure 2.3: Assumed Failure Mechanism in Durgunoglu and Mitchell (1973) Theory
Figure 2.4: Assumed Failure Mechanism in Janbu and Senneset's Theory (1974)
\[ q_v + a = N_q \cdot \left( \sigma'_v + a \right) + u_0 - N_u \cdot \Delta u_b + 1/2 \cdot \gamma \cdot B \cdot N_y \quad (2.2) \]

where \( q_v \) is vertical ultimate bearing capacity, \( N_y \) and \( N_q \), \( N_u \) are bearing capacity factors, \( u_0 \) is initial pore pressure and \( \Delta u_b \) is pore pressure at foundation base, \( a \) is attraction that represents the maximum tensile strength intercept, \( \sigma'_v \) is effective vertical stress at the depth of penetrometer and \( B \) is width or diameter of footing or penetrometer.

Janbu & Senneset (1974) theory considers the excess pore pressure effects along the shear surface on the bearing capacity of the cone. The bearing capacity factors are derived from the equilibrium of the given shear surface geometry (Senneset et al., 1982).

### 2.6.2 Sleeve Friction

Sleeve friction is generated due to shear resistance along the sides of the penetrometer. The following formula

\[ f_s = S_s \cdot (\sigma'_v + a) \quad (2.3) \]

is used for the calculation of sleeve friction. In the formula, \( f_s \) is friction resistance, \( S_s \) is \((r \cdot \tan \phi) \ast K\), where \( r \) is interface friction ratio or the roughness coefficient(\( \tan \delta / \tan \phi \)), and is taken as 0.65 in this research study. \( K \) is the earth pressure coefficient, and it is taken as the \( K_0 \) in the pile friction capacity calculations. In the present study, the chart of dilation angle versus \( K/K_0 \) proposed by Puppala (1993) is used to estimate \( K \) values. The upper limit for the roughness coefficient is taken as 1.0 and the lower limit is taken as 0.55 in practice (Janbu, 1976; Acar et al., 1982).
Acar and Tumay, (1986) reported that "the strains in the vicinity of the sleeve are well beyond the strains corresponding to peak strength values". Therefore, using residual strength values of the soil in estimating friction resistance would be more appropriate when bearing capacity theories are used for calculation.
CHAPTER 3. METHODOLOGY

This chapter describes the equipment used, specimen preparation technique, the undisturbed specimen retrieval methods, and the testing procedures used in this investigation. Table 3.1 shows the number of tests conducted on naturally cemented loess deposits. Table 3.2 presents the tests conducted on artificially cemented (1%) specimens cured for 14 days in order to assess the performance of the newly purchased and calibrated triaxial equipment by comparing the results with that of Rad (1984). Table 3.3 presents the number of tests conducted on artificially cemented and uncemented Monterey No. 0/30 sand. Naturally cemented specimens are obtained from the loess bluff located in Waterways Experiment Station (WES) in Vicksburg, Mississippi. This bluff where cone penetration soundings are also conducted is shown in Plate 1.1. Undisturbed naturally cemented specimens are obtained from two different levels in the slope. First level is at 0-0.50 m depth named as WES 2 and second level is at 7-7.5 m depth named as WES 1. Cone penetration soundings of 10 to 20 m deep were also conducted on this bluff using the Louisiana Transportation Research Center (LTRC) Research Vehicle for Geotechnical Insitu-testing.

3.1 Equipment

3.1.1 LSU LoadTrac System for Triaxial Testing

The LoadTrac system was recently purchased by the LSU Civil Engineering Department. This machine was manufactured by GEOCOMP Corporation. The LoadTrac system consists of a load frame, pore pressure, vertical displacement, and load cell (force) transducers, a microprocessor for test control and data acquisition, and IBM PC compatible software to set up the tests conditions and store the test results in a file on disk for analyses and plots. The
Table 3.1: Testing Program on Naturally Cemented (Loess) Deposits

<table>
<thead>
<tr>
<th>Site</th>
<th>Hydrometer</th>
<th>CID Triaxial</th>
<th>Unconfined Cone Compression</th>
<th>Penetration Soundings</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.25 cm/sec</td>
<td>1 cm/sec</td>
</tr>
<tr>
<td>WES 1</td>
<td>2</td>
<td>7</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>WES 2</td>
<td>1</td>
<td>6</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>3</td>
<td>13</td>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2: Testing Program on Artificially Cemented (1%) Monterey No. 0/30 Sand Specimens Cured for 14 Days.

<table>
<thead>
<tr>
<th>Cement Content (%)</th>
<th>Confining Pressure (kPa)</th>
<th>Void Ratio</th>
<th>Relative Density (%)</th>
<th>CID Triaxial Tests</th>
<th>Total Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100 200 300</td>
<td>0.60</td>
<td>88</td>
<td>1 1 1</td>
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</table>
Table 3.3: Testing Program on Artificially Cemented and Uncemented Monterey No. 0/30 Sand Specimens Cured for 7 Days.

<table>
<thead>
<tr>
<th>Cement Content C.C. (%)</th>
<th>Confining Pressure (kPa)</th>
<th>Void Ratio</th>
<th>Relative Density (%)</th>
<th>CID Triaxial Cone Penetration (Chamber)</th>
<th>Cone Penetration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
<td>0.69</td>
<td>57</td>
<td>1</td>
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<tr>
<td></td>
<td>200</td>
<td>0.69</td>
<td>55</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>0.69</td>
<td>55</td>
<td>1</td>
<td></td>
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<tr>
<td></td>
<td>100</td>
<td>0.69</td>
<td>57</td>
<td>1</td>
<td></td>
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<tr>
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<td>200</td>
<td>0.69</td>
<td>55</td>
<td>1</td>
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</tr>
<tr>
<td></td>
<td>300</td>
<td>0.69</td>
<td>55</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>0.68</td>
<td>60</td>
<td>1</td>
<td></td>
</tr>
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<td></td>
<td>200</td>
<td>0.68</td>
<td>60</td>
<td>1</td>
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<tr>
<td></td>
<td>300</td>
<td>0.68</td>
<td>60</td>
<td>1</td>
<td></td>
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<tr>
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<td>0.65</td>
<td>69</td>
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<td>69</td>
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<td>1</td>
<td></td>
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<tr>
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<td>70</td>
<td>1</td>
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<tr>
<td></td>
<td>100</td>
<td>0.59</td>
<td>91</td>
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<tr>
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<td>87</td>
<td>1</td>
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<tr>
<td></td>
<td>300</td>
<td>0.59</td>
<td>89</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
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<td>0.60</td>
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<td>1</td>
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<tr>
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<td>0.60</td>
<td>87</td>
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<td>0.59</td>
<td>89</td>
<td>1</td>
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<tr>
<td></td>
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<td>87</td>
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</tr>
<tr>
<td></td>
<td>300</td>
<td>0.60</td>
<td>90</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>27</td>
<td>5</td>
</tr>
</tbody>
</table>
control panel and volume change transducer connected to the LoadTrac system are manufactured by ELE. The vacuum pump manufactured by Welch Company was used to apply confinement to the uncedmented sands specimens. A suction of up to 100 kPa can be generated by the vacuum pump. Schematic of the LSU static triaxial LoadTrac system and other equipment connected to the system are shown in Figure 3.1. A general view of the triaxial room is presented in Plate 3.1.

The LoadTrac hardware consists of six distinct parts: (1) LoadTrac load frame; It contains the components that generate the force on a sample, measure the force on a sample, and measure the displacement of a sample. (2) triaxial cell; It confines the sample inside an impermeable membrane. (3) signal conditioning unit; This contains electronics for sensor excitation and sensor signal conditioning. (4) CPU; it is a microprocessor with memory, analog and digital signal card, hard disk and floppy disk. (5) keyboard; this is a 84 key standard keyboard for inputting data and controlling LoadTrac. (6) display; it is a 12" amber monitor with tilt-swivel base (LoadTrac Operator's Manual, GEOCOMP Corp., 1988).

The LoadTrac software has two separate programs, LoadTrac or LT, and TRIAX. LoadTrac program is for running the test, collecting the tests data, and placing the data in a disk file. TRIAX program is for reading the data from the disk file, performing the necessary calculations and preparing the final tables and graphs of the test results. LoadTrac program is used in this study for testing and the test data is analyzed using Grapher and Lotus 123 software for calculations and preparations of the graphs.

LoadTrac system automatically controls the conduct of triaxial tests from start to end. Specimens of up to 2.8" (7.11 cm) in diameter and up to 6" (15.24 cm) in height can be tested in this system. The system can apply a constant rate of strain at any strain rate up to 1% of the height of the specimen per minute. The
Figure 3.1: Schematic of the Triaxial LoadTrac System with all Connected Equipment
Plate 3.1: A General View of the Triaxial Room
vertical load is applied to the specimen by a high speed, precision stepper motor. The microprocessor takes readings from the force transducer to control the motor.

The SCU used in the LoadTrac triaxial system has components for 8 channels. The channels, transducers and, their calibration factors that are used in this investigation are given in Table 3.4.

Table 3.4: Transducers, Their Channel Numbers, and the Calibration Factors Used in the Triaxial Testing Program

<table>
<thead>
<tr>
<th>Channel No.</th>
<th>Transducer</th>
<th>Calibration Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>load cell for vertical force</td>
<td>0.5173 lb/bit</td>
</tr>
<tr>
<td>2</td>
<td>pore water pressure</td>
<td>0.03727 lb/bit</td>
</tr>
<tr>
<td>3</td>
<td>volume change</td>
<td>0.1205 cm³/bit</td>
</tr>
<tr>
<td>4</td>
<td>LVDT for vertical displacement</td>
<td>0.0003097 in/bit</td>
</tr>
</tbody>
</table>

Channel numbers 1, 5, 6, and 7 are not used. Some changes are made on the channels. The Channel no. 3 is set up for pore pressure transducer for consolidated undrained tests data in the factory and the software can only read the data from channel no. 0, 3, and 4. In this investigation, the volume change transducer is needed to be installed to obtain volume change data for
consolidated drained test. Therefore, the change is made and Channel No. 3 is set up and used for volume change transducer. During the saturation phase, Channel No. 2 is used for pore pressure transducer for observation of pore water pressure increments.

3.1.2 Unconfined Compression Testing Equipment

The test equipment is manufactured by ELE. Unconfined compression tests can be done with specimens of different sizes up to 7 cm (2.75") in diameter and at different speeds varying from 2 to 9 cm/sec.

3.1.3 LSU Calibration Chamber Facilities

Pluviation setup that is shown in Figure 3.2 is used for specimen preparation. Pluviation method simulates the depositional process in natural deposits. Bellotti. et. al., (1991) reported that "pluviation not only provides homogeneous specimens with the desired relative density but also simulates a soil fabric most similar to the one found in natural deposits formed by sedimentation".

The setup consists of three parts. The sand is placed in the top chamber. The middle chamber is for necessary height of fall for the sand leaving the top chamber. The bottom chamber is a specimen mold that the sand is deposited and the specimen is formed in. The specimen mold is a diametrically split mold. The mold sits on a wooden four wheel trolley so that it can be moved. The middle and the top parts are fixed on to a table. An aluminum shutter having a set of two plates with identical holes separates the top and middle parts. Rotating the bottom plate allows the sands to start falling and getting pluviated, and rotating the bottom plate opposite direction closes the aluminum shutter.
Figure 3.2: Schematic of the Pluviation Setup (Puppala, 1993)
For saturation purpose of the cemented specimens, a 50 gallon water tank, a carbon dioxide cylinder and a set of tubing connections are used.

The LSU calibration chamber system was first assembled by de Lima (1990). Then, Puppala (1993) calibrated and used in his investigation on cemented sands. The chamber is of 1.78 m in height and 0.64 m in diameter. The calibration chamber system, that is shown in Figure 3.3, consists of the piston cell and the chamber cell unit. The chamber cell is double walled flexible steel cylinder. This units sits on a bottom plate. A vertical stress is applied to the specimen with the piston that pushes the bottom plate upwards. The sample cell can house a specimen of 0.53 m in diameter and 0.79 m in height. The chamber top plate, sample cell inner and outer walls and the piston cell ring are tightened together after the sample is placed in the sample cell. The space between the sample, and inner wall, and the space between inner and outer walls are filled with deaired water in order to apply horizontal stress.

The miniature quasi-static cone penetrometer (MQSC) is used in the calibration chamber. MQSC has a 6.3 cm long friction sleeve and apex angle of 60 degrees. Its cross-sectional area is 1.27 cm², and its push rod is 9.53 mm in diameter and 1.82 m in length. Table 3.5 gives the calibration factors used for MQSC.

Pressure regulators, electro-pneumatic transducers, pressure transducers, pressure gauges are connected to the control panel which is a wooden panel of 1.22 m by 1.96 m. There are four electro-pneumatic transducers in the control panel. Two of them having the pressure range of 40 to 215 kPa are used for applying vertical stress, and the other two having the pressure range of 20 to 850 kPa are used for the pressure compensation between inner and outer cell. They work in the range of 0 to 10 volts DC. There are five pressure transducers in the range of 0 to 215 kPa and 0 to 715 kPa, and are connected to the water line.
Figure 3.3: Schematic of the Calibration Chamber (Puppala, 1993)
Table 3.5: Calibration Factors for MQSC (Puppala, 1993)

<table>
<thead>
<tr>
<th>Tip Area (cm²)</th>
<th>Calibration (kg/volts)</th>
<th>Sleeve Area (cm²)</th>
<th>Calibration 1 (sleeve loaded) (kg/volts)</th>
<th>Calibration 2 (tip loaded) (kg/volts)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.27</td>
<td>210</td>
<td>25</td>
<td>204</td>
<td>188</td>
</tr>
</tbody>
</table>

related to piston cell, inner, and outer water lines. Five pressure gauges which work in the range of 0 to 715 kPa are the other units of the control panel. The software for consolidation and testing at different boundary conditions was developed by de Lima (1989), and was used by Puppala (1993). The software has five computer programs, one for consolidation phase (CHAMBKO.EXE) and the four (CHAMBC1.EXE, CHAMBC2.EXE, CHAMBC3.EXE, CHAMBC4.EXE) for the penetration phase. Piston cell, inner and outer cell transducers are connected to the A/D channels. LVDT and tip and friction load cells are also connected to A/D channel. Two D/A channels send the data to two electro-pneumatic transducers in order to apply pressures to the specimen.

3.1.4 LTRC Research Vehicle for Geotechnical Insitu Testing

The Research Vehicle for Geotechnical Insitu Testing (REVEGIT) that is used in this investigation for the field cone penetration data is a 20 tonne all wheel-drive vehicle. Plate 3.2 shows the general view of the LTRC REVEGIT. The vehicle is well equipped for in-situ subsurface soil exploration for civil and geo-environmental engineering purposes. The cone penetration testing system is placed in a van body mounted vehicle. The vehicle also includes hydraulic leveling and CPT operation with a 1 m stroke chucking system. There are three
Plate 3.2: The General View of LTRC REVEGIT Positioned on Top of the Loess Bluff at WES.
jacks for the hydraulic leveling system. Two of them are mounted behind the driver's cab, and one jack at the rear of the vehicle frame. The vehicle penetration thrust system has two double acting hydraulic cylinders. Maximum drive load is 200 kN and pulling load is 260 kN. With the clamping device, rods of 35.6 mm and 55 mm in diameter can be penetrated and extracted.

The Reference Friction Cone Penetrometer with a nominal diameter of 35.7 mm is used for cone penetration. The cross-sectional area of the cone is 10 cm², and the friction sleeve area and the cone apex angle of the cone are 150 cm² and 60 degree respectively. It measures cone and side friction resistance.

REVEGIT'S data acquisition hardware consists of a signal conditioning unit (PCU-M), a Compaq Portable III micro computer, a forty megabyte internal hard disk drive, and the Data Translation DT-2801A analog to digital conversion and digital I/O board. The data acquisition and reduction software are programmed by Borland International and the HALO'88 graphics library by Media Cybernetics. The displacement transducer manufactured by Fugro-McClelland is used for depth measurement purposes.

3.2 Specimen Preparation and Testing Procedures

3.2.1 Triaxial Tests on Artificially Cemented and Uncemented Sand Specimens

The strain controlled consolidated drained triaxial tests were conducted using Monterey No. 0/30 Sand in LSU LoadTrac triaxial system. The Monterey No. 0/30 sand is commercial beach washed sand. Figure 3.4 presents the grain size distribution of this sand. The Monterey No. 0/30 sand has a sub rounded to rounded shape. Its specific gravity (Gₛ) is 2.65. Maximum dry density (γ_{max}) and minimum dry density (γ_{min}) are 16.9 kN/m³ and 14.5 kN/m³, respectively.
Figure 3.4: Grain Size Distribution of Monterey No. 0/30 Sand (Puppala, 1993)
3.2.1.1 CID Tests on Uncemented Monterey No. 0/30 Sand.

a) Specimen Preparation

1. A cylindrical Plexiglass split mold with a clamp, and a stretched membrane inside is placed in the triaxial cell on the bottom cap of the cell. Between the bottom cap and the mold, a porous stone is placed. The amount of dry Monterey No. 0/30 needed to achieve required density is weighed and pluviated in the mold. Pluviation is done with a funnel by keeping the height of fall constant. The height of fall is kept constant to assure the homogeneity. The pluviation procedure with the funnel and the clamped mold is shown in Plate 3.3.

2. Another porous stone and the top cap are placed over the pluviated sand specimen. The membrane is pulled over the top cap and bottom cap, and four O-rings are stretched around the top and bottom caps.

3. Saturated back pressure line is connected to the top cap. After applying a vacuum pressure of 30 to 40 kPa through the drainage valve to make the specimen stand without a mold, the clamp is loosened and the split mold is removed.

4. The height and the diameter of the specimen are measured with a caliper, and average relative density is calculated. This procedure is repeated until the desired density is achieved.

5. After the outer cell and the top plate with the attached triaxial piston are placed, bolted and tightened, a cell pressure of 35 to 70 kPa is applied around the specimen with deaired water. The triaxial piston is lowered and locked such that the piston sits in the gap which is on the top cap of the specimen.

6. The vacuum pump is disconnected, and the backpressure line is opened to make the specimen saturated. The water then flows into the specimen. When the water starts coming out from the drainage valve, the drainage valve is closed. Then, backpressure is applied in steps subsequent to increase in increments of
Plate 3.3: Pluviation Procedure with the Funnel and the Mold
cell pressure. Backpressure has to be kept always less than cell pressure. During triaxial testing, a backpressure of 300 kPa is used to ensure full saturation.

7. It is believed that high amount of backpressure is not necessary for full saturation in sand specimen which has high hydraulic conductivity. Therefore the backpressure used in sand specimens testing program is sufficient to achieve full saturation. After the saturation is achieved, the cell pressure is adjusted in order to achieve required effective stress.

b) Testing Procedure

1. Once the desired effective stress is achieved, the saturated water line of the volume change transducer is connected to the drainage valve, and the backpressure that the specimen is subjected to is applied to the water line connected to the left side of the volume change transducer. The valves are opened, and let the water get balanced in the both sides of the transducer before any load is applied.

2. The triaxial cell is placed onto the platen of the triaxial frame, and it is positioned over the centering lip of the bottom platen.

3. The system is turned on in the order of computer, monitor, signal conditioning unit and load frame. CD/LOADTRAC is typed in order to get the LoadTrac subdirectory, and LT is typed to load the LoadTrac program.

4. In order to enter the Motor-Control menu from the main menu of LoadTrac, Alt key is held and M is Pressed. Now, The loading frame is positioned such that the triaxial piston just touches the loading frame by holding alt key and pressing U to unload or L to load. The triaxial piston is unlocked.

5. For sample information screen, Alt key is held, and E for edit menu and S for sample information screen are pressed. Now, the sample diameter, height, date etc. are entered. For getting back to the main menu Alt key and X are used.
6. In the edit submenu, there are also time table and test information displays. The times at which the data reading is taken and saved are inputted on the time table menu. The strain rate, which is selected as 1% of the height of the specimen per minute, test duration are inputted on test information display.

7. Correct calibration factors are checked by holding Alt key and pressing C for the calibrate submenu.

8. After getting back to the manual menu, final adjustment is made on physical position of the displacement transducer until the displacement reading is between 10 and 100.

9. Using Alt key and R, the run submenu is displayed. Now, everything is ready for the test. Alt key is held and S is pressed to start the test. The valve of the volume change transducer is opened. The software questions for a file name. A file name is given to store the test results in. The test begins automatically.

10. During the test, the view of the current status of the test can be displayed by using Alt key and V.

11. After the change in volume becomes zero, the test is stopped by pressing Alt and A for abort in run submenu.

12. The software questions whether the sample is unloaded. Pressing yes makes the platen to move back down to the bottom position.

13. The cell pressure is released, and the water is drained. From three different position (the top, the middle, and the bottom), soil samples are taken for moisture content determination.

14. The triaxial cell and platen are cleaned and dried for the next test.

15. After using the convert function in the LoadTrac program, the test data is copied to a floppy disk. The results are transferred to another PC for reduction and plotting. Lotus 123, and Grapher were used for reduction and plotting.
3.2.1.2 CID Tests on 1 and 2% Cemented Monterey No. 0/30 Sand Specimens.

a) Specimen Preparation
1. 30 Plexiglass specimen molds and acrylic bottom plates are used. The split molds are 7.24 cm (2.85 in.) diameter and 16.51 cm (6.5 in.) height. They are placed on the bottom plates after the membranes are stretched inside. The bottom plate has four small holes which are used for saturation purposes. Before the pluviation, a filter paper is placed between the mold and the bottom plate.
2. A cement sand mixture is prepared in the container. 1000 gram of dry Monterey No. 0/30 Sand is placed in the container and mixed with 4 gms of water (0.4 percent of the dry weight of the sand). The cement is added and mixed gradually in order to make perfect distribution of the cement.
3. Using the same pluviation procedure shown in Plate 3.3, desired relative density is achieved.
4. Four of the specimens in the Plexiglass molds with the bottom plates are placed on a bed of sand in a container. This procedure is presented in Plate 3.4.
5. Water is applied slowly to the container. When the water level rises gradually, it flows through the bottom plate of the molds. It takes 4 to 5 hours for the water level to rise over the specimens.
6. The submerged cemented sand specimens are transferred to the humidity room for curing. Plate 3.5 shows a photography of the humidity room and the specimens.

A curing period of 14 days is selected for some specimens in order to compare with the results of Rad in evaluating the performance of the LoadTrac triaxial system. The rest of the specimens were cured for 7 days since one of the objectives of this study was to provide strength parameters for Puppala (1993).
Plate 3.4: Saturation Procedure.
Plate 3.5 : Curing Procedure.
A curing period of 7 days was sufficient to form cemented bonds between the sand particles (Puppala 1993). That is why the 7 day curing period was selected in this study. The scanning electron micrograph of the cemented sands are shown in Plate 3.6.

b) Testing Procedure

1. After the curing period is reached, the specimen is transferred from the humidity room to the triaxial room with a special care of not disturbing it.
2. The bottom plate is removed and four O-rings are placed on the Plexiglass mold.
3. The specimen with the mold is placed on the bottom cap of the triaxial cell after a porous stone is placed.
4. The membrane is stretched on the bottom cap, and two O-rings are stretched around the bottom cap on the membrane.
5. A porous stone, top cap, and two O-rings are placed on the top of the specimen as well.
6. The clamp is loosened, and the Plexiglass mold is removed after applying a vacuum pressure of 30 kPa to 40 kPa.
7. The rest of the test procedure is similar to that of uncemented specimens.

3.2.2 CID Tests on Undisturbed Naturally Cemented Specimens from Loess Deposit.

The naturally cemented soil samples were obtained from a bluff of loess deposit in the US Army Corps of Engineers Waterways Experiment Station (USAEWES) in Vicksburg, Mississippi. The water table is said to be located 10 m below the bottom of the slope. The cone penetration study was conducted along this bluff. The laboratory triaxial study was conducted on specimens taken from this bluff. Undisturbed specimens were obtained from the elevation of 0-0.5
Plate 3.6: Scanning Electron Micrograph of Cemented Sand (1 and 2 %) (Puppala, 1993)
m which is named as WES 2 and the elevation of 7.0-7.50 m which is named as WES 1. Plate 3.7 and Plate 3.8 show the locations of the specimens, WES 1 and WES 2, in the slope, respectively. A photograph of the slope is shown in Plate 3.1. The method of retrieving of undisturbed samples is discussed in the specimen preparation section.

a) Specimen Preparation

1. Approximately 0.03 m$^3$ to 0.04 m$^3$ block samples of the soil were hand carved from the bluff. The blocks are taken at 7 to 7.50 m depth. These specimens are called WES 1. Also, blocks are taken at 0-0.50 m depth in the slope and these are called WES 2. Plate 3.7 shows the slope and WES 1 blocks. Plate 3.8 presents the top of the slope where WES 2 blocks are obtained. The plan of the slope is given in Figure 3.5.

2. First the left and the right sides of the blocks (WES 1) are carved using cane knives. After carving the bottom of the blocks, aluminum plates are placed into the slope under the bottom of the blocks. Finally the top of the blocks are carved, and the blocks are retrieved from the slope. Similar technique is used for the WES 2 blocks that are taken from the top of the slope. First 0.20 m of soil removed from the top, and soil around the block is dug without disturbing the block. Next, using hand saw, the blocks are removed.

3. Using a hand saw, these blocks are cut into smaller rectangular blocks (25 to 30 cm x 15 cm). Extreme care has to be taken for handling the samples. A small shock breaks the samples into pieces.

4. Three field trips were taken to the bluff to retrieve undisturbed samples. From the first trip 10 rectangular samples (WES 1) were obtained but during handling, shipping and trimming 3 of them were lost. In the second trip, two 0.04 m$^3$ block samples (WES 1) were carved from the slope shown in Plate 3.7. Fifteen
Plate 3.7: The Slope and Blocks Located at 7-7.5 m Depth of the Slope (WES1)
Plate 3.8: A View of the Location of the Specimen Blocks Taken from 0-0.5 m Depth on the Top of the Bluff (WES 2)
Figure 3.5: The Sketch of the Slope and the Location of the Block Specimens
rectangular blocks (approximately 25 cm X 15 cm) were obtained out of two cubic blocks. From the third trip to the slope, ten rectangular blocks (WES 2) are obtained.

5. After the rectangular blocks are prepared, they are wrapped carefully in plastic zip-lock bags. They are placed in plastic thermos in transporting to the laboratory. In the laboratory the thermos is kept in the humidity room to retain the natural moisture contents.

6. The final shape of the specimen is given by a trimming procedure. The specimen is placed in a trimmer and trimmed very carefully with a wire until the final cylindrical shape is given. The trimmer and the wire that are used in this study are shown in Plate 3.9. Snail shells exist in the soil. These snails are cemented together with the soil. During trimming, these snails make the trimming process difficult. When the wire hits a cemented snail shell, a piece of soil comes off. Then, the circular shape of the specimen is not retained. Even though the specimens were trimmed very patiently and smoothly, several specimens were lost during the trimming process.

7. The trimmed soil is placed in a container for moisture content study.

b) Testing Procedure

The consolidated drained testing on undisturbed, naturally cemented specimens is similar to that of artificially prepared specimens except the saturation process and the strain rate. Four hydraulic conductivity tests are conducted on both undisturbed cemented loess specimens and artificially cemented sand specimens on triaxial system. The silty loess specimens have lower hydraulic conductivity values \((7 \times 10^{-5} \text{ cm/s})\) than the artificially cemented sand specimens \((2.5 \times 10^{-3} \text{ cm/s})\) therefore lower strain rates and longer saturation periods were used. The following procedure was used for saturation and in selection of strain rates.
Plate 3.9: The Trimmer and the Trimming Wire
I- Selection of strain rate: Since the drained strength parameters are investigated, drained conditions prevail during the testing. Strain rate used should be sufficient to dissipate all the excess pore pressure developed during triaxial testing. Head (1986) gives a methodology to select the appropriate strain rate for drained triaxial test. The value of the time required to failure in a drained test, \( t_f \), is obtained by conducting a consolidation test and using the methodology given by Head (1986). The result of the consolidation tests, calculation of the strain rate, and the methodology proposed by Bishop and Henkel (1965) are presented in Appendix C. The strain rate of 0.2 percent of the height of the specimen/min (approximately 0.30 mm/min) is used in drained triaxial testing on undisturbed silt specimens. One test took approximately 1.5 hours to complete with a final strain of 20% at which no more changes in volume were observed.

II-Saturation procedure:

1. First, a small amount of cell pressure (50 kPa) is applied, and the backpressure valve is opened to allow flow of water in the specimen under a 20 kPa backpressure. In the meantime, the drainage valve is kept open until the water starts flowing from the drainage valve.

2. The cell and backpressure are raised to the next increment. Saturation is checked by calculating the Skempton's B parameter. The next increment is calculated according to the value of the current Skempton's B parameters. The calculation of the Skempton's B parameter and the increments are explained in Appendix D.

3. When a B value of 0.90 to 0.95 is achieved, saturation process is stopped and full saturation is assumed to be achieved (Bishop and Henkel, 1965).

4. Full saturation of the specimen was achieved in 2 to 3 days.
3.2.3 Unconfined Compression Tests on Undisturbed Naturally Cemented Specimens from the Loess Deposit.

Three unconfined compression tests were conducted on undisturbed naturally cemented silt specimens taken from the slope in WES, Vicksburg, Mississippi. Specimen preparation procedure (retrieving undisturbed samples etc.) for unconfined compression tests is similar to the procedures used in preparing undisturbed triaxial specimens. The size of the specimens are 2.7" in diameter and 5" in height.

3.2.4 Cone Penetration Tests in The Calibration Chamber

Several cone penetration tests were conducted in the calibration chamber to evaluate the repeatability and accuracy of some of Puppala's test results. For specimen preparation and testing procedure, Puppala's (1993) chamber study program was reviewed. Same equipment was used, identical specimen preparation and testing procedures were followed.

a) Specimen Preparation

1. Cement-sand mix is prepared. 13,500 gm of dry sand is weighed and place in the mixture. 1 or 2 percent (135 gm or 270 gm) of portland cement is added after spraying 54 gm of water (0.4 percent by the dry weight) on the sands. The mixing procedure is continued until an even distribution of the cement and the coating of the sand particles by cement are achieved.

2. The mixing procedure is repeated (approximately 13-15 times) until the top chamber of the pluviation setup is filled with cement-sand mix.

3. The specimen chamber is prepared. A membrane with O-rings is placed around the bottom plate and vacuum is applied in order to stretch the membrane.

4. The specimen chamber is rolled underneath the pluviation setup table, and the
shutter is rotated to make the holes align so that the sand starts pluviating. Lower relative densities (45 and 57 percent) were achieved using a shutter with a 15% porosity and a sieve with opening size of 12.5 mm. Higher relative densities (74, 81, and 88) were achieved using a shutter with porosity of 6.5% and sieves with opening sizes of 6.3 mm and 9.5 mm.

5. When the pluviation is finished, the specimen chamber is rolled out, and necessary measurements are taken for calculation of relative density of the specimen.

6. The water line is connected to the bottom of the specimen chamber and the water is applied slowly from the saturation tank. It takes 12 to 15 hours to make the specimen saturated.

7. The specimen chamber is moved to the humidity room for a 7 day curing.

The specimen preparation procedure for uncemented specimens is similar to that of cemented specimen. Dry Monterey No.0/30 Sand is used and no cement or water is added. After the steps of pluviation and relative density calculation, the specimen chamber is directly moved to the calibration chamber for testing. Specimens are tested in dry conditions.

b) Testing Procedure

Similar testing procedure is followed for cemented and uncemented specimens. The only difference is resaturation of cemented specimens subsequent to their placements in the chamber under a vacuum.

1. The specimen chamber is lifted on the calibration chamber.

2. A suction is applied with vacuum pump from the top plate after the top plate and the O-rings around the plate are placed.

3. The split molds of the specimen chamber are carefully removed, and the inner and outer chambers are lifted and placed over the specimen. During this process,
the vacuum that induces a confinement in the specimen has to remain the same. A loss of suction pressure causes the collapse of the specimen.

4. After the placement of inner and outer chambers, the outer top plate is placed and connected to the top plate of the specimen with twelve bolts. Finally, the outer top plate is connected to the piston cell with twelve rods, and the rods are tightened under a torque of 65 kN-m.

5. The water lines are connected to the outer top plate to fill the inner and outer cells with deaired water.

6. After the inner and outer cells are filled, the specimen is consolidated under \( K_0 \) conditions using the software program, CHAMBK0 (de Lima, 1990). File name, the value of consolidation stress (four specimens were tested under the consolidation pressure of 300 kPa, and one specimen under 200 kPa.), details about the specimen, the ranges of transducers are entered and consolidation under \( K_0 \) condition is automatically achieved.

7. After the consolidation phase is completed, the hydraulic jack is lifted above the outer top plate, and connected to the place such that the cone can be driven at the center of the specimen. The miniature cone is placed on the jack and positioned such that it touches the specimen.

8. The software program, CHAMBC3 is used for cone penetration. File name, testing information, transducer ranges are entered. Then, the graphics screen that shows the graphics of the tip resistance, friction resistance, vertical stress, inner and outer horizontal stress versus depth appears.

9. The cone is pushed into the specimen at a rate of 2 cm/sec.

10. After the cone penetration is completed, the setup is disassembled in the order of cone, hydraulic jack, outer top plate, outer and inner chambers, and the sands. The tested cemented sands are thrown away, but tested uncemented sands are kept to prepare another specimen.
CHAPTER 4. TEST RESULTS ON MONTEREY NO. 0/30 SAND

4.1 CID Tests on Artificially Cemented and Uncemented Monterey No. 0/30 Sand.

The triaxial testing program consisting of 40 static strain-controlled drained triaxial tests was carried out on artificially cemented and uncemented sands. Cementation levels of 0, 1, and 2 percent were used. Three ranges of relative density values, 55 to 60, 65 to 70, and above 80 percent were used. The tests were conducted at confining pressures of 100, 200, and 300 kPa. The cemented specimens were cured for 7 days. Initially, three triaxial tests were conducted on 1 percent cemented specimens prepared at approximately 88 percent relative density and cured for 14 days. These tests were carried out to compare with the results of Rad (1984) in an attempt to evaluate the performance of the newly purchased and calibrate the LoadTrac triaxial equipment. The stress-strain and volume change curves are given in Figure 4.1. Figures 4.2 and 4.3 compare these results with those of Rad (1984). The repeatability of the tests is demonstrated. The author believes that the slight differences noted are due to 5 to 8 percent difference in relative densities.

A semi-drained condition would exist during a quasi-static cone penetration test due to high hydraulic conductivity of the sand (Rad, 1984). Puppala (1993) cone penetration testing program demonstrated that drained conditions prevailed due to high hydraulic conductivity of Monterey No.0/30 sand. Therefore, the drained strength parameters were investigated.

Figure 4.4 presents the stress-strain curves for uncemented Monterey No.0/30 sand specimens at a relative density of 55 %. The stress-strain curves for uncemented and artificially cemented specimens (1 to 2 %) prepared to relative
Figure 4.1: The Stress-Strain and Volume Change Behavior of Artificially Cemented Monterey No.0/30 Sand with 1% Cement at a Relative Density of 88%.
Figure 4.2: A Comparison between the Stress-Deformation Behavior for Artificially 1% Cemented Monterey No. 0/30 Sand Specimens Cured for 14 Days and Rad (1984) Results
Figure 4.3: A Comparison of Effective Stress Path and Failure Envelope.

- ○○○○○ $D_r = 80\%$, RAD (1984)
  - $\phi' = 38.0$, $c' = 14$ kPa

- ••••• $D_r = 88\%$, THIS STUDY
  - $\phi' = 38.5$, $c' = 18$ kPa
Figure 4.4: Stress-Strain Curves for Uncemented Monterey No.0/30 Sand Specimens at a Relative Density of 55 %
densities ranging from 55 to 89 percent are given in Appendix A in Figures A.1 to A.8. Due to difficulty in achieving exactly the same relative density for each specimen, the average relative densities for three confining pressures of 100, 200, and 300 kPa are given in these plots. Table 4.1, 4.2 and 4.3 present detailed information for individual specimens.

Table 4.1: CID Test Results on Uncemented Monterey No. 0/30 Sand.

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<tr>
<th>Sample No.</th>
<th>Cement Content (%)</th>
<th>$\sigma_3$ (kPa)</th>
<th>$D_r$ (%)</th>
<th>Axial Strain (%)</th>
<th>$\sigma_1-\sigma_3$ (kPa)</th>
<th>Volume Change (%)</th>
<th>Peak Strength</th>
<th>Residual Strength</th>
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Table 4.2: CID Test Results on Cemented (1 %) Monterey No. 0/30 Sand.

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<th>Relative Density Dr (%)</th>
<th>Axial Strain (%</th>
<th>σ1−σ3 (kPa)</th>
<th>Volume Change (%)</th>
<th>Axial Strain (%)</th>
<th>σ1−σ3 (kPa)</th>
<th>Volume Change (%)</th>
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<td>1086</td>
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<td>11.1</td>
<td>960</td>
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Table 4.3: CID Test Results on Cemented (2%) Monterey No. 0/30 Sand.

<table>
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<tr>
<th>Sample No.</th>
<th>Cement Content (%)</th>
<th>$\sigma_3$ (kPa)</th>
<th>Relative Density $D_r$ (%)</th>
<th>Axial Strain (%)</th>
<th>$\sigma_1 - \sigma_3$ (kPa)</th>
<th>Volume Change (%)</th>
<th>Axial Strain (%)</th>
<th>$\sigma_1 - \sigma_3$ (kPa)</th>
<th>Volume Change (%)</th>
</tr>
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<td>4.4</td>
<td>440</td>
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<td>1146</td>
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<td>87</td>
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<td>853</td>
<td>1.9</td>
<td>10.4</td>
<td>652</td>
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<td>10.8</td>
<td>1002</td>
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</table>
Volume changes were also measured. These are plotted below the stress-strain curves. These figures show that increase in the confining pressure increases the strength of the specimens, and also increase in the cementation level increases the brittleness and peak strength of the specimens. Another observation is that the sands show dilation during shear. An increase in confining pressure causes a decrease the dilation.

Figure 4.5 shows a plot of the peak strength results in the form of $p'\text{-}q$ diagram for the uncemented and artificially cemented specimens with different relative densities and cement content. The rest of the peak strength envelopes in the form of $p'\text{-}q$ diagram is given in Figure A.9 and Figure A.10 in Appendix A. These figures reveal that increase in the cement content increases the cohesion intercept. The cohesion intercept is 0 for uncemented specimen at the relative density of 89% whereas the cohesion intercepts become 16 and 31 kPa for 1 and 2% cemented specimens respectively at same relative density. Effect of cementation on cohesion intercept is presented by plotting cohesion intercept against cementation level in Figure 4.6. The cohesion intercept also increases when the relative density increases for the same cement content. For example, the cohesion intercept increases from 6 to 16 kPa when the relative density increases from 57 to 88% for one percent cemented specimens. Cohesion intercept against relative density is also plotted and shown in Figure 4.7. Rad (1984) reported an explanation for above behavior that "this is likely due to the fact that at higher densities, more contact exists between the sand particles and thus more opportunity exists for cementation". Also, increase in relative density increases the friction angle. For example, the friction angle increases from 35.8 to 40.5 degrees, when the relative density increases from 55 to 89% for uncemented specimens. Effect of relative density on friction angle is presented by plotting relative density against friction angle in Figure 4.8.
Figure 4.5: Peak Strength Envelopes for Uncemented Monterey No. 0/30 Sand Specimens at Relative Densities of 55, 69, and 89 %.

- $D_r = 55\%$, $\phi' = 35.8$, $c' = 0$
- $D_r = 69\%$, $\phi' = 38.9$, $c' = 0$
- $D_r = 89\%$, $\phi' = 40.5$, $c' = 0$

C.C. = 0 %
Figure 4.6: Effect of Cementation on Peak Cohesion Intercept of Cemented and Uncemented Monterey No.0/30 Sands.
Figure 4.7: Effect of Relative Density on Cohesion Intercept of Cemented Monterey No. 0/30 Sands.
Figure 4.8: Effect of Relative Density on Friction Angle of Cemented and Uncemented Monterey No. 0/30 Sands.
Residual strength envelopes on p'-q diagrams are presented in Figure A.11, A.12, and A.13 in Appendix A. The residual friction angles and the cohesion intercepts are less than peak values in all cases. Table 4.4 presents the drained peak strength parameters obtained from this study (1993) and Rad's (1984) study for comparison. In both cases, there is no cohesion intercept for uncemented specimens and friction angles increase with an increase in relative density.

Table 4.4: Drained Peak Strength Parameters.

<table>
<thead>
<tr>
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<th>C.P. (Days)</th>
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<th></th>
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<tbody>
<tr>
<td></td>
<td>C.P. (Days)</td>
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<td></td>
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</tr>
<tr>
<td>THIS</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C.C. (%)</td>
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<td>2</td>
</tr>
<tr>
<td>STUDY (1993)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D_r (%)</td>
<td></td>
<td>55</td>
<td>69</td>
<td>89</td>
</tr>
<tr>
<td>θ' (Degrees)</td>
<td></td>
<td>35.8</td>
<td>38.9</td>
<td>40.5</td>
</tr>
<tr>
<td>c' (kPa)</td>
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<tr>
<td>RAD (1984)</td>
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<td>14</td>
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<td></td>
</tr>
<tr>
<td>C.C. (%)</td>
<td></td>
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<td>2</td>
</tr>
<tr>
<td>D_r (%)</td>
<td></td>
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<tr>
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</tr>
<tr>
<td>c' (kPa)</td>
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</table>

C.P.: Curing Period,  C.C.: Cement Content,  D_r: Relative Density,
θ': Angle of Friction,  c': Cohesion Intercept
An important observation is that 7 and 14 days curing would not make significant difference on strength parameters. In this study and Rad (1984) study the lowest value of the cohesion intercept is 6-7 kPa and highest value is 30-31 kPa for 1 and 2% cemented specimens. The lowest value of the friction angle is 33-36 and the highest value is 39-40 for cemented and uncemented specimens. Table 4.5 presents the drained residual strength parameters obtained from this study (1993) and Rad's (1984) study. Residual strength parameters are identical for all practical purposes.

Table 4.5: Drained Residual Strength Parameters.

<table>
<thead>
<tr>
<th></th>
<th>C.P. (Days)</th>
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<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td>THIS</td>
<td>C.C. (%)</td>
<td>0</td>
<td>1</td>
<td>2</td>
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</tr>
<tr>
<td>STUDY</td>
<td>D_r (%)</td>
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<td>69</td>
<td>89</td>
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<tr>
<td>(1993)</td>
<td>(\phi)' (Degrees)</td>
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<td>34.5</td>
<td>36.0</td>
<td>33.8</td>
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<tr>
<td></td>
<td>c' (kPa)</td>
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<tr>
<td>RAD</td>
<td>C.P. (Days)</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1984)</td>
<td>C.C. (%)</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>D_r (%)</td>
<td>31</td>
<td>45</td>
<td>77</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>(\phi)' (Degrees)</td>
<td>33</td>
<td>34</td>
<td>35</td>
<td>33</td>
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<tr>
<td></td>
<td>c' (kPa)</td>
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</table>

C.P.: Curing Period,  C.C.: Cement Content,  D_r: Relative Density,  
\(\phi\)': Angle of Friction,  c': Cohesion Intercept
4.2 Assessment of Repeatability of Cone Penetration Tests in The Calibration Chamber.

Five cone penetration tests using Monterey No. 0/30 sand were conducted in the calibration chamber. The sample preparation and testing procedures were identical to those used by Puppala (1993). It was not possible to obtain the exact relative densities achieved by Puppala (1993). Comparative plots of tip resistance, sleeve resistance, and friction ratio versus depth are presented in Figures 4.9 through 4.13. The result of the test conducted on uncemented specimen at a relative density of 74% and at confining pressure of 300 kPa is different than the result obtained in the previous test. Tip resistance in the previous test is lower within the first 20 cm. The difference is probably due to a lack of confinement or nonhomogeneity of the specimen in previous study. The comparison of the second test conducted on uncemented specimen at a relative density of 57% and at a confining pressure of 300 kPa suggests a similar conclusion. However, the value of the tip resistances are close to each other after 20 cm. Third test conducted on a 1% cemented specimen at a relative density of 45% and at confining pressure of 300 kPa appear to be repeatable even though there is a difference between the value of tip resistances. This difference is most probably due to the difference of 8% in relative densities. Previous test was conducted at a relative density of 53%. The one conducted at a relative density of 88% and cementation level of 1% at confining pressure of 300 kPa is different from Puppala (1993) result. The result of the test conducted in this study gives lower tip resistance than that of Puppala (1993). The author believes that the difference is mainly due to the procedural change in conducting this test. The high relative density and confining pressure forced the author to stop the test very often and withdraw the cone in order not to damage the cone penetrometer. That probably resulted in a disturbance on the specimen. The result of the last
Figure 4.9: Cone Penetration Test Results Obtained by Puppala (1993) and Present ($\sigma_v' = 300$ kPa, C.C. = 0, $K = 0.38$)
Figure 4.10: Cone Penetration Test Results Obtained by Puppala (1993) and Present ($\sigma_v' = 300$ kPa, C.C. = 0, K = 0.46)
Figure 4.11: Cone Penetration Test Results Obtained by Puppala (1993) and Present ($\sigma'_v = 300$ kPa, C.C. = 1%, K = 0.43).
Figure 4.12: Cone Penetration Test Results Obtained by Puppala (1993) and Present ($\sigma_v' = 300$ kPa, C.C. = 1%, $K = 0.35$)
Figure 4.13: Cone Penetration Test Results Obtained by Puppala (1993) and Present ($\sigma_v' = 200$ kPa, C.C. = 2 %, K = 0.43)
test conducted on 2 % cemented specimen at a relative density of 81 % and at confining pressure of 200 kPa is similar to the result obtained by Puppala if the tip resistances are compared. It is assumed that the difference of 10-15 % on tip and sleeve resistances between present and previous tests are acceptable due to difference in relative densities and testing procedure.
CHAPTER 5. TEST RESULTS ON NATURALLY CEMENTED LOESS DEPOSIT

5.1 Compositional Analysis

Hydrometer analyses were conducted on soil samples obtained from different levels of the slope (WES 1 and WES 2) shown in Plates 3.7 and 3.8. Grain size distribution curves for the two samples are shown in Figure 5.1. The soil from the loess bluff has a uniform grain size. The specific gravity of the soil, $G_s$, is 2.70 and the initial degree of saturation is 50%. The soil specimens taken from 7-7.5 m (WES 1) and from 0-0.5 m (WES 2) both consist of 75-80% silt and 20-25% clay.

Scanning electron micrographs are shown in Plate 5.1 for the soil (WES 1) located at 7-7.5 m. Hypothetically, the micrograph shows the clay bonds that possibly act as a binder and cause cementation in the deposit. However, these micrographs do not have sufficient resolution to observe and identify the clay bonds and cementation.

5.2 CID Tests

Seven static strain-controlled drained triaxial tests were conducted on undisturbed naturally cemented specimens cut from the WES 1 sample. Confining pressures of 25, 50, 100, 150, 200, 250, and 300 kPa were used. The moisture contents for the specimens were approximately 15%. Specimens were fully saturated before shearing. The saturation procedure was discussed in Chapter 3. Typical stress-strain and volume change curves for seven of the specimens are given in Figure 5.2. Peak strength was observed to increase with an increase in confining pressure. Contraction was observed for all the tests. The
Figure 5.1: Grain Size Distribution of the Soil - Hydrometer Analyses (WES 1 and WES 2)
Plate 5.1: Scanning Electron Micrograph of the Soil (WES 1).
Figure 5.2: Stress-Strain Curves for Naturally Cemented Specimens from the Loess Deposit (WES 1)
peak strength values for the loess specimens are presented in Figure 5.3 plotted in the form of a $p'$-$q$ diagram. The best-fit line gives a friction angle of 25 degrees and a cohesion intercept of 7 kPa. The residual strength values were also determined and are presented in Figure 5.4. The residual cohesion intercept and friction angle are also 7 kPa and 25 degree respectively. Five static strain-controlled drained triaxial tests were also conducted on undisturbed naturally cemented specimens (WES 2) at confining pressures of 100, 200, and 300 kPa. These specimens were obtained from the same bluff but at 0-0.50 m depth. Two CID tests were conducted at confining pressures of 100, and 200 kPa in order to evaluate the repeatability and accuracy of the tests. The tests results demonstrated excellent repeatability. Typical stress-strain and volume change curves for five of the specimens are given in Figure 5.5. The peak strength increases with an increase in the confining pressure. Contraction was observed in the tests conducted at confining pressures of 200 and 300 kPa. Dilation was also observed in the test conducted at lower confining pressure. This test was conducted at a confining pressure of 100 kPa. Dilational behavior of the soil at 100 kPa indicates that the deposit may be overconsolidated. The peak strength values for the silt specimens are presented in Figure 5.6 plotted in the form of a $p'$-$q$ diagram. The best-fit line gives a friction angle of 21 degrees and a cohesion intercept of 50 kPa. Due to time constraint and difficulty of sampling and transporting undisturbed specimens, CID tests were not conducted at lower confining pressures. The residual strength values are also determined and presented in Figure 5.7. The residual cohesion intercept and friction angle are 35 kPa and 22 degrees respectively.
Figure 5.3: Peak Strength Envelope for Naturally Cemented Specimens from the Loess Deposit (WES 1)
Figure 5.4: Residual Strength Envelopes for Naturally Cemented Specimens from the Loess Deposit (WES 1)
Figure 5.5: Stress-Strain Curves for Naturally Cemented Specimens from the Loess Deposit (WES 2)
Figure 5.6: Peak Strength Envelopes for Naturally Cemented Specimens from the Loess Deposit (WES 2)
Figure 5.7: Residual Strength Envelopes for Naturally Cemented Specimens from the Loess Deposit (WES 2)
5.3 Unconfined Compression Tests

The unconfined compression tests were conducted on undisturbed naturally cemented samples obtained from the slope at 7-7.5 m depth (WES 1). The results of the unconfined compression tests are presented in Figure 5.8. In order to assess the repeatability of the test result, two tests were conducted. Both test results showed that the soil has an unconfined compressive strength of 125 kPa. An unconfined compression test was also conducted on undisturbed cemented specimens taken from the slope at 0-0.50 m (WES 2). Figure 5.9 presents the result of this test. An unconfined compressive strength of 125 kPa was also obtained from these tests. This deposit, as per Rad’s (1984) classification, can be termed as a weakly cemented.

5.4 Cone Penetration Tests

Cone penetration soundings of 10 to 20 m deep were performed in the cemented loess bluff shown in Plate 3.7. The ground water table is located 20 to 25 m below the top ground level. The research vehicle was taken to the top of the slope (see Plate 3.2) and located 10 m away from the edge of the slope for the first cone penetration test. Four cone penetration tests were performed within a distance of 1 to 1.5 m and using three different penetration rates: 0.25 cm/sec, 1 cm/sec and 2 cm/sec. The results of the test conducted at a penetration speed of 2 cm/sec are shown in Figure 5.10. The rest of the results are presented in Figures B.1, B.2 and B.3 in Appendix B. The reference friction cone penetrometer with a 35.7 mm nominal diameter, a friction sleeve area of 150 cm² and a cone apex angle of 60 degrees was used. The purpose of conducting the test at low penetration rate was to determine whether there will be any differences due to changes in resistance. The test results showed that change in the penetration rate
Figure 5.8: The Result of Unconfined Compression Tests (WES 1)

Unconfined Compression Tests
Naturally Cemented Specimens
(WES 1)

Unconfined Compression Tests
Naturally Cemented Specimens
(WES 1)

\[ q_u = 125 \text{ kPa} \]
\[ w = 15 \% \]
\[ \gamma = 17 \text{ kN/m}^3 \]
Figure 5.9: The Result of Unconfined Compression Test (WES 2)

Unconfined Compression Test
Naturally Cemented Specimen (WES 2)

$\sigma_u = 125$ kPa
$w = 24\%$
$\gamma = 20$ kN/m$^3$
Figure 5.10: The Result of In-Situ Cone Penetration Test Conducted on Naturally Cemented Loess Deposit
from 0.25 to 2 cm/sec, did not affect the tip and sleeve resistances. It is assumed that drained conditions prevail during penetration in such deposits.

The cone penetration test results presented in Figure 5.10 show that tip and sleeve resistances are higher within the first 7 m. Therefore, two different soil layers are considered in this deposit. In the first layer, 0-7 m, the tip resistance shows approximately a constant value of 50 kg/cm². This indicates that the increase in confinement of about 140 kPa within the first 7 m does not affect the generated tip resistance. In the second layer, 7-15 m, there is a slight effect of confinement and the average tip resistance is 15-20 kg/cm². The difference in static drained behavior of the soil between the two layers was also demonstrated in triaxial laboratory study. The cohesion intercept is higher in the first layer than it is in the second layer. The friction angle is lower in the first layer than it is in the second layer. The fact that tip resistance does not change within the first layer, 0-7 m, suggests a low friction angle and a high value of cohesion intercept. In the second layer the tip resistance increases slightly with the increase in depth. This shows that the effect of confinement is greater in this layer indicating a higher drained friction angle.

5.5 Analysis of Test Results

Predicted tip resistances were evaluated using two bearing capacity theories: Durgunoglu and Mitchell (1973) and Janbu and Senneset (1974). The peak shear strength parameters ($c'$, $\phi'$) of the naturally cemented loess deposit at 7-7.5 m (WES 1) and at 0-0.5 m (WES 2) were used to predict tip resistance. Residual strength values of the naturally cemented loess deposit were used in calculating the predicted friction resistance. Figure 5.11 shows the comparison between predicted tip resistance obtained using the Durgunoglu and Mitchell (1973)
Figure 5.11: Comparison Between Predicted and Measured Tip Resistances for Cemented Loess Deposit (Using D & M Theory)
theory and measured tip resistances in the field. Equation 2.1 was used for
calculation of tip resistance, and $N_c$ and $N_{yq}$ are taken from the charts proposed
by Durgunoglu and Mitchell (1973) as shown in Figure 2.3. Figure 5.12 shows
the comparison between predicted tip resistance obtained by using Janbu and
Senneset (1974) theory and measured tip resistance obtained by conducting cone
penetration sounding in the field. Equation 2.2 was used for calculation of tip
resistance, and $N_q$ was taken from the chart proposed by Janbu and Senneset
(1974). The values of plastification angle were estimated using contraction angles
for WES 1 soil and dilation angles for WES 2 obtained from the test results and
Puppala's (1993) dilation angle versus plastification angle chart. It is assumed
that this chart is identical on the contraction side and the line ($\beta = 1.43 \nu - 3.88$)
proposed by Puppala (1993) is valid for both contraction and dilation. The
numerical values used in prediction of tip and sleeve resistances are tabulated in
Table 5.1. The results show that the predicted tip resistance estimated using both
theories show varying degrees of agreement with the measured tip resistance.
Predicted tip resistance is lower than measured tip resistance within the first
layer, 0-7 m. However, the predicted tip resistance correlates well with measured
tip resistance in the second layer 7-15 m.

Sleeve resistances were also predicted. The chart of dilation angle versus
$K/K_0$ proposed by Puppala (1993) was used for estimation of K values for the
soil. Predicted and measured sleeve resistances are shown in Figure 5.13. The
predicted sleeve resistance values were lower than measured sleeve resistance
values for the first layer. However, the predicted sleeve resistance correlates
quite well with measured sleeve resistance in the second layer.

The strength parameters of the soil in the first layer were backcalculated
using the Janbu and Senneset (1974) theory and measured tip resistance. A
cohesion intercept of 150 kPa and friction angle of 20 degrees were obtained.
Figure 5.14 shows a new comparison between predicted tip resistance and measured tip resistance. The strength parameters of the soil in the first layer were also backcalculated by using Durgunoglu and Mitchell (1973) theory and measured tip resistance. A cohesion intercept of 150 kPa and the friction angle of 21 degrees were obtained.

Table 5.1: The Numerical Values Used in Prediction of Tip and Sleeve Resistances.

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<th>Depth (m)</th>
<th>$\sigma_v$ kg/cm$^2$</th>
<th>$\nu$ dilation angle</th>
<th>$\beta$ plastif. angle</th>
<th>Nq</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ degrees</th>
<th>$q_c$ kg/cm$^2$ (D&amp;M)</th>
<th>$q_c$ kg/cm$^2$ (J&amp;S)</th>
<th>$f_s$ kg/cm$^2$</th>
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<td>15.5</td>
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Figure 5.12: Comparison Between Predicted and Measured Tip Resistances for Cemented Loess Deposit (Using J & S Theory)
Figure 5.13: Comparison Between Predicted and Measured Sleeve Resistances for Cemented Loess Deposit
Figure 5.14: Comparison Between Predicted and Measured Tip Resistances for Cemented Loess Deposit (Cohesion Intercept is 150 kPa and Friction Angle is 20 Degrees)
Figure 5.15 shows a comparison between predicted and measured tip resistances. Backcalculation using sleeve resistance values gives 75 kPa of cohesion intercept and 15 degrees of friction angle in the first layer (Figure 5.16).

5. 6 Hypothesis on Analysis of Test Results

Backcalculations show that the value of the cohesion intercept would be more than the one obtained from triaxial study to achieve better correlations of penetration results in the first layer. The cohesion intercept of 150 kPa gives excellent correlations between predicted and measured tip resistances. The following hypotheses are given to explain the cause of low cohesion intercept obtained from the triaxial study:

1. The strength parameters were obtained by conducting CID tests at higher confining pressures. That resulted in missing the overconsolidation part of the failure envelope. This portion can be determined by conducting CID tests at low confining pressures such as 10, 20, and 60 kPa. The value of cohesion intercept is expected to be much higher than the one used in prediction. Lower friction angles are also expected.

2. The measured tip resistance is affected by partial saturation that induces an extra confinement. Therefore, the cohesion values obtained from CID tests conducted on fully saturated specimens would give lower values of predicted tip resistance.

3. Softening occurs in the cementation bonds during the saturation process. The cementation is partially due to clay particles which soften easily when water is introduced. Therefore, the strength caused by this cementation may be lost in the CID tests after saturation resulting in lower strength parameters.
Figure 5.15: Comparison Between Predicted and Measured Tip Resistances for Cemented Loess Deposit (Cohesion Intercept is 150 kPa and Friction Angle is 21 Degrees)
Figure 5.16: Comparison Between Predicted and Measured Sleeve Resistances for Cemented Loess Deposit (Cohesion Intercept is 75 kPa and Friction Angle is 15 Degrees.)
6.1 Discussion

Cemented sand deposits were searched for a field study to assess the reliability of the prediction schemes. The loess bluffs in Natchez Trace Park and Waterways Experiment Station were selected for a field study. The study of grain-size and scanning microscope indicated that these slopes in Natchez Trace Park are cemented fine sand deposits. Samples taken from these slopes would simulate the laboratory specimens, but the Research Vehicle could not be taken to the top of these slopes due to the topography of the place. Therefore, tip and sleeve resistance could not be obtained for these slopes. Waterways Experiment Station is another location that some of the slopes were investigated. The loess bluff found consisted of 75-80 % silt and 20-25 % clay. An unconfined compressive strength of 125 kPa was obtained. This soil according to Rad's (1984) classification can be termed as weakly cemented deposits. Puppala's (1993) study was conducted on 1 to 2 percent artificially cemented sands. Based on the same classification chart, these materials are classified as very weak to weakly cemented deposits (unconfined compressive strength < 30 kPa). Despite the variations in unconfined compressive strengths, the weakly cemented loess deposits in Waterways Experiment Station seem to be appropriate for a field study on cementation.

Several factors had to be considered because of the difference between grain size distribution of loess deposits and Monterey No 0/30 sand. The first question is whether drained or undrained conditions prevail during cone penetration testing in the cemented loess deposit. It is known that if the hydraulic conductivity of the soil is high enough to dissipate the excess pore
pressures developed, drained conditions would prevail during cone penetration testing. Puppala (1993) observed drained conditions due to the high hydraulic conductivity of the sand. The loess deposit has lower hydraulic conductivity \((7 \times 10^{-5} \text{cm/s})\) than the artificial cemented sand specimens \((2.5 \times 10^{-3})\), therefore undrained conditions may prevail during cone penetration on cemented loess bluffs. It was decided to decrease the penetration rate to a level which possibly resulted in drained conditions. Three different penetration speeds \((0.25 \text{ cm/sec}, 1 \text{ cm/sec and } 2 \text{ cm/sec})\) were selected. The results of these tests showed that tip and sleeve resistances were not affected by the penetration speed. The penetration speeds were sufficient to dissipate all the excess pore pressure developed during cone penetration testing. Therefore, drained conditions were assumed to prevail during penetration testing.

In the present study that assesses the reliability of the prediction scheme proposed by Puppala (1993), tip resistances were calculated using the plastification versus dilation angle chart. Puppala (1993) used Monterey No. 0/30 and proposed a prediction scheme for this sand. Dilation was observed during CID tests conducted on this sand. Therefore, Puppala (1993) proposed a chart for dilation angle versus plastification angles by demonstrating his predicted tip resistance which correlate well with his measured tip resistance. An assumption was made that Puppala's (1993) chart would be also valid on the contraction side. The line \((\beta = 1.43 \nu - 3.88)\) is used on contraction side to estimate a plastification angle to use in the J & S theory.

### 6.2 Summary and Conclusions

The following conclusions are drawn from this research.

1. The effect of cementation on drained behavior of artificially cemented and
uncemented Monterey No. 0/30 sand was investigated. An increase in the confining pressure increases the strength of the specimens. An increase in the cementation level also increases the brittleness and peak strength of the specimens. The sands show dilation during shear. An increase in confining pressure decreases the dilation.

The p-q diagrams for the uncemented and artificially cemented specimens with different relative densities and cement contents are prepared. These figures reveal that an increase in the cement content increases the cohesion intercept. The cohesion intercept also increases when the relative density increases for the same cement content. Rad (1984) reported an explanation for above behavior, That is: "this is likely due to the fact that at higher densities, more contact exists between the sand particles and thus more opportunity exists for cementation". An increase in relative density also increases the friction angle.

2. The repeatability and accuracy of the calibration chamber test results obtained by Puppala (1993) were demonstrated by conducting five cone penetration tests in the calibration chamber. Four of the present and Puppala's (1993) test results render similar values even though there is a slight difference between them. This difference is most probably due to the difference in relative densities. The one conducted at a relative density of 88 % and cementation level of 1 % at confining pressure of 300 kPa was different. The result of this test gave a lower tip resistance than that of Puppala (1993). However, due to the high relative density and confining pressure, the test was stopped several times during the soundings in an attempt not to damage the cone penetrometer. The author believes that this was main reason for the difference.

3. Cone penetration soundings were performed in the cemented loess bluff. These test were conducted using three different penetration speeds, 0.25 cm/sec, 1 cm/sec, and 2cm/sec in order to assess the effect of drainage. Test results showed
that penetration speed had an insignificant effect on these results. Also, test results showed that there are mainly two different layers in this slope. The first layer from 0 m to 7.0 m showed higher tip resistance (an average of 40-50 kg/cm²) than second layer 7.0 m to 15 m (an average of 15-20 kg/cm² of tip resistance). Also, there is no effect of confinement in the first layer on tip resistance. This shows that cohesional term is predominant in this deposit. In the second layer, there is slight effect of confinement on tip resistance.

6. The results of CID tests conducted on naturally cemented specimens obtained from the loess bluff at 0-0.5 m showed that the soil (WES 2) has a cohesion intercept of 50 kPa and a friction angle of 21 degrees. The residual shear strength parameters for the soil in this layer are 35 kPa and 22 degrees. These tests were conducted at confining pressures of 100, 200, and 300 kPa.

The results of CID tests conducted on naturally cemented specimens obtained from loess bluff at 7.0-7.5 m showed that the soil (WES 1) has a cohesion intercept of 7 kPa and a friction angle of 25 degrees. The residual strength parameters for the soil in this layer are 7 kPa and 25 degrees. The unconfined compression strength of 125 kPa was found to be the same for two layers of the soil.

7. Using the shear strength parameters noted above and (D & M) bearing capacity theory, tip resistance is predicted. Also, (J & S) bearing capacity theory is used to predict tip resistance. Predicted tip resistance correlate quite well with measured tip resistance in the lower deposit (>7 m). However, the predicted tip resistance was lower than measured tip resistance within 7 m zone.

Sleeve resistance was also predicted. The predicted sleeve resistance values correlate quite well with measured sleeve resistance in the second layer, (7-15 m). However, the predicted sleeve resistance is lower than measured sleeve resistance in the first layer, (0-7m).
The strength parameters of the soil in the first layer were backcalculated using the Janbu and Senneset (1974), Durgunoglu and Mitchell (1973) theories and measured tip resistance. A cohesion intercept of 150 kPa and friction angle of 20-21 degrees were obtained. Backcalculation using sleeve resistance values gives 75 kPa of cohesion intercept and 15 degrees of friction angle.
REFERENCES


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LoadTrac Operator's Manual, 1988 GEOCOMP Corporation


APPENDIX A. CID TRIAXIAL TESTS

In this section, the stress-strain and volume change behaviors of artificially cemented and uncemented Monterey No. 0/30 sand with cementation level of 0, 1, and 2 % are presented. This section also covers the peak and residual strength parameters in the form of p-q' diagrams for artificially cemented and uncemented Monterey No. 0/30 sand with cementation level of 0, 1, and 2 %.
Figure A.1: Stress-Strain Curves for Uncemented Monterey No. 0/30 Sand Specimens at Relative Density of 69 %.
Figure A.2: Stress-Strain Curves for Uncemented Monterey No. 0/30 Sand Specimens at Relative Density of 89%.
Figure A.3: Stress-Strain Curves for Artificially Cemented Monterey No. 0/30 Sand Specimens with One Percent Cement at Relative Density of 57 %.
Figure A.4: Stress-Strain Curves for Artificially Cemented Monterey No. 0/30 Sand Specimens with One Percent Cement at Relative Density of 65%.
Figure A.5: Stress-Strain Curves for Artificially Cemented Monterey No. 0/30 Sand Specimens with One Percent Cement at Relative Density of 88%.
Figure A.6: Stress-Strain Curves for Artificially Cemented Monterey No. 0/30 Sand Specimens with Two Percent Cement at Relative Density of 60%.
Figure A.7: Stress-Strain Curves for Artificially Cemented Monterey No. 0/30 Sand Specimens with Two Percent Cement at Relative Density of 70%.
Figure A.8: Stress-Strain Curves for Artificially Cemented Monterey No. 0/30 Sand Specimens with Two Percent Cement at Relative Density of 88%.
Figure A.9: Peak Strength Envelopes for Artificially One Percent Cemented Monterey No. 0/30 Sand Specimens at Relative Densities of 57, 65, and 88%.
Figure A.10: Peak Strength Envelopes for Artificially Two Percent Cemented Monterey No. 0/30 Sand Specimens at Relative Densities of 60, 70, and 88 %. 

- $D_r = 60\%$, $\phi' = 38.6$, $c' = 16$ kPa
- $D_r = 70\%$, $\phi' = 39.7$, $c' = 22$ kPa
- $D_r = 88\%$, $\phi' = 40.0$, $c' = 31$ kPa

C.C. = 2 %
Figure A.11: Residual Strength Envelopes for Uncemented Monterey No. 0/30 Sand Specimens at Relative Densities of 55, 69, and 89%.
Figure A.12: Residual Strength Envelopes for Artificially One Percent Cemented Monterey No. 0/30 Sand Specimens at Relative Densities of 57, 65, and 88 %.
Figure A.13: Residual Strength Envelopes for Artificially Two Percent Cemented Monterey No. 0/30 Sand Specimens at Relative Densities of 60, 70, and 88%.
APPENDIX B. CONE PENETRATION TESTS

In this section, the results of in-situ cone penetration test conducted on naturally cemented loess deposit with penetration speeds of 2, 1, and 0.25 cm/sec are presented.
Figure B.1: The Result of In-Situ Cone Penetration Test Conducted on Naturally Cemented Loess Deposit with a Penetration Speed of 2 cm/sec.
Figure B.2: The Result of In-Situ Cone Penetration Test Conducted on Naturally Cemented Loess Deposit with a Penetration Speed of 1 cm/sec.
Figure B.3: The Result of In-Situ Cone Penetration Test Conducted on Naturally Cemented Loess Deposit with a Penetration Speed of 0.25 cm/sec.
APPENDIX C. STRAIN RATE

The result of the consolidation tests, calculation of the strain rate, and the methodology proposed by Bishop and Henkel (1965) are presented in this section. This methodology is given with detailed explanation in Soil Laboratory Testing, Volume 3, (Head, K.H., 1986).
Consolidation test is conducted in order to obtain $t_{100}$. The test is conducted on a naturally cemented saturated specimen. During consolidation, the sample volume change is recorded and plotted against square-root time (minutes). The initial part of the plot is obtained linear. This straight line is extended to the horizontal line that represents the end of consolidation. This point, A, is shown in the Figure C.1. At point A, the value of square-root of $t_{100}$ is read off. $t_{100}$ gives the time of theoretical 100% consolidation. Using the following formula,

$$t_f = \left( \frac{20 \cdot \lambda}{\pi \cdot \eta} \right) \cdot t_{100}$$

the time to failure, $t_f$ is calculated. $\eta$ is a factor which depends on drainage condition. Table 15.4 (Soil Laboratory Testing, Volume 3 by Head, 1986) presents these values for different drainage conditions. In this case, drainage is from one end therefore $\eta$ is 0.75. Values of $\lambda$ are also given in Table 15.4 and it is one in this case. $t_{100} = 2$ min is read off from the figure. Once, $t_f$ is known, strain rate is calculated assuming the failure at 10% strain. In this case, $t_f$ is 17 minutes. Deformation is approximately 15 mm for a specimen having 150 mm height and failing at 10% strain. This gives approximately a strain rate of 0.90 mm/min. Considering this value, a strain rate of 0.30 mm/min is selected in order to be on safe side.
Figure C.1: The Result of Consolidation Test, Volume Change Versus Square-Root Time (minutes)
APPENDIX D. SATURATION

The calculation of the Skempton's B parameter and the cell and backpressure increments are explained in this section.
Saturation is checked by measuring the Skempton's B parameter. Once over 0.90 is achieved for the value of B parameter, saturation procedure is ended. First small amount of cell pressure, 50 kPa, and 25 kPa of backpressure are applied. Water flows through the specimen. After water starts draining from the drainage valve, the valve is closed and cell pressure and backpressure are increased according to the current B parameters. Following formula is used to calculate B parameter.

\[ \Delta u = B \cdot [\Delta \sigma_3 + A \cdot (\Delta \sigma_1 - \Delta \sigma_3)] \] .........................................(D.1)

Since \( \Delta \sigma_1 = \Delta \sigma_3 \) in triaxial cell, formula becomes as follow

\[ \Delta u = B \cdot \Delta \sigma_3 \] .................................................................(D.2)

Using water pressure transducer, \( \Delta u \) is measured after the cell pressure is increased. The next increment of cell pressure, \( \Delta \sigma_3 \), and backpressure, \( U_{bp} \), are calculated as follows.

\[ \sigma_{cell} = 50 + \Delta \sigma_3 \] .................................................................(D.3)
\[ U_{bp} = 25 + B \cdot \Delta \sigma_3 \] .................................................................(D.4)
\[ \sigma_{cell} - U_{bp} = \sigma' \] .................................................................(D.5)

Where \( \sigma' \) is confining pressure. The required value of confining pressure is also achieved with small increments. For example, in this case if required confining pressure is 200 kPa, the pressure is increased in 50 kPa increments. This procedure is repeated until the required confining pressure and required B parameter are achieved.
APPENDIX E. THE SCANNING ELECTRON MICROGRAPHS

This section presents the scanning electron micrographs of the soil taken from the loess bluff in Waterways Experiment Station in Vicksburg, Mississippi. Also, scanning micrographs of the soil taken from the loess bluff in Natchez Trace Park, Mississippi are given here.
Figure E.1: The Scanning Micrographs of the Soil (WES 1) Taken from the Loess Bluff in Waterways Experiment Station in Vicksburg, Mississippi
Figure E.2: The Scanning Micrographs of the Soil (WES 2) Taken from the Loess Bluff in Waterways Experiment Station in Vicksburg, Mississippi
Figure E.3: The Scanning Micrographs of the Soil Taken from the Loess Bluff in Natchez Trace Park, Mississippi
VITA

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Title of Thesis: Cone Penetration in Cemented Deposits - A Field Study

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Date of Examination:

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