Effect of Cementation on Cone Resistance in Sands: A Calibration Chamber Study.

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Puppala, Anand Jagadeesh, Ph.D.

The Louisiana State University and Agricultural and Mechanical Col., 1993

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EFFECT OF CEMENTATION ON CONE RESISTANCE IN SANDS: A CALIBRATION CHAMBER STUDY

A Dissertation

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 Doctor of Philosophy

in

The Department of Civil Engineering

by

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To My Mom and Dad

- Chinni
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Abstract

An understanding of the effect of cementation on geotechnical properties of soil deposits is gaining increasing attention in the profession. When low levels of cementation in sands are neglected, pile capacity and slope stability are underestimated and liquefaction is overestimated. The recent Loma Prieta earthquake led to failures along the northern Daly City bluffs causing catastrophic failures in residences scattered on these slopes. These failures were not anticipated, possibly due to the confidence felt in constructing on bluffs of cemented deposits. It is essential to devise schemes to identify cementation in soil investigations and develop methods in evaluating engineering characteristics of cemented deposits. The objective of this study is to develop a method to identify cementation in sands and assess the engineering characteristics of cemented sand deposits using the cone penetration testing scheme.

The scope of the study includes evaluation of the effect of cementation on cone penetration testing (experimental model) and comparison of these experimental results with theoretical models of penetration mechanism in cemented sands. Existing models based upon the bearing capacity theories and cavity expansion models are utilized in theoretical modeling. A constitutive model is developed for strength-deformation behavior of cemented sands and is used in theoretical modeling.

Artificially cemented Monterey No. 0/30 is used in the calibration chamber study. A total of 30 tests are conducted at three ranges of relative density (45-55, 65-75 and above 85 %), three confining pressures (100, 200 and 300 kPa) and three different cement content (0, 1 and 2 %). Pluviation method is used for specimen preparation. Specimens are cured for 7 days, transferred into the flexible wall calibration chamber and then consolidated under $K_0$ conditions. Penetration testing was conducted with a 1.27 cm diameter miniature cone which reduced the chamber size effects on the results significantly. Separate drained triaxial tests provided the necessary parameters for strength-deformation modeling of cemented sands.

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The experimental model results coupled with the theoretical model predictions provide a semi-empirical and empirical schemes for evaluating engineering characteristics of cemented sand deposits. An assessment of the applicability of these models in prediction of cementation in such deposits is also provided. The results indicate that tip resistance and sleeve friction in cone penetration testing provide a reasonable assessment of cementation. The charts and the analysis method provided can be used to estimate the engineering characteristics of such deposits. It is found essential that reliability and accuracy of the proposed methods of analysis be evaluated by insitu tests in a well-documented, naturally cemented sand deposit.
Chapter 1

INTRODUCTION

1.1 Introduction

Naturally cemented deposits are very common throughout California, Texas, along the banks of the Mississippi (loess deposits in the Vicksburg area), India, Canada and the world. These deposits are often characterized by their ability to withstand steep natural slopes (Clough et al., 1981). Large deposits of cemented sands are found along the California coast and are typically 60 degrees or steeper from the horizontal and reach heights of 100 m (Clough et al., 1981). Similar deposits can be found near Natchez and Vicksburg area in Mississippi. Plate 1.1 shows the near vertical slopes in a cemented deposit along the Natchez Trailway near Natchez, Mississippi. Recent data and evidence suggest that even the cleanest sand deposits are naturally cemented and therefore engineering judgments made from specimens reconstituted in the laboratory may not be valid (Mitchell and Solymar, 1984). This cementation is generally provided by agents such as silica, hydrous silicates, hydrous iron oxides, and carbonates deposited at the point of contact between sand particles (Clough et al., 1981). In some cases, the cementation is due to welding at the contacts or time dependent strength gain (Mitchell and Solymar, 1984). This type of cementation generally results in low to moderate cementation.

An understanding of the effect of this low to moderate degrees of cementation on the static and dynamic strength and deformation behavior of sands is becoming increasingly important in design and analysis in geotechnical engineering. At the present state of the art, the effect of cementation on strength-deformation behavior of sands is neglected in the design since cementation often improves the strength properties. However, recent studies indicate that neglecting cementation, particularly the smaller degree of cementation bonds results in overestimation of the liquefaction resistance, underestimation of the strength of the soil deposits and also underestimation of the stability of the slopes (Rad and Clough, 1982; Poulos, 1980; Frydman et al., 1980). Slope failures are very common in such cemented deposits and they lead
Plate 1.1: A Vertical Slope in Cemented Loess Deposits Along Natchez Trail, Natchez, Mississippi
to loss of life and property (Clough et al., 1981). Recent Loma Prieta San Francisco earthquake led to failures along the cemented northern daly city bluffs (Sitar, 1990) (Plate 1.2). Liquefaction phenomenon was also observed at several sites during the Loma Prieta earthquake.

Cementation is also a popular method used for soil stabilization. When soils with unsatisfactory engineering properties are encountered, some method of stabilization by using cement, fly ash and other additives is needed prior to any future construction on that soil. This procedure is frequently used in many engineering projects like improvement of subgrades and airport runways, stabilizing the slopes and embankments and also increasing the bearing capacity of the soil using cemented sand columns. The loads imposed by traffic have to be transferred to soil layers capable of supporting them without shear failure or excessive deformations. In brief, cement stabilization improves modulus of elasticity, shear modulus, coefficient of subgrade reaction and unconfined compression.

The difficulty in sampling natural or stabilized cemented deposits prompts the need to use insitu testing schemes (Beckwith and Hansen, 1981; Bachus et al., 1981; Frydman et al., 1980). Among several different insitu testing methods, cone penetration testing is gaining wide acceptance and use in the USA and the world due to its repeatability, economy and capability to provide accurate, repeatable vertical soil profiles and pertinent engineering parameters related to the sounded deposits.

Cone penetration tests in granular deposits are currently used to measure tip resistance $q_c$, sleeve friction $f_s$, and if piezocone is used, the total pore pressure $u_t$ along the tip and/or the shaft of the penetrometer. Estimates of relative density of the uncemented granular deposits are made using charts obtained in calibration chambers. Internal friction angles are obtained indirectly by correlating the tip resistance with the relative density. Charts are updated for the influence of other variables such as overconsolidation by conducting tests in a calibration chamber (Schmertmann, 1977; Villet and Mitchell, 1981; Baldi et al., 1981; Jamiolokowski, 1985).

The influence of cementation on penetration resistance is yet to be investigated. Preliminary studies investigating the possible effects of low degrees of cementation in cone penetration indicated that cementation increases the tip and friction resistances and decreases the friction ratio (Rad and Tumay, 1986; Akili and Nabil, 1988). These
Plate 1.2: Slope Failure Along a Weakly Cemented Deposit in California due to the Loma Prieta Earthquake (Photograph by Dr. Wayne Clough)
studies clearly showed that estimates of engineering parameters of cemented deposits using the available methods which are developed for clean sands, would be invalid.

Hence, it is proposed to initiate a testing scheme involving cemented specimen testing in calibration chambers. The mechanical behavior of cemented sand deposits is studied by using artificially cemented sands in the laboratory. Previous studies indicate that artificially cemented sand deposits simulate the behavior of natural deposits. Hence, artificially cemented specimens are used in the tests. Results from these tests are used to provide a classification scheme for cemented deposits and also to provide a methodology for estimating the strength properties.

1.2 Objectives

The proposed study aims to develop the scheme and methodology to determine the strength parameters of artificially cemented sands by cone penetration testing conducted in a calibration chamber.

The objectives of this work are:

1. to perform a literature review about strength-deformation behavior of artificially cemented sands,

2. to perform laboratory triaxial and unconfined compression tests on artificial cemented specimens for reassessing the available data and evaluating the necessary strength-deformation parameters for both modeling and calibration purposes,

3. to conduct calibration chamber tests on artificially cemented and uncemented specimens using a miniature cone penetrometer and to evaluate these test results in the study of various variables like chamber size, boundary conditions, sand compressibility, size and shape of the aggregates and cementation,

4. to simulate the cone penetration with existing bearing capacity and cavity expansion models and then develop a methodology to evaluate the strength parameters in light of comparisons of the theoretical predictions with experimental results.
1.3 Organization of the Manuscript and Summary

The following presents a brief summary of the contents of various chapters:

Chapter 2 covers the literature review pertaining to cemented sands and their behavioral studies conducted by various investigators, cone penetration testing and their applications, calibration chambers and various methods of analyses used in cone penetration testing. Definitions, various factors causing cementation and cemented soil structure are presented. Static and dynamic behavior of cemented sands and their findings are summarized. In another section, cone penetration testing history is briefly reviewed. This is followed by a discussion on calibration chambers and their applications using cone penetrometer. Studies involving cemented specimens in rigid chambers are presented along with various studies on uncemented sands. Different methods used in the analysis of cone penetration testing, their advantages and disadvantages are also discussed.

Chapter 3 presents the methodology for various tests used in the present investigation. Equipment and their use are also briefly described. Cemented and uncemented specimen preparation procedures are documented. Large scale specimen preparation for calibration chambers is then presented. Saturation procedures used in these specimens are explained followed by the testing procedures adopted in the tests.

Chapter 4 covers the results of undrained, drained triaxial and unconfined compression tests. Total and effective stress parameters are evaluated and compared with drained parameters. Critical state lines for both cemented and uncemented Monterey No. 0/30 sand are presented. Curing period influence on unconfined compression strength is evaluated and discussed. Juran-Guermazi model is updated for the effect of cementation and is used to model the drained and undrained triaxial test results. These modeling parameters are later used in cavity expansion modeling.

Chapter 5 summarizes the cone penetration testing conducted on both cemented and uncemented specimens in the calibration chamber. Specimens are first consolidated under $K_0$ conditions. Cone penetration tests are then conducted under zero lateral strain boundary conditions (Traditional BC 3). These results are first assessed for repeatability, precision and accuracy. Influence of relative density, cement content and confining pressures on tip and friction resistances are also evaluated.
Chapter 6 presents a summary of various factors affecting the calibration chamber testing. Influence of boundary conditions, chamber size effects, grain size and shape, compressibility and crushability of the sands tested and cementation are investigated. Several chamber investigations on various sands are used in the above analysis.

Chapter 7 compares the predictions of theoretical models with experimental results. Tip and friction resistances are first predicted using two rigid-plastic bearing capacity models and two cavity expansion model. The elasto-plastic model developed in Chapter 4 is then used in an incremental cavity expansion analysis of the problem. The theoretical predictions are compared with the experimental results. Based upon this evaluation, a methodology is developed for evaluating the cohesion intercept and relative density from cone penetration testing in cemented sands. Empirical correlations are developed for estimating the relative density and unconfined compression strength. Two procedures are introduced. The first procedure is based on parameters obtained by normalizing the tip resistance and effective stress and the second procedure is based on the steady state line concept. The influence of various factors such as boundary conditions, chamber size, sand grain size and shape, compressibility of the sands and cementation on cone test results in chambers are also evaluated.

Chapter 8 summarizes the findings, conclusions and the shortcomings of this study. Recommendations for future research are also provided.
Chapter 2

SYNTHESIS OF AVAILABLE INFORMATION

2.1 Introduction

Most sands in nature are lowly cemented. Cementation has a significant influence on their engineering properties (Schmertmann, 1991). In normal practice, geotechnical engineers do not account for cementation in sands in design and analysis. However, neglecting the cementation bonds will result in underestimation of the strength and the liquefaction resistance. The significance of studying the behavior of cemented sands under dynamic loading gains more importance due to slope failures in cemented deposits during the recent earthquakes in the San Francisco Bay area.

The difficulty in sampling and also non availability of samples of different density and cementation levels as needed to conduct comprehensive studies led many investigators to use artificially cemented specimens. Past studies conducted by Clough et al. (1981) and Rad and Clough (1982) demonstrated that 1 to 2 % artificially cementation using Portland cement will simulate the naturally cemented sands. These studies on artificially cemented sands have also been beneficial in evaluating the feasibility of improving subgrades under highways and runways, stabilizing slopes in embankments and cuts and also in improving the bearing capacity by adopting cement stabilization.

In the present state of the art, there is no insitu method to identify and determine the engineering characteristics of naturally cemented deposits. The present investigation aims at the above need by using cone penetration testing. Artificially cemented specimens are prepared and tested in calibration chambers. These results are used in preparing a prediction scheme to identify naturally cemented sands and also to determine the engineering characteristics.

In this chapter, a review of the past studies on static and dynamic tests conducted on artificially and naturally cemented sands is presented. The engineering characteristics of cemented Monterey No. 0/30 and No. 0 sand are discussed. The presented static and dynamic test results are used in the later chapters in analysis.
of the results and in preparing semi-empirical relationships. The history and applications of cone penetrometer and calibration chambers are then reviewed. In section 2.4, the limited data available on cone penetration testing of cemented sands under low confining stresses are reviewed. The last section covers the different methods of analysis used in the interpretation of cone penetration results. The advantages and disadvantages of each method are also discussed.

2.2 Engineering Behavior of Cemented Sands

Sand deposits may undergo a significant loss of strength due to sample disturbance, thus behaving similar to sensitive clays (Mitchell and Solymar, 1984). Recent evidences show that freshly deposited or densified saturated clean sands may exhibit time-dependent stiffening and strength gain. These phenomena appear to be due to the cementation at interparticle contact points (Mitchell and Solymar, 1984). It is necessary to consider the effect of this low level cementation when evaluating the results of laboratory tests on reconstituted samples, in assessment of ground modification using deep densification, in evaluation and interpretation of relative density measurements, and in estimation of liquefaction potential. The magnitude of increase in cone penetration resistance at the main dam foundation at Jebba Hydroelectric Development in Nigeria, over a period of several weeks to months following deep densification, clearly suggested the time dependent strength gain in the sandy deposits. Similar observations were noted by Durante and Voronkevich in the hydraulically placed embankments (Mitchell and Solymar, 1984). The time dependent strength gain was sufficient enough to satisfy design criteria, however sampling or cone testing immediately after construction rendered different conclusions. Hence, it is important to understand the effect of cementation on engineering behavior and the possible mechanisms that cause the phenomenon in sands. The next few sections cover these aspects and also review the fundamentals of different cementation processes encountered in deposits other than sands.

2.2.1 Definitions and Physio-Chemical Characteristics

Cemented soils are defined as soils composed of sand or gravel sized particles or fragments of rocks bonded together by a cementing agent to form a larger com-
posite structure with distinctive geological and geotechnical properties (Al-Ghanem, 1989). This bonding is due to the cementation by physical or chemical processes or a combination of both. The processes will generally take place over a period of time.

The time dependent strength gain in sands at a test site densified by vibro compaction and blasting is attributed to different mechanisms (Mitchell and Solymar, 1984). The first mechanism discussed is development of excess pore pressures and their dissipation (Mitchell and Solymar, 1984). This may be true in case of cohesive clays, but in sands, the dissipation takes place rather instantaneously. Hence, it can not be considered as a process leading to strength gain in sands.

The second mechanism is that explosive gases after blasting may cause the strength increase as a result of an increase in sand compressibility. However, this mechanism could not be accounted for strength gain over a time period of weeks to months (Mitchell and Solymar, 1984). The third mechanism is the thixotropic strength gain. This phenomenon is evident in case of fine grained soils; however, in sandy soils, the extent of strength loss on disturbance followed by strength gain at rest is not well established.

Mitchell and Solymar (1984) offers the most probable cause or mechanism as formation of silica acid gel films on particle surfaces and precipitation of silica or other material from solution or suspension as a cementation species at particle contact points. They propose that this gel adheres to the surface in a thin layer and has cementing properties. The dissolution and precipitation of silica in the form of amorphous silica and crystalline quartz may lead to cementation (Mitchell and Solymar, 1984). Equilibrium relationships are difficult to obtain with silica. Equilibrium, if reached at all, will require weeks to months, i.e., time periods consistent with the observed strength increase in the field.

The other processes that lead to cementation are the presence of metallic ions such as Al and Fe. Mitchell and Solymar (1984) propose formation of crystalline iron oxide coatings which may cause cementation. Pressure solution due to high stress at grain contact points also leads to preferential solution (Mitchell and Solymar, 1984). The liberated SiO$_2$ supersaturates the pore water so that some SiO$_2$ may precipitate as quartz over growths and causes interpenetration of grains (Mitchell and Solymar, 1984).
All the above mechanisms depend on sand particle surface characteristics, the amount and type of cementing agent, the ground water movement and the extent/amount of weathering. The common cementing agents that are naturally found are: silica, iron oxides, carbonates in marine environments, clay and silt (Rad, 1984; Al-Ghanem, 1989).

2.2.1.1 Cementation in Sands

Cemented sands cover extensive areas in the U.S. (California, Texas, Mississippi, Arizona, North Louisiana) and also many regions like the Middle East, South East Asia and Africa. They are formed in arid and semi-arid environments. The strength properties of these soils are different from those generally observed in geotechnical engineering. This difference is due to the presence of chemical agents like hydrous silicates, iron oxides, calcium carbonate, calcium sulphate and sometimes the clay particles which bond adjacent particles in the soils. This bond which describes the coherence between the particles is known as cementation and the chemical agents are known as cementing agents.

The three reported causes of cementation in sands are:

1. The welding between the particles at their contact points due to the internal heat at the time of deposition or due to prolonged pressure at prominent points of contact between grains (Lee, 1975, Mitchell and Solymar, 1984),

2. The presence of cementing agents like silica or siliceous cement, calcium carbonate or calcareous cement, clay or argillaceous cement and iron bearing minerals or ferruginous cement (Krynine and Judd, 1957, Mitchell and Solymar, 1984). The cementation described by Mitchell and Solymar (1984) is of this kind.

3. In some cases, clay may also participate in the bonding. These bonds are weak in strength and such bonding is generally encountered in loessial soils.

2.2.1.2 Cementation in Collapsing Soils

Collapsing soils are defined as soils which normally have some strength but experience a loss of volume upon loading, wetting or both (Al-Ghanem, 1989). These soils are found in many parts of the world and can be formed in different environments
such as loessial, colluvial, alluvial, subaerial and aeolian (Al-Ghanem, 1989). These soils are porous in fabric and they are geologically young. Their structure consists of sand grains bonded in loose silty sand. The fine fraction that exists in small gaps between adjacent grains of the soils will undergo local compression which ultimately bonds the larger grains.

Clay is another binding agent between sand and silt grains. Several structures are formed with clay as binding agent and these cementation bonds can be destroyed with the addition of water (Al-Ghanem, 1989). Loessial soils are wind-borne, naturally cemented collapsing soils. Calcareous materials and/or clay is generally the binding agent. In these soils, the bonds are weakened by either leaching out or softening of the binders.

2.2.1.3 Cementation in Rocks

When the fragments of a rock are bonded firmly together with a cementing agent to form a new rock type, the resulting material is classified as a cemented rock (Al-Ghanem, 1989). This cementation can take place either from the infiltration of water carrying chemicals or the dissolution of certain minerals in the mass to form new bonding material (Al-Ghanem, 1989).

Sedimentary rocks are formed due to the consolidation and cementation of sediments. These are the end products of the weathering process. The most common cements found in these rocks are: silica or siliceous cement, calcium carbonate, argillaceous cement and iron oxides. Limestones are another example of cemented rocks. They are composed of calcium and magnesium carbonates, and are found in marine deposits (Al-Ghanem, 1989). Cemented rocks exhibit higher compressive strength when quartz is the cementing medium and lower strength is obtained when they are cemented entirely or partially with clay.

2.2.1.4 Structure of Cemented Granular Soils

The cementation process depends on a number of factors including the type and amount of cementation, the degree of packing, the density and characteristics of the soil particles and the method of deposition. Because of these factors, the cemented
soil structure also varies. In fact, the structure, in some cases, explains the chemical agent and the process in the cementation that might have taken place.

Sowers and Sowers (1979) classified the structure into two categories: *matrix structure* and *skeletal structure*. Figure 2.1 shows these structures. The matrix structure develops when the volume of the bulky grains is less than about twice the amount of binder. In other words, most of the volume is occupied by the binder and there is little contact between the bulky grains. The strength of these structures depends upon the strength of either the binder or bulky grains, whichever is weaker.

The skeletal structure develops when the volume of bulky grains is more than twice the volume of the binder. This structure can be subdivided into either contact-bond structure or a void-bond structure. In contact bond structure, particles are cemented at the contact points. This structure can be formed in soils with large particle sizes. The structure is relatively rigid and incompressible. This bond is not stable and can be lost due to leaching by ground water. The void-bond structure is due to the contacts among individual particles. The voids are filled with binders such as carbonates, iron oxides and silicates. Here, cementation develops subsequently after the structure forms. This structure is more stable than contact bond structure.

### 2.2.2 Displacement Rate Controlled Stress-Deformation Behavior (Static Behavior)

The shearing resistance of artificially cemented sands can be considered to be composed of two elements, one of which is independent of the normal stress on the failure plane (cohesion intercept), and the other increasing with the normal effective stress on the failure envelope (friction angle). The increase in cohesion intercept with the increase in cement content has been shown by several investigators (Rad and Clough, 1982; Acar and El-tahir, 1986).

The static triaxial properties of cemented sands are presented in this section. A total of 43 drained triaxial tests conducted on cemented sands are reported by Rad and Clough (1981). Monterey No. 0 sand, a commercially available washed and sieved beach sand was used in their investigation. Artificial cementation of 1 and 2 % was used. The important variables used in this study were relative density ($D_r$), cement content (C.C.) and confining pressure ($\sigma_c$). Three ranges of relative density...
Figure 2.1: Various Structures in Cemented Soils (Sowers and Sowers, 1979)
(20 - 30 %, 45 - 55 %, 70 - 85 %), and four ranges of confining pressures (35, 100, 200 and 300 kPa) and three ranges of cement content (0 (uncemented), 1 and 2 %) were used. A 14 day curing period was adopted. The drained triaxial tests conducted on 0, 1 and 2 % cement content and 45 - 55 % relative density are shown in Figures 2.2, 2.3 and 2.4. These data are used for obtaining modulus parameters like Young’s modulus and shear modulus.

The increase in relative density increases the strength of the specimen. In unce­
mented cohesionless sands, a rise in relative density results in the increase of friction angle (Figure 2.5). Similar observations are made in cemented sands. This increase is due to the increase in relative density which implies a densely packed soil mass with more contacts between the soil grains. However as depicted in Figure 2.5, the increase in cement content at a given relative density does not result in any significant rise in friction angle.

The peak and residual strength parameters are reported in Table 2.1. Figure 2.6 demonstrates that the increase in the relative density and cementation of the speci­men results in an increase in the cohesion intercept. This phenomenon is attributed to the fact that as relative density increases, the number of contacts between the particles increase and consequently stronger bonds form (Acar and El-Tahir, 1986; Riccobono, 1985). Another important observation is that even though the cement content induces cohesion intercept at peak strains, this cohesion is almost zero at residual strains (Table 2.1). The cementation bonds will be destroyed at the peak failure and the residual friction angles at failure are approximately the same as that of uncemented sand.

In brief, the main conclusions drawn from the static loading response of cemented sands are summarized as follows.

1. Cementation increases the peak strength. Cementation results in a cohesion intercept due to the bonding between particles.
2. The strength is mainly due to cohesion at low confining stresses and strains and due to friction at higher strains.
3. Failure is of brittle nature.
4. Cementation has minor effect on the friction angle.
Figure 2.2: Drained Triaxial Results on an Uncemented Specimen of Monterey No. 0 Sand at Relative Density 45-55 % (Rad and Tumay, 1986)
Figure 2.3: Drained Triaxial Results on a 1% Cemented Specimen of Monterey No. 0 Sand at Relative Density 45-55% (Rad and Tumay, 1986)
Figure 2.4: Drained Triaxial Results on a 2% Cemented Specimen of Monterey No. 0 Sand at Relative Density 45-55% (Rad and Tumay, 1986)
Figure 2.5: Influence of Cement Content on Friction Angle of Monterey No. 0 Sand
(Rad and Tumay, 1986)
Figure 2.6: Influence of Cement Content on Cohesion (Rad and Tumay, 1986)

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Table 2.1: Strength Parameters of Cemented Sands (Drained Tests) (Rad and Clough, 1982)

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>Cement Content</th>
<th>Cohesion ($k_N$)</th>
<th>Friction Angle</th>
<th>Attraction ($k_N$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>peak</td>
<td>res</td>
<td>peak</td>
</tr>
<tr>
<td>31</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>33</td>
</tr>
<tr>
<td>45</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>35</td>
</tr>
<tr>
<td>77</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>39</td>
</tr>
<tr>
<td>20</td>
<td>1</td>
<td>4.4</td>
<td>0</td>
<td>33.0</td>
</tr>
<tr>
<td>47</td>
<td>1</td>
<td>8.6</td>
<td>0</td>
<td>35.5</td>
</tr>
<tr>
<td>71</td>
<td>1</td>
<td>12.4</td>
<td>0</td>
<td>37.9</td>
</tr>
<tr>
<td>18</td>
<td>2</td>
<td>10.0</td>
<td>4.4</td>
<td>32.6</td>
</tr>
<tr>
<td>40</td>
<td>2</td>
<td>17.0</td>
<td>9.6</td>
<td>34.8</td>
</tr>
<tr>
<td>80</td>
<td>2</td>
<td>30.0</td>
<td>18.9</td>
<td>38.8</td>
</tr>
</tbody>
</table>

5. Cementation results in an increase in dilation.

6. Residual friction angles of cemented and uncemented sands are similar.

2.2.3 Unconfined Compressive Strength, $q_f$

Unconfined compression strength, $q_f$ is generally used as an index to classify cohesive soils. Table 2.2 provides the proposed classification of cemented sands based on unconfined compressive strength values. This classification by Rad and Clough (1982) provides both simplicity and versatility. Rad and Clough propose the classification for all cemented soils irrespective of the cementing agent.

Acar and El-Tahir (1986) conducted unconfined compression tests on the artificially cemented soils. The tests were conducted on specimens of varying relative densities (40-50, 60-75 and above 80 %) and cement contents (1 and 2 %). A curing period of 14 days was adopted. The results of this study are presented along with Rad and Clough’s (1982) results in Table 2.3. Both studies used pluviation for specimen preparation. Table 2.3 indicates that both results are quite similar implying that there is not a significant difference between Monterey No. 0 and No. 0/30 sands. These results also demonstrate the repeatability in preparing artificially cemented specimens.
Table 2.2: Proposed Classification System for Cemented Granular Soils (Rad and Clough, 1982)

<table>
<thead>
<tr>
<th>CLASSIFICATION</th>
<th>$q_f$ (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 fingernail</td>
</tr>
<tr>
<td>Very strongly cemented</td>
<td>&gt; 3000</td>
<td>very low strength soft rock</td>
</tr>
</tbody>
</table>

$q_f$ - Unconfined Compressive Strength

According to the classification system, the 1 and 2% artificially cemented sands are categorized as very weakly cemented.

2.2.4 Hydraulic Conductivity

The influence of cement content on the hydraulic conductivity is presented (El-Tahir and Acar, 1983) in Figure 2.7. The increase in cement content results in reduction of permeability. This reduction is due to the clogging of the pores by the finer cement particles. As expected, denser specimens have lower permeabilities than loose specimens. The question then arises whether drained or undrained conditions prevail during cone penetration testing. This definitely will depend upon the ratio of the rate of penetration, $\delta$, to the permeability, $k$, of the medium, $\frac{\delta}{k}$. It is not well established above which $\frac{\delta}{k}$ values drained conditions will prevail. It is necessary to conduct experiments to assess the effects of $\frac{\delta}{k}$.

2.2.5 Compressibility

An equivalent Young's modulus is generally used for cases other than one dimensional compression (Schmertmann, 1977). The equivalent Young's modulus is
Compaction to 95% of Maximum Proctor Density
Compaction to 100% of Maximum Proctor Density

(a) 14 days old sample
(b) 28 days old sample
(c) 90 days old sample

Figure 2.7: Hydraulic Conductivity of Cemented Specimens (El-Tahir and Acar, 1983)
Table 2.3: UCS of Cemented Sands

<table>
<thead>
<tr>
<th>Cementation C.C.</th>
<th>Relative Density</th>
<th>$q_f$ (a)</th>
<th>$q_f$ (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>%</td>
<td>(kPa)</td>
<td>(kPa)</td>
</tr>
<tr>
<td>0</td>
<td>31</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>45</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>77</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>25</td>
<td>10</td>
<td>7</td>
</tr>
<tr>
<td>1</td>
<td>35</td>
<td>15</td>
<td>NA</td>
</tr>
<tr>
<td>1</td>
<td>50</td>
<td>19</td>
<td>20</td>
</tr>
<tr>
<td>1</td>
<td>80</td>
<td>28</td>
<td>30</td>
</tr>
<tr>
<td>2</td>
<td>25</td>
<td>22</td>
<td>25</td>
</tr>
<tr>
<td>2</td>
<td>35</td>
<td>33</td>
<td>NA</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>41</td>
<td>42</td>
</tr>
<tr>
<td>2</td>
<td>80</td>
<td>54</td>
<td>55</td>
</tr>
</tbody>
</table>

a - Acar and El-Tahir (1986)
b - Rad (1984)

calculated from drained triaxial test results. The Young's modulus at the 50 and 25% failure stresses are calculated and are shown in the Tables 2.4, 2.5 and 2.6.

2.2.6 Natural Versus Artificial Cementation

The difficulty in sampling naturally cemented deposits leads investigators to use artificially cemented deposits. The simulation of naturally cemented deposits by using artificially cemented laboratory specimens is discussed in this section. This discussion is primarily focused on the strength and deformation behavior under confined and unconfined loading. The study conducted by Clough et al. (1981) is used for comparison purposes as this is the only comprehensive study on the subject.

Naturally cemented soil samples were obtained from two sites, both located on the San Francisco Peninsula. The first location was situated west of Stanford University and the second was located on the bluffs along the Pacifica Coast at the northern end of the city, Pacifica, California. SLAC-1 and SLAC-2 were the sands collected from the first site and PAC-1, PAC-2 were the sands collected from the second site. The SLAC-2 and PAC-1 are weakly cemented and have a lower density and fines content than SLAC-1 and PAC-2 sands, which are strongly cemented. Block sampling method was used for obtaining undisturbed samples. Drained triaxial tests

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Table 2.4: Young's Modulus at the 50 and 25 % Failure Stresses for Uncemented Monterey No. 0 Sand (C.C. 0 %)

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Relative Density</th>
<th>Confining Stress</th>
<th>Young's Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>%</td>
<td>kPa</td>
<td>$E_{25}$</td>
</tr>
<tr>
<td>0</td>
<td>32</td>
<td>103</td>
<td>35.5</td>
</tr>
<tr>
<td>0</td>
<td>32</td>
<td>207</td>
<td>45.7</td>
</tr>
<tr>
<td>0</td>
<td>32</td>
<td>345</td>
<td>53.3</td>
</tr>
<tr>
<td>0</td>
<td>45</td>
<td>103</td>
<td>26.7</td>
</tr>
<tr>
<td>0</td>
<td>45</td>
<td>207</td>
<td>80.0</td>
</tr>
<tr>
<td>0</td>
<td>45</td>
<td>345</td>
<td>160.0</td>
</tr>
<tr>
<td>0</td>
<td>77</td>
<td>103</td>
<td>64.0</td>
</tr>
<tr>
<td>0</td>
<td>77</td>
<td>207</td>
<td>266.6</td>
</tr>
<tr>
<td>0</td>
<td>77</td>
<td>345</td>
<td>400.0</td>
</tr>
</tbody>
</table>

Table 2.5: Young’s Modulus at the 50 and 25 % Failure Stresses for Cemented Monterey No. 0 Sand (C.C. 1 %)

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Relative Density</th>
<th>Confining Stress</th>
<th>Young’s Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>%</td>
<td>kPa</td>
<td>$E_{25}$</td>
</tr>
<tr>
<td>1</td>
<td>25</td>
<td>103</td>
<td>17.7</td>
</tr>
<tr>
<td>1</td>
<td>25</td>
<td>207</td>
<td>35.6</td>
</tr>
<tr>
<td>1</td>
<td>25</td>
<td>345</td>
<td>64.0</td>
</tr>
<tr>
<td>1</td>
<td>50</td>
<td>103</td>
<td>35.5</td>
</tr>
<tr>
<td>1</td>
<td>50</td>
<td>207</td>
<td>49.2</td>
</tr>
<tr>
<td>1</td>
<td>50</td>
<td>345</td>
<td>67.1</td>
</tr>
<tr>
<td>1</td>
<td>80</td>
<td>103</td>
<td>64.0</td>
</tr>
<tr>
<td>1</td>
<td>80</td>
<td>207</td>
<td>85.3</td>
</tr>
<tr>
<td>1</td>
<td>80</td>
<td>345</td>
<td>91.4</td>
</tr>
</tbody>
</table>
Table 2.6: Young’s Modulus at the 50 and 25 % Failure Stresses for Cemented Monterey No. 0 Sand (C.C. 2 %)

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Relative Density</th>
<th>Confining Stress</th>
<th>Young’s Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>kPa</td>
<td>MPa</td>
</tr>
<tr>
<td>C.C.</td>
<td>$D_r$</td>
<td>$\sigma$</td>
<td>$E_{25}$</td>
</tr>
<tr>
<td>2</td>
<td>25</td>
<td>35</td>
<td>16.0</td>
</tr>
<tr>
<td>2</td>
<td>25</td>
<td>103</td>
<td>18.3</td>
</tr>
<tr>
<td>2</td>
<td>25</td>
<td>207</td>
<td>24.6</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>35</td>
<td>26.8</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>103</td>
<td>44.4</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>301</td>
<td>71.1</td>
</tr>
<tr>
<td>2</td>
<td>77</td>
<td>35</td>
<td>29.4</td>
</tr>
<tr>
<td>2</td>
<td>77</td>
<td>103</td>
<td>110.3</td>
</tr>
<tr>
<td>2</td>
<td>77</td>
<td>207</td>
<td>128.0</td>
</tr>
</tbody>
</table>

and unconfined compression tests were conducted.

Artificially cemented samples (C.C. 1, 2 and 4 %) were prepared by using Monterey No.0/30 sand and they were tested under identical conditions as above. The results of artificially and naturally cemented sands are shown in Table 2.7.

Some observations made from the above study are:

- The strength envelopes of the artificially cemented soils closely resemble those of the naturally cemented soils, except that the friction angles are somewhat lower.

- Both artificial and natural weakly cemented sands show a brittle failure mode at low confining stresses with a transition to ductile failure at higher pressures.

- Volumetric strains increase during shear at a faster rate and at a smaller strain for cemented sands (natural and artificial) than uncemented sands.

- The residual strength for a cemented sand is close to that of an uncemented sand, although some degree of residual cohesion intercept was observed for all the cemented sands investigated.

- Like uncemented sands, density, particle size and shape and grain size distributions all have a significant effect on the behavior of cemented sands.
Table 2.7: Strength Parameters of Artificial and Natural Cemented Sands

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Relative Density</th>
<th>Cohesion (kN/m²)</th>
<th>Friction Angle (°)</th>
<th>Unconfined Strength (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>peak res</td>
<td>peak res</td>
<td>peak res</td>
</tr>
<tr>
<td>Uncemented</td>
<td>31</td>
<td>0</td>
<td>0</td>
<td>35</td>
</tr>
<tr>
<td>AC2</td>
<td>74</td>
<td>46</td>
<td>5</td>
<td>34</td>
</tr>
<tr>
<td>AC4</td>
<td>74</td>
<td>143</td>
<td>20</td>
<td>35</td>
</tr>
<tr>
<td>AC4</td>
<td>90</td>
<td>152</td>
<td>25</td>
<td>41</td>
</tr>
<tr>
<td>SLAC-1</td>
<td>NA</td>
<td>365</td>
<td>75</td>
<td>49</td>
</tr>
<tr>
<td>SLAC-2</td>
<td>NA</td>
<td>12</td>
<td>6</td>
<td>40</td>
</tr>
<tr>
<td>PAC-1</td>
<td>NA</td>
<td>25</td>
<td>5</td>
<td>40</td>
</tr>
<tr>
<td>PAC-2</td>
<td>NA</td>
<td>175</td>
<td>60</td>
<td>37</td>
</tr>
</tbody>
</table>

Rad and Clough (1982) conclude that artificial cementation simulates natural cementation reasonably well. Cementation of 1 to 2% simulates very weakly cemented sands whereas more than 4% cementation is needed to simulate the behavior of strongly cemented deposits.

A bibliography of several studies conducted on both naturally and artificially cemented sands is presented in Table A.1 in Appendix A. The main conclusions of these investigations are reported in Table A.2.

2.2.7 Dynamic Characteristics - Introduction

A brief summary of the dynamic characteristics is compiled in this section. The primary objective of this review is to quantify the influence of cementation on liquefaction resistance and low strain shear modulus with the aim to provide semi-empirical correlations between penetration parameters and dynamic characteristics. Dynamic characteristics of cemented sands are compiled from the studies conducted by Rad and Clough (1982), Acar and El-Tahir (1986) and Saxena and Reddy (1988). The sands used in these studies are Monterey No. 0 and Monterey No. 0/30. Even though there is a slight difference in grain size distributions, the overall response of the two sands to dynamic loading can be considered as similar (Acar and El-Tahir, 1986). Hence, the results reported are assumed to be valid for Monterey No. 0/30 sand.
Different definitions of liquefaction are provided in the literature. The one proposed by Rad and Clough states, that for all practical purposes, failure occurs when a given value of double-amplitude axial strain is reached. They have considered 5% as double-amplitude axial strain. This selection is based upon the following arguments:

1. At this strain, it is reasonable enough to assume that all cementation bonds would be damaged.

2. Initial liquefaction for loose to medium dense uncedmented and cemented sands usually occurs when double-amplitude axial strain is equal to 4 to 6%.

3. At low relative densities (less than 50%), double amplitude axial strains higher than 5% usually result in unacceptable reductions in the applied vertical load.

For determining the influence of cementation or cement content on liquefaction resistance, dynamic triaxial tests were conducted by Rad and Clough (1982) and are briefly discussed in section 2.2.7.1.

Another property discussed in the dynamic behavior is the low strain dynamic shear modulus of the soil. The shear modulus is defined as the ratio of shear stress to the shear strain. The factors that affect shear modulus are: effective octahedral normal stress, void ratio, ambient stress history and vibration history, degree of saturation, octahedral shear stress, grain size characteristics, amplitude of strain, frequency of vibration, secondary effects due to time of loading and increment of load, soil structure and temperature (Acar and El-Tahir, 1986).

During the last two decades, a number of researchers have suggested empirical relationships for determination of maximum shear modulus of soils. The influence of cement content on dynamic shear modulus is investigated by Acar and El-Tahir (1986) and Saxena and Reddy (1988).

2.2.7.1 Large Strain Dynamic Stress-Deformation Behavior (Dynamic Triaxial Testing)

Undrained cyclic stress-controlled triaxial tests were conducted on uncemented and artificially cemented sand specimens. Figure 2.8 shows the test results plotted in terms of the stress ratio versus number of cycles necessary to achieve 5% double
amplitude axial strain for average relative density of 51%. Cementation increases resistance to liquefaction. This figure also reveals that the effect of cementation on the liquefaction resistance of sand decreases as the number of cycles to liquefaction increases.

2.2.7.2 Resonant Column Test (Low Strain Dynamic Behavior)

The maximum shear modulus of uncedmented sands can be evaluated by empirical correlations proposed by several investigators (Chung et al., 1984). The maximum shear modulus of sands is given as,

\[ G_{\text{max}} = \frac{S}{f(e)}(P_e)^{1-n}(\sigma_o^n) \]  

(2.1)

where \( S \) = stiffness coefficient; \( f(e) \) = a function reflecting the effect of void ratio, \( e \); \( \sigma_o \) = mean effective confining pressure; \( P_e \) = atmospheric pressure in the same units as \( G_{\text{max}} \) and \( \sigma_o \); and \( n = a \) constant. The maximum shear modulus is often normalized for the effect of density with,

\[ f(e) = 0.3 + 0.7e^2 \]  

(2.2)

The maximum shear modulus in the tests conducted by Acar and El-Tahir (1986) is expressed as,

\[ G_{\text{max}} = \frac{631}{0.3 + 0.7e^2}(P_e)^{0.57}(\sigma_o)^{0.43} \]  

(2.3)

Similarly, tests were also conducted on artificially cemented specimens at various relative densities and cement contents (1, 2 and 4 %). The results reported in this section are taken from the study conducted by Acar and Eltahir (1986). Solid cylindrical specimens of diameter of 36 mm and length of 80 mm were tested. Pluviation technique was used for low cementation and tamping or compaction was used for higher cement contents. Confining pressures of 35, 103 and 345 kPa were chosen to study the variability of shear modulus and damping.

The stiffness coefficient, \( S \), and \( n \) values of cemented specimens are shown in Table 2.8. It was found that low levels of cementation results in an increase in stiffness coefficient, while the exponent, \( n \), is within the variability of values presented for uncedmented specimens.
Figure 2.8: Stress Ratio Versus Number of Cycles: Cyclic Triaxial Tests on 1% Cemented Specimen of Relative Density 51% (Rad and Clough, 1982)
Table 2.8: Stiffness Coefficients and n Values of Cemented Sands

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>Cement Content</th>
<th>Stiffness coefficient</th>
<th>$n_{\text{mean}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>0</td>
<td>621</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>867</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1122</td>
<td>0.43</td>
</tr>
<tr>
<td>35</td>
<td>0</td>
<td>638</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>918</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1184</td>
<td>0.42</td>
</tr>
<tr>
<td>50</td>
<td>0</td>
<td>624</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>1028</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1318</td>
<td>0.42</td>
</tr>
<tr>
<td>75</td>
<td>0</td>
<td>658</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>1115</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1387</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Figure 2.9 depicts the variation of maximum shear modulus versus confining stress for cemented specimens prepared at relative density of 50 % (results at other relative densities are presented in Appendix A). The increase in dynamic shear modulus of artificially cemented specimens is attributed to the increase in stiffness coefficient.

The equation expressing the maximum dynamic shear modulus of sands is revised for the effect of cementation as

$$G_c = R \frac{S}{0.3 + 0.7e^2} (P_o)^{0.57} (\sigma_o)^{0.43}$$

(2.4)

where $G_c =$ maximum dynamic shear modulus including the effect of cementation; $R =$ stiffness ratio ($\frac{S_c}{S}$); $S_c =$ stiffness coefficient for uncemented specimens (631 for Monterey No. 0/30 sand). $R$ values are reported by Acar and El-Tahir (1986) for 1 and 2 % cementation.

### 2.3 Cone Penetration Testing in Sands

Several types of insitu testing equipment have been gaining wide popularity in geotechnical investigations for the past two decades. The cone penetration test (CPT) has become the most widely used insitu technique in the last few years. Even
Figure 2.9: The Variation of Maximum Shear Modulus Versus Confining Stress for Cemented Specimens Prepared at Relative Density of 50% (Acar and El-Tahir, 1986)
in the U.S., where standard penetration test is the conventional testing method, cone penetration testing has been gaining more and more popularity in the last decade.

2.3.1 Cone Penetrometer

The cone penetrometer test consists of advancing a cylindrical rod with a conical tip into the soil and measuring the forces required to push this rod. There are two resistances measured during the CPT, the tip resistance is the soil resistance to advance the cone tip and friction resistance is the friction between the soil and the sleeve of the cone. Friction ratio is defined as the ratio between the friction resistance and tip resistance and is expressed in percent. All these properties are used to identify and determine the soils and their properties. A typical cone penetrometer record is presented in Figure 2.10.

CPT parameters are used for the following assessments:

- Continuous soil stratification,
- Assessment of the undrained shear strength, stress history or over consolidation ratio (OCR), consolidation parameters and conductivity characteristics of cohesive soils,
- Assessment of relative density, drained strength parameters and compressibility characteristics of cohesionless soils,
- Evaluation of liquefaction potential of cohesionless soils,
- Determination of pile foundation capacities,
- Assessment of ground water pressures, if piezocone is used,
- Settlement calculations of footings in cohesionless soils.

There are quite a few interesting developments in the state of the art in cone penetration testing. Different sizes and shapes of cones are in use (Baligh, 1981; Tumay, et al., 1981). ASTM Standard D3441 recommends a standard cone with an apex angle of 60 degrees, tip area of 10 cm$^2$ and a sleeve area of 150 cm$^2$. Cones smaller than the standard cone are generally used in sites where shallow depths need
Figure 2.10: Schematic of a Electrical Cone (Juran and Tumay, 1989)
to be explored, in pavement and subgrade explorations and in finer classification of the strata. Larger cones are generally used when standard cone can not be used to penetrate harder strata.

The major breakthrough in cone penetration testing is the measurement of pore water pressures during penetration (PCPT). This was first introduced in the early 1970’s by advancing a separate pore pressure probe into the ground (Wissa et al., 1975; Torstenson, 1975). Similarly, in 1970-73, the Norwegian Institute of Technology, Trondheim, used a pore pressure probe of the same shape as standard cone, but only pore pressure measurements could be made and it was necessary to carry out a separate CPT in order to correlate the cone resistance and the pore pressure (Janbu and Senneset, 1974).

In the mid 70’s, the piezometric elements were incorporated into standard electric cone penetrometers in which pore pressures were measured along with cone penetration resistance in some cases and with sleeve friction and cone inclination in other cases (Acar, 1981; Baligh et al., 1981; De Ruiter, 1982). Subsequently PCPT which is also known as piezocone was used to measure the pore pressures at the cone tip and along the shaft (Tumay et al., 1981). The piezocone was also used to measure the dynamic pore pressures (Smits, 1981).

Currently new devices are developed and incorporated in this versatile piece of equipment to measure shear velocity, conductivity and even a fiber-optic eye for chemical characterization.

2.3.2 Calibration Chamber Testing

The calibration of geotechnical instruments like CPT for insitu tests is achieved by carrying out laboratory tests on homogeneous and reproducible soil samples, under accurately controlled states of stress and deformation. Such calibration is indispensable in development of insitu testing equipment and in the interpretation of the parameters obtained from testing (Bellotti et al., 1988).

Most of the laboratory study in the literature using CPT or PCPT was conducted either in large triaxial tests (Canou et al., 1988) or in calibration chambers (Holden, 1971 and 1977; Schmertmann, 1976; Tumay, 1976; Parkin, 1988; Been et al., 1987; Baldi et al., 1981). Empirical correlations in clays are more reliable since
insitu vane tests can be conducted adjacent to a CPT profile. This makes it possible to empirically correlate the undrained shear strength parameter (measured with vane shear tests) with the penetration parameters like tip resistance, friction resistance and friction ratio. However in granular soils, it is very difficult to correlate the strength parameters with the penetration parameters. There is no insitu testing method for measuring strength parameters directly in sands. Generally there are two approaches for interpreting the results in such materials; either correlating directly with measured quantities or by conducting tests in large calibration chambers. The latter approach has been extensively used by different investigators (Veismanis, 1974; Chapman, 1974; Holden, 1977; Bellotti et al., 1982; Villet and Mitchell, 1981; Eid, 1987). The chambers designed by several investigators are capable of housing large dimensioned soil samples (Ghionna and Jamiołkowski, 1991).

The use of calibration chamber to calibrate an insitu device has the following advantages:

1. Tests can be performed on uniform and highly reproducible sand specimens whose properties are well known. Hence, empirical or semi-empirical correlations can be made,

2. It is possible to monitor the stress and strain conditions around the sample,

3. Saturation can easily be achieved.

The main four phases of operation in a calibration chamber are:

1. Preparation of the specimen,

2. Saturation (if required),

3. One dimensional compression,

4. Conducting the requisite test.

A sample of the soil at a particular relative density is prepared in the calibration chamber and then consolidated to the desired stress levels. On this sample, the tests are conducted with the insitu testing apparatus and the parameters $q_c$ and $f_s$ are recorded along the vertical profile. Laboratory tests are then carried out on the
same soil at the same relative density to determine the engineering properties. The parameters measured in calibration chamber can be correlated directly or can first be correlated with relative density.

In general, two types of calibration chambers are used: chambers with rigid or flexible walls. A rigid wall calibration chamber imposes a boundary condition of zero lateral strain on the specimen. The flexible wall chamber allows the lateral movement. The design of a flexible wall calibration chamber permits an accurate control and measurement of the vertical and horizontal stresses and strains. Four types of boundary conditions can be simulated (Bellotti et al., 1985; Holden, 1977) in the tests:

- **BC1**: constant stresses on the boundaries,
- **BC2**: zero strains in horizontal and vertical directions,
- **BC3**: zero lateral strain,
- **BC4**: zero vertical strain.

Holden (1971) proposed that the field boundary conditions would lie somewhere between constant stress condition and zero lateral strain conditions i.e. BC1 and BC3. This statement is valid for lower diameter ratios. For larger diameter ratios, both boundary conditions should give identical results.

The factors that affect the test results in calibration chamber are described in the following sections.

### 2.3.2.1 Chamber Size and Boundary Condition Effects

Chamber size and boundary conditions affect the chamber test results considerably. Hence, an attempt is made in this section to review and discuss them. Chamber size implies that the size of the chamber specimen should be such that the results will be representative of those obtained in insitu field testing. The $q_c$ and $f_s$ charts produced on loose sands were found to have the characteristic shapes that reflect the density conditions. However the $q_c$ curve in a dense sand instead of reaching a plateau, was observed to increase with depth (Parkin and Lunne, 1982). This reflects the density and boundary condition effects. This effect, known as the ‘chamber
size effect', was pointed out by Parkin (1988) and Parkin and Lunne (1982). Tip resistances are mainly influenced by the diameter ratio $\frac{D}{d}$ where $D$ is the diameter of the specimen and $d$ is the diameter of the penetrometer tip. It was observed that the chamber size effect is not very important in loose sands but becomes very important when the relative density of the specimen increases. Particularly in dense to very dense sand specimens, the measured $q_c$ value increases as the diameter ratio decreases (Bellotti et al., 1985), possibly due to the restraint dilatancy effects.

It was observed by Bellotti et al. (1985) that the chamber size effect is subject to a complex interaction with boundary conditions imposed during the test. Bellotti et al., (1985), Parkin and Lunne (1982) and many others believe that the problems of the boundary conditions used in the chamber and the related chamber size effect require further intensive experimental and theoretical investigations. Figure 2.11 depicts the effect of chamber size and boundary conditions on the CPT for Hokksund sand (Parkin and Lunne, 1982). Despite the large diameter ratios, the $q_c$ is still a function of diameter ratio for dense sands. However, for loose sands, a diameter ratio of 20 seems to be sufficient and beyond this value, there is no significant increase in tip resistance. At a given diameter ratio, the test under boundary condition 3 predicts higher tip resistance than under boundary condition 1 (Been et al., 1988). In the case of zero lateral strain condition (BC3), higher stresses will exist at the chamber boundaries than in the field at an equal distance from cone, hence higher tip resistance is recorded. In constant stress boundary conditions (BC1), higher stresses will develop in the field at an equal distance from the cone, hence lower tip resistances are recorded in the chamber (Been et al., 1988).

### 2.3.2.2 Crushability

Crushability of the sands used in the chamber also influences the cone resistance. Several studies conducted by Baldi (1981) and Bellotti et al. (1988) showed that extensive grain crushing was observed in the sand during cone testing. The crushed aggregates may have different strength properties than the original sand tested and the influence of this crushing on the tip and specifically the sleeve friction resistance need to be investigated. Figure 2.12 presents the effect of crushability on cone resistance (Robertson and Campanella, 1984; Bellotti et al., 1991).
Figure 2.11: Chamber Size and Boundary Condition Effects (Parkin and Lunne, 1982)

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Figure 2.12: Influence of Crushing (Bellotti et al., 1991)
2.3.2.3 Fabric, Shape and Texture of Grains

The other factors that may influence the test results are fabric, shape and texture of grains in sands. It is extremely difficult to reproduce the field fabric in the chamber. However for sands, Been et al., (1988) believe that the fabric will have minor influence on the penetration resistance. They advocate that the cone measures the resistances in a remoulded zone in which fabric is destroyed. The size and shape of the sands used may also influence the tip and friction resistances significantly. The grain size to the cone size ratio may be one variable affecting the tip resistance and there is no information on this aspect.

Most of the above problems can be alleviated by constructing a large chamber or a flexible small chamber and using a small cone for the studies (Eid, 1989; Dario De lima, 1990). This means a large diameter ratio can be achieved and hence, chamber size and boundary condition effects on results can be minimized. However, other parameters, like ratio of cone diameter to grain size and crushability of sand used may also affect the results. In an attempt to evaluate the effect of these parameters, cone test results on different sands are collected. The next section covers different investigations and variables used in each study.

2.3.3 Synthesis of Experimental Data Obtained in Calibration Chamber Testing of Uncemented Sands

Cone test results conducted on several sands in different calibration chambers are collected and presented in this section.

The cone test results are collected from different research projects conducted by the following investigators:

1. Present Study - Monterey No. 0/30 Sand, USA
2. Eid (1987) - Monterey No. 0/30 Sand, USA
3. Baldi et al. (1981) - Medium Coarse Sands, SATAF 1 and SATAF 2, Italy
4. Villet and Mitchell (1981) - Monterey Sand Nos. 2, 30 and 60, USA
5. Harman (1976) - Ottawa No. 90 (5a) and Hilton Mine Tailings (5b), USA
Table 2.9: Characteristics of Tested Sands

<table>
<thead>
<tr>
<th>Sand No.</th>
<th>$G_s$</th>
<th>$\gamma_{\text{max}}$ ($kN/m^3$)</th>
<th>$\gamma_{\min}$ ($kN/m^3$)</th>
<th>$C_u$</th>
<th>$e_{\text{max}}$</th>
<th>$e_{\text{min}}$</th>
<th>Shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.65</td>
<td>16.90</td>
<td>14.51</td>
<td>1.6</td>
<td>0.803</td>
<td>0.563</td>
<td>SR-R</td>
</tr>
<tr>
<td>2</td>
<td>2.65</td>
<td>16.92</td>
<td>14.56</td>
<td>1.58</td>
<td>0.811</td>
<td>0.566</td>
<td>SR-R</td>
</tr>
<tr>
<td>3</td>
<td>NA</td>
<td>16.66</td>
<td>14.12</td>
<td>1.625</td>
<td>NA</td>
<td>NA</td>
<td>SA-A</td>
</tr>
<tr>
<td>4</td>
<td>NA</td>
<td>13.9</td>
<td>16.9</td>
<td>1.48</td>
<td>NA</td>
<td>NA</td>
<td>SR-R</td>
</tr>
<tr>
<td>5 a.</td>
<td>2.65</td>
<td>NA</td>
<td>1.85</td>
<td>0.789</td>
<td>0.486</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>5 b.</td>
<td>3.02</td>
<td>NA</td>
<td>2.0</td>
<td>1.05</td>
<td>0.620</td>
<td>A</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>2.65</td>
<td>NA</td>
<td>1.5</td>
<td>0.977</td>
<td>0.605</td>
<td>SA</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>3.02</td>
<td>14.2</td>
<td>17.0</td>
<td>1.3</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>8</td>
<td>NA</td>
<td>NA</td>
<td>2.75</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

Note: SR - Sub Rounded, R - Rounded, SA - Sub Angular, A - Angular

6. Fioravante et al. (1992) - Toyoura Sand, USA

7. Lhuer (1976) - Edgar and Reid-Bedford, USA

8. Nutt and Houlsby (1992) - Corbonate Sand, UK

The physical properties of these sands are presented in Table 2.9. The cone test results like tip resistance, friction resistance, friction ratio along with the vertical stress and relative density are reported in Table A.1 to A.8 in Appendix A. Other details like diameter ratio and the boundary condition in which the test was conducted are also reported in the same table. These results are used to study and analyze the effect of different variables affecting the calibration chamber testing. The important point worth mentioning here is all the test results may have been influenced by one or more than one variable. Hence, while analyzing each, data that is obtained under identical conditions are used. For example, data obtained from the sands of similar compressibility, size and shape and specimens tested under identical boundary conditions are used to investigate the influence of chamber size effects.

2.4 Cone Penetration Testing in Cemented Sand

Very few studies have been reported in the literature covering cone penetration testing in cemented sands (Rad and Tumay, 1986; Akili and Nabili, 1988). These
studies aimed at providing preliminary assessment of the influence of cementation on cone resistance parameters. Both studies are conducted in rigid chambers at the diameter ratio of 15 and the results are valid for very low confining pressures (less than 5 kPa).

Rad and Tumay (1986) presented tip resistance and sleeve friction values measured in experimental modeling of penetration in cemented sands. Artificially cemented Monterey No. 0 sand was employed in this study. Pluviation method was used to prepare specimens in a PVC mold. The diameter and length of the specimens were 30 and 45 cm respectively. Specimens were cured for 7 days. Testing was conducted with a 2.0 cm diameter cone at 2 cm\(^2\) sec\(^{-1}\) speed. The tests were conducted in the middle of the specimen in order to eliminate the chamber size effects. Figure 2.13 shows the penetration profile along the depth. It is observed that the tip and friction resistance increases with the cement content and relative density. The penetration values at 25 cm depth were reported in order to reduce the rigid bottom effects. The tip resistance and friction resistance values are presented in Table 2.10.

Table 2.10: Tip and Friction Resistances Reported for Cemented Sands

<table>
<thead>
<tr>
<th>Reference</th>
<th>Relative Density</th>
<th>Cement Content</th>
<th>Tip (\frac{MN}{m^2})</th>
<th>Friction (\frac{kN}{m^2})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rad and Tumay</td>
<td>20</td>
<td>1</td>
<td>0.48</td>
<td>15.0</td>
</tr>
<tr>
<td>(1986)</td>
<td>47</td>
<td>1</td>
<td>1.04</td>
<td>17.0</td>
</tr>
<tr>
<td></td>
<td>71</td>
<td>1</td>
<td>2.23</td>
<td>24.0</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>2</td>
<td>1.37</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>2</td>
<td>2.04</td>
<td>23.0</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>2</td>
<td>4.09</td>
<td>27.0</td>
</tr>
<tr>
<td>Akili and Nabil</td>
<td>43</td>
<td>0.2</td>
<td>4.2</td>
<td>NA</td>
</tr>
<tr>
<td>(1988)</td>
<td>90</td>
<td>0.2</td>
<td>8.9</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>43</td>
<td>1</td>
<td>6.7</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>1</td>
<td>10.7</td>
<td>NA</td>
</tr>
</tbody>
</table>

NA - Not Available

Akili and Nabil (1988) also investigated the influence of cementation on cone resistance parameters of beach sands and their results are also depicted in Table 2.10. The results obtained in both studies are influenced by the rigid boundaries and low diameter ratios (15). This argument is valid particularly in the case of dense sands.
Figure 2.13: Penetration Profiles in Laboratory Tests (Rad and Tumay, 1986)

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since dilation of the sand around the cone will be restricted (restrained dilatancy) by the presence of the rigid boundaries. This will, in turn, influence the cone results.

A flexible double walled calibration chamber was built at LSU (de Lima and Tumay, 1991) and is used in the present investigation. A miniature cone is used in the testing resulting in a diameter ratio of 42 (de Lima and Tumay, 1991). This diameter ratio reduces the chamber size and boundary condition effects significantly. The results of this study are analyzed along with the above reported test results.

2.5 Cone Penetration Testing Analysis

The current state-of-the-art regarding the analysis and application of cone penetration test results depend largely on semi-empirical and empirical correlations. These correlations are based on the following approaches: bearing capacity theories, simulation of the penetration mechanism by the cavity expansion theories and the strain path approach. Numerical simulations of the problem have been attempted by the finite difference and finite element methods. Theoretical studies involving in friction resistance predictions is also presented in section 2.5.6. Existing empirical and semi-empirical method in sands are explained in the next section. The final section covers state parameter interpretation as suggested by Been et al. (1986).

2.5.1 Bearing Capacity Theories

Many investigators have analyzed cone penetration as a bearing capacity problem (Meyerhof, 1963; Durgunoglu and Mitchell, 1973; Janbu and Senneset, 1974; Baligh, 1975). These theories assume different failure mechanisms which are then used to calculate ultimate bearing capacities using limit equilibrium approach. When penetration is treated with the conventional bearing capacity theories, soil is often assumed to behave as a rigid-perfectly plastic material. Therefore, the strains and compressibility of the material are neglected. These theories assume different failure surfaces. The cone penetration is analyzed as an axisymmetric problem and the solutions are developed under plane strain conditions using limit equilibrium approach. For shapes other than the idealized problems, the solutions are modified by applying empirical shape factors.
Several studies (Durgunoglu and Mitchell, 1973; Janbu and Senneset, 1974; Baldi et al., 1981; Villet and Mitchell, 1981) have successfully used these bearing capacity theories in determining the measured resistance parameters. Two theories which are used in the present research are described briefly here.

Durgunoglu and Mitchell [D & M] (1973) modified the Terzaghi (1943) bearing capacity equation by considering the effect of symmetry, effect of foundation shape and the effect of roughness:

$$q_{uc} = c N_c \zeta_c + \gamma B N_{\gamma\phi} \zeta_{\gamma\phi}$$  \hspace{1cm} (2.5)

where $N_{\gamma\phi}$ is the bearing capacity factor for the friction-surcharge term and $\zeta_{\gamma\phi}$ is the corresponding shape factor. The bearing capacity factors depend on the soil friction angle $\phi$, base semi-apex angle $\alpha$, base roughness $\frac{d}{D}$ and relative depth of penetrometer base $\frac{D}{B}$. The shape factors are calculated using Brinch-Hansen (1961) parameters. Figure 2.14 presents the bearing capacity factors and the assumed failure mechanism.

Janbu and Senneset [J & S] (1974) assumed the failure surface shown in Figure 2.15. The stress field is also depicted in this figure. Plane strain conditions are assumed. This method assumes that the failure surface fans out to different planes of plastification, $\beta$, depending upon the dilational characteristics of the soil deposit. This solution is successfully used in estimating the long term bearing capacity for point bearing piles in both fine and coarse grained soils (Senneset et al., 1982).

The general bearing capacity as per this method is expressed as:

$$q_v + a = N_q (\sigma'_{vo} + a) + u_o - N_u \Delta u_b + \frac{1}{2} \gamma B N_{\gamma}$$  \hspace{1cm} (2.6)

where $q_v$ is vertical ultimate bearing capacity, $N_u$ is the bearing capacity factor for pore pressure, $u_o$ is initial pore pressure, $\Delta u_b$ is pore pressure at foundation base, $a$ is attraction $\left(\frac{a}{\tan\phi}\right)$, $\sigma'_{vo}$ is effective vertical stress, $N_q$ and $N_{\gamma}$ are bearing capacity factors and $B$ is width or diameter of footing or penetrometer.

This approach uses attraction $a$ and friction angle $\phi$ in the analysis. The attraction represents the maximum tensile strength intercept. J & S 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).
Figure 2.14: Failure Mechanism Assumed in Durgunoglu and Mitchell's Theory (Durgunoglu and Mitchell, 1973)
Figure 2.15: Failure Mechanism Assumed in Janbu and Senneset's Theory (Janbu and Senneset, 1974)
In the analysis, attraction parameter and the friction angle from triaxial testing are used for estimating the cone resistances. The bearing capacity parameters are taken from charts. A proper plastification angle $\beta$ value has to be chosen. The plastification angle can be either positive or negative and this sign depends on the compressibility of the material. For loose sands and normally consolidated clays, positive values of 0 to 30 degrees are adopted whereas for dense and over consolidated clays, the range is 0 to $-40^\circ$ or in some cases beyond 40. The negative value implies that the soil will dilate during penetration. Therefore, negative plastification angles are selected for dilating sands.

Villet and Mitchell (1981) and Baldi et al. (1981) demonstrated that Durgunoglu and Mitchell correlation gave reasonable agreement between the predicted and measured friction angles. Their results are presented in Figures 2.16 and 2.17.

Figure 2.18 compares the predicted and measured values of tip resistance in cemented sands (Acar, 1987; Puppala et al., 1993). The predicted values are obtained by using Durgunoglu and Mitchell (1973) and Janbu and Senneset (1974) theories. The measured values are taken from the study conducted by Rad and Tumay (1986). This figure depicts that the predictions correlate quite well with measured values. These theories assume rigid-plastic behavior for the medium. Cemented sands which are brittle in nature exhibit a behavior which is close to the rigid-plastic assumption. This may be the reason behind the good correlations obtained between theoretical and measured values. However, it is noted that these results can not yet be generalized to higher confining stresses.

### 2.5.2 Cavity Expansion Theories

Analyses by cavity expansion theories are generally used for interpretation of pressuremeter tests (Hughes, et al., 1977), studies involving installation of driven piles (Carter et al., 1986) and in the interpretation of cone penetrometer tests (Greeuw et al., 1988).

The penetration mechanism is idealized as that of a spherical or cylindrical cavity expanding in a semi-infinite medium. The advantage of such an assumption lies in uni-dimensional formulation of the problem which facilitates incorporation of plasticity models. Ladanyi (1969) used the cavity expansion theories in brittle rocks.
Figure 2.16: Comparison Between the Measured and Predicted Parameters (Baldi, 1981)

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Figure 2.17: Comparison Between the Measured and Predicted Parameters (Villet and Mitchell, 1981)
Figure 2.18: Comparison Between the Measured and Predicted Parameters (Acar, 1987; Puppala et al., 1993)
and also applied the approximate solutions for limit pressures in the spherical case to the bearing capacity problems. Theories proposed by Vesic (1972) and Baligh (1975) derive the cone resistance solutions in terms of soil compressibility characteristics. These theories are based on the work required to expand a cylindrical or spherical cavity around the cone tip as it is driven into the soil (Eid, 1987).

The limit pressure in the cavity expansion model is defined as the pressure in the cavity which corresponds to a continuous deformation without pressure increase. This limit pressure can be related to tip resistance in a simple way as suggested by Carter et al. (1986). Closed form solutions are presented for cohesionless soils (in the case of uncemented sands) and also cohesive frictional soils (cemented sands). Mohr-Coulomb yield criterion is adapted in the analysis. The solution for the pressure-expansion curve is obtained by considering small strains. This theory is also used in the present analysis.

The cavity is shown in the Figure 2.19. The plastic domain of the cavity is extended up to a radius R from the center. Beyond this domain, the soil is in elastic state. The final solution for limiting pressure is a function of strength properties and the dilation angle. Greeuw et al. (1988) used this approach and demonstrated that the theoretical predictions correlated fairly well with experimental tip resistance.

2.5.3 Strain Path Approach

A recent and third approach to the penetration in soft cohesive soils was proposed by Baligh and Lebadoux (1980), Tumay (1985) and Acar and Tumay (1986). This approach takes into consideration the steady nature of the problem and finds the strains and displacements around a cone penetrating an inviscid and incompressible fluid. The strain field obtained from this method would provide a first approximation to strains induced by cone penetration testing and/or pile driving in soft cohesive soils. The strain field could then be used as an input to recently improved large strain soil plasticity models to estimate the generation of pore pressures, effective stresses during penetration and dissipation of pore pressures when the driving or penetration is stopped.
Figure 2.19: Cavity Used in the Cavity Expansion
2.5.4 Numerical Methods

Another approach is adopting a large strain, elasto-plastic formulation for simulating the penetration mechanism using the finite element method. The formalism developed by Kiousis et al. (1988), is applied to the solution of the cone penetration problem in a soft cohesive soil. The plasticity models used in this specific study were Von Mises and the cap model by Dimaggio and Sandler (1971). This theory predicts the displacement-strain, stress and pore pressure fields around the cone.

2.5.5 Discussion

All of these methods have their advantages and disadvantages. The disadvantages of each method are:

- In the bearing capacity approach, the effects of factors like soil compressibility, excess pore water pressure, initial state of stress, progressive rupture, dependence of the internal friction angle on the mean effective normal stress and the effective stress path are neglected (Tumay, 1985). Bearing capacity theories consider the penetration of a rigid cone into compressible soil as a stress controlled problem even though it is a strain controlled problem.

- Cavity expansion theories have been used by many authors. Simulation of cone penetration using cavity expansion theory is complex because of the large strains around the cone tip and also due to the shape of the cavity which is neither a cylinder nor sphere.

- The approach proposed by Baligh and LeVadoux (1980) and Acar and Tumay (1986) may be valid for soft cohesive soils. The method assumes incompressibility in predicting the strain paths. Compressible soils like carbonate sands can not be interpreted using this approach.

- The disadvantages of finite element solutions are the cost of computation, difficulty in simulation of the problem particularly near the cone where interface elements and a knowledge of the behavior of such elements is needed. The constitutive models used for the interface elements strongly affect the obtained predictions. Kiousis et al. (1988) notes that a viscoplastic analysis will be a
more appropriate approach than the classical plasticity approach in the simulation of the penetration problem by the finite element method. This may be necessary in cohesive soils since strain rates in penetration are orders of magnitude higher than conventional geotechnical problems. However, the behavior of soils at such large strains and strain rates is not well-defined in the current state of the art.

All the above theories despite their limitations are often used in penetration analysis and in estimating tip resistance.

2.5.6 Friction Resistance (Sleeve Friction)

The predictive methods for estimating friction resistance assume that sleeve friction is only due to shear resistance. The following formula can be used in calculating friction resistance:

\[ f_s = S_s (\sigma'_w + a) \]  

in which \( f_s \) is friction resistance, \( S_s \) is \( |r| \tan \phi K \) and \( |r| \) is interface friction ratio defined as \( \frac{\tan \theta}{\tan \phi} \).

The strains in the vicinity of the sleeve are well beyond the strains corresponding to peak strength values (Acar and Tumay, 1986). Therefore, it is more appropriate to use residual values in estimating friction resistance. The roughness coefficient should be considered as a product of two factors: one corresponding to the mechanical roughness of pile/cone surface, and the other relating to relative vertical movement between the pile/cone surface and adjacent soil. The upper limit for this factor is 1.0 and the lower limit is around 0.55 in practice (Janbu, 1976; Acar et al., 1982). In the present case, \( |r| \) is taken as 0.65 (i.e. for steel to sand).

Figure 2.20 depicts the comparison between the measured values by Rad and Tumay (1986) and predicted friction resistances for different coefficients of earth pressure. The above described approach gives very low values compared to the measured resistances, if earth pressure coefficients at rest are used. However if passive earth pressure coefficients are used, the friction resistances come close to the measured values. The implication is that the dilation at the tip results in expansion loading on the shaft to increase the confinement on the sleeve. The extent to which
this mobilization of dilation occurs will depend upon the type of soil, the density of the deposit, the fabric and the depth of penetration. This implies that the friction resistance mainly depends on the dilational characteristic of sands.

2.5.7 Empirical Methods

Empirical methods are also used in estimating the strength and deformation properties. Schmertmann (1976) proposed an indirect method for estimating the friction angle. This method is based on the calibration chamber results and is shown in Figure 2.21. Figure 2.21 is used to estimate the relative density, once the cone resistance, \( q_c \) and vertical effective stress, \( \sigma'_v \) are known. The second figure (2.21) is used to determine the friction angle with the known relative density.

Other empirical methods are used to estimate the constrained modulus, elastic modulus and shear modulus (Baldi et al., 1981). In case of constrained modulus, most of these empirical correlations take the following form:

\[
M = \alpha q_c
\]  

(2.8)

The \( \alpha \) value ranges from 3 to 11 (Veismanis, 1974; Parkin et al., 1982; Acar, 1981). The more recent recommendation varies between 1.5 and 4 (Lunne, 1991). Other results (Robertson and Campanella, 1984 and Jamiolkowski et al., 1988) are shown in Figures 2.22 and 2.23. Relative density need to be determined prior to the use of these figures.

Lunne and Christopherson (1983) recommended the following simple approach for the estimation of constrained modulus in normally consolidated sands.

\[
M_o = \begin{cases} 
4q_c & \text{for } q_c < 10 \text{ MPa} \\
(2q_c + 20) & \text{for } 10 \text{ MPa} < q_c < 50 \text{ MPa} \\
120 \text{ MPa} & \text{for } q_c > 50 \text{ MPa}
\end{cases}
\]  

(2.9)

(2.10)

(2.11)

For Young’s modulus, the following empirical formula is generally used.

\[
E = \beta q_c
\]  

(2.12)

The \( \beta \)-value generally lies between 1 to 2. The small strain shear modulus can be evaluated either by using Robertson and Campanella’s (1984) chart or by using Baldi et al. (1981) chart. Both the charts are shown in the Figure 2.24.
Figure 2.20: Comparison Between Theoretical and Measured Friction Resistances

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Figure 2.21: Schmertmann's Method for Estimating the Friction Angle
Figure 2.22: Evaluation of Constrained Modulus (Robertson and Campanella, 1984)
Figure 2.23: Evaluation of Constrained Modulus (Jamiolkowski et al., 1988)
Figure 2.24: Evaluation of Small Strain Shear Modulus (Robertson and Campanella, 1984; Baldi et al., 1981)
2.5.8 State Parameter Interpretation

The state parameter is a quantitative measure of the state of a sand that combines the effect of void ratio and effective stress in a unique way. The state parameter, $\psi$, is defined as the void ratio difference between the current void ratio and steady state void ratio at the same stress level as depicted in Figure 2.25 (Been et al., 1986). Steady state line represents a condition of zero dilation during shear and this state reflects a combination of many physical properties of sands including compressibility, grain size shape and distribution, limiting void ratio, mineralogy and friction angle at constant volume, $\phi_{cv}$.

Undrained triaxial tests with pore pressure measurements are needed to define the steady state line for each sand. Once steady state line is known, the steady state parameter of a test can be determined by using the test density and the stress level under which the test is conducted. The cone test data is related to stress level and density state as state parameter. Hence, they both can be correlated. Been et al., (1986) plotted $q_c - P$ versus $P'$ for different state parameter values, where $P$ and $P'$ are total and effective octahedral stresses. Later, the same results are adjusted by applying a chamber diameter correction factor and they are replotted. Linear contours of equal $\psi$ are noticed. Hence, the use of a normalized cone tip resistance in the form of $\frac{q_c - P}{P'}$ is considered appropriate for the standardized calibration chamber data. Figure 2.26 presents normalized tip resistance versus $\psi$ for Monterey sand data as presented by Been et al., 1986. The well defined correlation is evident and it suggests the relationship is good even when the tests are conducted under saturated and dry conditions, normal and over consolidated states and at different $K_o$ values.

By using this figure, $\psi$ value can be estimated if cone test results are known. Once $\psi$ is known, the friction angle and other parameters can be estimated from Figure 2.27.

The difficulties associated in determining or measuring the void ratio accurately using the CPT data can be overcome by using this approach. However, further work is needed to clarify the influence of fabric, insitu stress strain fields, chamber boundary conditions and cementation on the results. This approach is also adopted in preparing empirical relationships. Hence, several undrained tests are conducted on cemented specimens to define steady state lines.
Figure 2.25: Definition of Steady State Parameter (Been et al., 1986)
Figure 2.26: Normalized Cone Resistance Versus State Parameter for Monterey No 0 Sand (Been et al., 1986)
Figure 2.27: Properties of Sands Versus State Parameter (Been et al., 1986)
2.6 Summary

Cementation and their causes are explained. Cementation phenomena in different soils are also described. Static and dynamic properties of artificially cemented sands are presented. This is followed by a section in which cone penetration testing in calibration chambers is described. Limited data available on cemented sands are presented. Various analysis tools used in the interpretation of cone penetration test results are discussed.
Chapter 3

METHODOLOGY

3.1 Introduction

This study is conducted in three phases. The first phase is the experimental model and it deals with performing cone penetration tests in a calibration chamber on both cemented and uncemented sands. Cementation levels of 0, 1 and 2 % are investigated since these represent lower cementation levels in natural deposits. Specimens are prepared at three different relative densities (45-55, 65-75 and above 85 %) and consolidated under three different vertical stresses of 100, 200 and 300 kPa. Density levels represent the possible field densities expected and the effective confining pressures correspond to depths of 5, 10 and 15 m of unsaturated or dry sands or 10, 20 and 30 m of saturated sands. All the above variables will cover weakly cemented sands and up to depths of 30 m. Several undrained triaxial and unconfined compression tests are also conducted on both cemented and uncemented specimens. These results are used in the proposed prediction scheme developed in the second phase.

The second phase is theoretical assessment. Available theoretical models are used to predict the parameters measured in the experimental model. Using the predictions of both experimental and theoretical models, a methodology is formalized to identify cemented sand deposits and evaluate their engineering characteristics. The last phase is evaluation of the experimental results in light of the above correlations. Semi-empirical predictive methods are developed in this section.

This chapter covers the first phase, the experimental model. The information pertaining to the equipment used, specimen preparation method and testing procedures are discussed. The experimental work is composed of undrained triaxial tests, unconfined compression tests and calibration chamber tests. Table 3.1 gives the summary of the calibration chamber tests. Table 3.2 presents the list of undrained triaxial tests conducted. Separate drained triaxial tests are conducted by Arslan (1993). Table 3.3 provides a summary of these tests.
Table 3.1: Number of Tests (Calibration Chamber)

<table>
<thead>
<tr>
<th>B.C</th>
<th>C.C.</th>
<th>SIGC</th>
<th>RELATIVE DENSITY</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(kPa)</td>
<td>45-55</td>
<td>65-75</td>
</tr>
<tr>
<td>BC1</td>
<td>0</td>
<td>100</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td></td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>BC2</td>
<td>0</td>
<td>200</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td></td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td></td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>BC3</td>
<td>0.1</td>
<td>300</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td></td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td></td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

BC-Boundary Condition; C.C.-Cement Content

Table 3.2: Number of Undrained Triaxial Tests

<table>
<thead>
<tr>
<th>CEMENT CONTENT</th>
<th>RELATIVE DENSITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>45 - 55</td>
</tr>
<tr>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Total</td>
<td>9</td>
</tr>
</tbody>
</table>

3.2 Experimental Model

The calibration chamber is the testing equipment used in experimental modeling of cone penetration. Pluviation setup is associated with specimen preparation procedure for the calibration chamber. Other equipment used in calibration chamber testing are the auxiliary system including the control panel, hydraulic system and supporting equipment like cranes and the data acquisition and monitoring system. Other testing devices used in experimental testing are the triaxial system and unconfined compression testing setup.
Table 3.3: Drained Triaxial Tests (Arslan, 1993)

<table>
<thead>
<tr>
<th>CEMENT CONTENT</th>
<th>RELATIVE DENSITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>45 - 55</td>
</tr>
<tr>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Total</td>
<td>9</td>
</tr>
</tbody>
</table>

3.2.1 Equipment

3.2.1.1 Pluviation Setup

Specimen preparation is an important aspect of testing. The difficulties in obtaining undisturbed natural sand specimens prompt the need to use a technique that simulates the depositional process in natural deposits. The laboratory method, known as ‘pluviation’ or ‘raining’ not only provides homogeneous specimens at the desired relative density but also simulates a soil fabric presumably most similar to the one found in natural deposits formed by sedimentation (Bellotti et al., 1991).

Figure 3.1 shows a schematic of the pluviation setup used in the present study. Basically, this is a three chamber setup placed one above the other. All the chambers are made of Poly Vinyl Chloride sections. The top chamber stores the sand that needs to be pluviated. The middle chamber gives sufficient height of fall for the sand leaving the top chamber. The bottom chamber is the one in which the sand is deposited and the specimen is formed. The bottom chamber is called the specimen chamber. The bottom chamber is a diametrically split chamber held together by a metal frame. This chamber is placed on a wooden trolley with four wheels on each side. It holds the bottom plate through four equally spaced, stainless steel 6 mm (1/4 in.) bolts.

Since the study involves cemented sands, the bottom chamber needs to be transferred first to the humidity room and then to the calibration chamber. This transferring requires that the top two chambers be permanent fixtures and the specimen chamber be the only removable member of the setup. Hence, the top two chambers are fixed on a table. The size of the table is 1.8 m × 1.5 m × 1.2 m. This table also provides enough room to place the sand in the top chamber.
Plate 3.1: Schematic of the Pluviation Setup
An aluminum shutter is placed between the top and middle chamber. This circular shutter mechanism consists of a set of two plates with an identical hole pattern. Initially, the two plates are placed one on top of each other and are bolted at the center. Handles are welded to this shutter system to facilitate the rotation in alignment of the holes and initiation of the pluviation. The holes in the two plates will align when the handles are brought together and will prevent pluviation in other positions. When the holes are aligned, the sand will be released from the top container, pluviating down to the bottom chamber.

The diffuser consists of a set of two sieves positioned at 45 degrees to each other and is placed in the specimen chamber. Four hooks are placed on top of the diffuser at equal distances. Strong threads or metal cables are connected to these hooks. The other end of the threads are fastened to the top cap which is placed on top of the sand in the top chamber. During pluviation, when sand starts pouring in, the top cap moves downward. This downward movement of the top cap raises the diffuser and the height of fall of sand from the diffuser is maintained constant throughout specimen preparation. This height of fall influences the density of the specimen. It is necessary to keep this height constant in pluviation in order to obtain a homogeneous specimen.

3.2.1.2 Saturation Setup

It is essential to allow water access into the specimens both to saturate the specimens and also to allow the pozzalonic reaction initiate the necessary cementation. A system is developed to saturate the cemented specimens. The setup consisted of a 50 gallon tank and a carbon dioxide cylinder. The tubing connections, the water and $CO_2$ tank and the specimen chamber can be seen in Plate 3.1. $CO_2$ is used in specimen preparation since the $CO_2$ dissolve in water much more easily than air. The system is connected to the tubing coming from the bottom plate. Pressures of very low range (less than 10 kPa) are used since pressures higher than the overburden pressure of the sample may disrupt particle positions in the specimen and may result in inhomogeneities across the specimen. The tank is placed at an elevation
of 0.6 m above the bottom plate in an attempt to have low water pressures in the
tubing. Other details of the saturation process are presented in the section describing
cemented specimen preparation.

3.2.1.3 The LSU Calibration Chamber Facility

The Louisiana State University calibration chamber system (CALCHAS) was
developed by Dr. Dario de Lima as a part of his dissertation (de Lima and Tumay,
1992). This chamber permits testing cones of different sizes and shapes under con­trolled boundary conditions. The chamber is 1.78 m high and 0.64 m in diameter. The assembly (Figure 3.2) is divided into two sections namely the piston cell and
the chamber cell unit. The chamber cell is a double walled ‘flexible’ cylinder made
of steel. This unit rests on a bottom plate of 0.64 m in diameter and 38.1 mm in
thickness. The piston pushes the bottom plate upwards thereby applying a vertical
stress on the specimen.

The piston cell bottom plate carries a bearing shaft that houses the piston and
allows vertical movement during testing. The annular spaces in the piston cell as­
sembly are filled with deaired water. During testing, this deaired water is pressurized
and the inner piston cell moves upwards. The required vertical stress is thus gener­
at ed as a result of this upward thrust. The sample cell is a double-wall flexible cell
which can house a sample of 0.53 m in diameter and 0.79 m in height. The chamber
walls are made of stainless steel and the thickness of the walls is 6.35 mm. The
internal diameter of the outer and inner walls are 0.58 m and 0.56 m, respectively.

The sample top and bottom plates are made of 6061 T-6 aluminum and are of
0.53 m in diameter. The sample bottom plate rests on the piston cell unit. The
sample top plate is bolted to the chamber top plate which is 0.64 m in diameter and
38.1 mm in height. The chamber top plate, sample cell inner and outer walls and
the piston cell ring are kept together via twelve stainless steel rods. These rods are
tightened up to 65 N m. torque to ensure that the whole assembly does not have
any leaks during the testing.

The annular space between the sample and inner wall and between the inner and
outer walls are hereafter named as inner and outer cells. During testing, these are
Plate 3.1: Photograph of the Saturation Setup
Figure 3.2: Schematic of the Calibration Chamber (de Lima and Tumay, 1992)

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filled with deaired water via two water lines connected to the top plate. Horizontal stress is applied to the specimen by pressurizing these inner and outer cells.

### 3.2.1.4 Saturation and Vacuum Connections

The saturation and vacuum lines are made of PVC tubing and are 1.25 cm in diameter. The tubing can work under pressures as high as 800 kPa. The spirally placed saturation line is fixed to the bottom plate. This tubing has several holes and is wrapped with a screening cloth allowing water supply into the sample while preventing the sand to clog the holes. One end of the spiral is closed and the other end goes through the plate with a one-way valve at the end. The valve is connected to the saturation setup during the saturation phase.

The vacuum tubing is fixed on the top plate with similar arrangements as the saturation tubing. The outer end of this tubing is connected to the female end of a quick connector. This end is connected to the vacuum pump. Once the vacuum is applied for a sufficient time, the tubing is disconnected from the pump. The suction pressure inside the chamber is still intact due to the quick connection. The suction allows placement of inner and outer cells over the specimen and avoids any collapse in the specimen.

### 3.2.1.5 Vacuum Pump

The vacuum pump used in this study is manufactured by Welch Company. The pump is used in two operations: during specimen preparation and during placement of inner and outer cells over the specimen. This pump can generate suctions of 60 to 100 kPa.

### 3.2.1.6 The Miniature Quasi-Static Cone Penetrometer

The miniature quasi-static cone penetrometer (MQSC) is a 1.27 cm$^2$ cross-sectional area subtraction type Fugro-McClelland cone penetrometer with a 6.3 cm long friction sleeve and an apex angle of 60° (Plate 3.2) (de Lima and Turnay, 1992). The MQSC push rod is 9.53 mm in diameter and 1.82 m in length. This cone when used in CALCHIAS, a diameter ratio of 42 is obtained. Existing data indicate that
boundary effects are significantly reduced at this diameter ratio even in dense sands. Details of the calibration factors used for this cone are given in Table 3.3.

![Table 3.4: Calibration Factors of the Cone](image)

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

3.2.1.7 The Auxiliary equipment

Two movable cranes are built as part of the supporting unit for the calibration chamber operations. One crane is electrically operated and the other one is manually operated. The capacities of these cranes are 2 and 1 tons respectively. This system is used both in weighing and lifting the sample into the chamber and also in the placement of the inner and outer cells.

3.2.1.8 Control Panel

Controls which regulate operation of the chamber are grouped on a vertical wooden panel of 1.22 m x 1.96 m. Copper tubing was used for all control lines to reduce volume changes and hence the compressibility in the system. The water pressure lines are connected to the top plate through the quick connectors. Plate 3.3 shows a photograph of the controls on the panel board.

The main units in the panel of control are: pressure regulators, electro-pneumatic transducers, pressure transducers, pressure gauges. The back pressure regulators are used to provide protection against over pressure in downstream portion of pneumatic system. However, in the present system, these act as relief valves and keep the water pressure constant in the inner cell and piston cell during testing. In cases, where vertical and horizontal pressures are kept constant during the penetration phase, these regulators have an important role. Any rise in pressures due to penetration testing will be relieved by these regulators. The control panel has two of these units which operate within the range of 14 to 1070 kPa (2 to 150 psi).

The electro-pneumatic transducer converts an electric to a linear pneumatic signal. The chamber has four of these transducers in the panel, two of them for the
Plate 3.2: Photograph of the Miniature Cone (de Lima, 1990)
Plate 3.3: Photograph of the Controls on the Panel Board
pressure range of 40 to 215 kPa (5.6 to 30 psi) and the other two for the range of 21.5 to 860 kPa (3 to 120 psi). A DC signal of 0 to 10 V is generated. Two of the transducers are used in the piston cell operation for applying the vertical stress to the sample. The other two are used for the pressure compensation between the sample inner and outer cells during $K_o$ consolidation and penetration phase.

The chamber also houses five pressure transducers in the range of 0 to 215 kPa (0 to 30 psi) and 0 to 714 kPa (0 to 100 psi). Two of them are connected to the water line related to piston cell, two others are connected to the water line directed to inner cell and one in the range of 0 to 215 kPa (0 to 30 psi) is connected to the outer cell water line. Five marsh process gauges are used in the panel. They work in the range of 0 to 714 kPa (0 to 100 psi) with an accuracy of ±0.5 %. For applying the pressures (vertical and horizontal pressures), the air water interface system is used. Two PVC cylinders and their caps glued at high pressures are used to apply the above pressures. The cylinders are filled with water and air in a 90 % to 10 % proportion with an oil interface.

3.2.1.9 Hydraulic System

Hydraulics and the push jack system allow penetration of the cone in a single stroke. The maximum stroke is 0.79 m; however, in this study, the cone is pushed in two strokes since it was found that a single stroke sometimes buckles the cone in dense specimens. Even when sufficient grip length is provided, cone showed buckling in the 80 % relative density specimen consolidated under 300 kPa. In view of the cost, importance and the non availability of such an equipment, tests are performed by pushing the cone in either two or three strokes.

An analog to digital converter depth decoding system is developed and incorporated in this system. The depth decoder is composed of a metal disk, a light emitting diode and an optical sensor. Holes are drilled at equal distances on the circumference of the disk. As the cone penetrates the specimen, a cable connected from the drill rod to the shaft of the disk mechanically turns the disk. The distance between the holes on the disk represents a penetration depth of 2 cm. The light emitting diode and the optical sensor are placed on either side of the disk. When light emitted by the diode passes a hole, the optical sensor senses the light and generates a pulse to
the control unit. This triggers the multiplexer to switch on the channels for analog to digital conversion. This process continues until penetration motion is stopped.

3.2.1.10 Data Acquisition and Monitoring System

The hardware used in the data acquisition process consists of Zenith PC microcomputer with 640 K RAM, 40 MB hard drive, EGA monitor, a data translation DT 2801 A/D board and a HP7475 plotter. The flow chart for data acquisition and monitoring system is depicted in Figure 3.3.

The software for specimen consolidation and testing at different boundary conditions is developed in Turbo Pascal version 4.0 environment by Borland International, and the Halo'88 graphics library by Media Cybernetics (de Lima, 1990). The data translation board is used for analog to digital conversions (A/D), digital to analog conversions (D/A) and for performing digital I/O transfers. This board has sixteen 12 bit A/D channels and two D/A channels.

Six A/D channels receive the data in volts from three transducers (piston cell, inner and outer cell), one LVDT and tip and friction load cells in the cone. Two D/A channels send the data in volts to two electro-pneumatic transducers. The depth sensor sends signals to digital I/O transfer. The calibration of the electro-pneumatic transducers is done using the commercial software named Labtech Notebook.

The data acquisition software includes five computer programs, one for consolidation phase and the rest for the penetration phase. The names of these programs are: CHAMBK0.EXE for consolidation and CHAMBC1.EXE, CHAMBC2.EXE, CHAMBC3.EXE, CHAMBC4.EXE for the penetration phase. These programs originally written by de Lima (1990) are modified by the author to suit the present testing. The details of the modifications associated with the boundary conditions are discussed in the section pertaining to the penetration.

The data are acquired employing the following procedure. The initial readings in voltages are taken during the consolidation stage. The data are read and stored in the computer at every 2 cm depth interval during penetration. The initial readings are then subtracted to calculate the measured values. Finally, the data is sent to a plotter for graphing.
Figure 3.3: Flow Chart for Data Acquisition and Monitoring System
3.2.1.11 The Triaxial System

The triaxial system (manufactured by Wykeham Farrance, UK) at Louisiana Transportation Research Center (LTRC) geotechnical laboratories is used in the undrained triaxial tests. Samples of 7 cm (2.8 in.) in diameter and 15.2 to 16.5 cm (6 to 6.5 in.) in height are used. Cell pressures of 428 kPa (60 psi) can be applied to the specimens. Backpressure saturation is used. Backpressures of order 428 kPa (60 psi) can be applied.

This equipment is connected to a data logger (Autotech) and a computer. The data logger stores the readings from the load cell (deviatoric stress, channel 7), strain transducer (axial strain, channel 8) and a pore pressure transducer (excess pore pressure, channel 9). The frequency of reading the data can be varied. A five second interval is chosen for the present testing. The tests are conducted under strain controlled conditions.

A commercial software CLISP is used to analyze the data for each test. This software takes the input of sample information, sample dimensions, details about back pressures and cell pressure. The final output from the program consists of axial strain, axial load, pore water pressure, excess pore pressure and deviatoric stress.

Drained test are conducted in a triaxial apparatus designed and fabricated by Trautwein Soil Testing, USA. An ELE volumetric transducer is connected to this setup to record volume changes in the sample during shear. Cell and back pressures are applied through a control panel. During the test, LVDT, load cell and volume change transducer readings are recorded.

3.2.1.12 Unconfined Compression Tests

This machine is manufactured by ELE, USA. The unconfined compression setup allows testing of specimens of different sizes up to 5 cm in diameter. Testing can be done at different speeds varying from 2 to 9 cm/sec. Dial gauge and load cell readings are taken at constant intervals. The height and diameter of the specimen are recorded and are used in the calculation of strains and unconfined pressures.
3.2.2 Procedure - Uncemented Specimens

The uncemented specimen preparation and testing procedures are described in the following sections.

3.2.2.1 Triaxial Tests

Laboratory tests consisting of strain controlled consolidated undrained triaxial tests were conducted using Monterey No. 0/30 sand. This sand is a commercial beach washed sand. The grain size distribution of this sand is shown in Figure 3.4. The index properties are presented in Table 3.4. The main objective of this testing was to determine the strength-deformation characteristics, and critical state behavior of this sand and assess whether there were significant differences in the results reported for this sand by various investigators.

<table>
<thead>
<tr>
<th>PROPERTIES</th>
<th>RAD (1982)</th>
<th>EL-TAHIR (1985)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( G_s )</td>
<td>2.65</td>
<td>2.65</td>
</tr>
<tr>
<td>( \gamma_{d,\text{max}} ) (kN/m(^3))</td>
<td>16.65</td>
<td>16.65</td>
</tr>
<tr>
<td>( \gamma_{d,\text{min}} ) (kN/m(^3))</td>
<td>14.04</td>
<td>14.04</td>
</tr>
<tr>
<td>( e_{\text{max}} )</td>
<td>0.85</td>
<td>0.85</td>
</tr>
<tr>
<td>( e_{\text{min}} )</td>
<td>0.56</td>
<td>0.56</td>
</tr>
<tr>
<td>( C_u )</td>
<td>1.43</td>
<td>1.50</td>
</tr>
<tr>
<td>( C_r )</td>
<td>NA</td>
<td>0.95</td>
</tr>
<tr>
<td>( d_{50} ) (mm)</td>
<td>0.45</td>
<td>0.43</td>
</tr>
</tbody>
</table>

i) Specimen Preparation

The following steps describe the specimen preparation adopted in the present study.

1. Dry Monterey sand No. 0/30 needed to achieve the desired relative density was weighed and placed in equal proportions in six beakers. The height deposited during pluviation of the specimen in each beaker would provide an idea of the homogeneity.

2. The pore pressure and back pressure lines in the triaxial system are saturated with deaired water. Any visible air bubbles in the water inside the lines are flushed out.
Figure 3.4: Grain Size Distribution of Monterey Sand

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3. A porous disc is placed over the bottom plate of the triaxial cell. This bottom plate includes a drainage valve which is used in vacuum application.

4. A membrane is placed over the disc and the bottom plate. An O-ring is placed around the membrane on the bottom plate.

5. A split mold is placed over the membrane. The membrane is pulled up and is placed over the mold.

6. Sand from each beaker is poured into a funnel which is directly placed over the mold. The height of fall, defined here as the distance between the bottom of the funnel to the bottom of the soil layer deposited in the mold, is kept constant throughout the testing in an attempt to assure homogeneity throughout the specimen.

7. When all the sand in the beaker is poured, the mold is tapped along the sides so that the sand settles across the top uniformly.

8. The height of the specimen in the mold is measured with a vernier caliper and relative density is calculated. The height of fall and the amount of tapping necessary is varied to achieve different densities.

9. A porous stone is then placed over the top of the sand. The top cap with back pressure line is placed over the porous stone.

10. The membrane is pulled up and the O-rings are placed around the top cap.

11. A vacuum of $30 \text{kPa}$ to $40 \text{kPa}$ is applied to the specimen through the drainage valve in order to supply the necessary confinement to hold the specimen together when the mold is removed.

12. The outer cell is placed, bolted, tightened and filled with deaired water and a small amount of cell pressure of order $35 \text{kPa}$ is applied.

13. The back pressure valve is opened to allow flow of water in the specimen. It is left open until water flows through the drainage valve to the vacuum pump.
14. The back pressure valve is closed and a cell pressure of 80 kPa is applied. At this stage, the drainage valve is closed and vacuum is disconnected.

15. Back pressure of 50 kPa is applied and this value is always kept lower than the cell pressure.

16. The pore pressure measured was always higher than the applied back pressure. This excess pressure is due to the confining pressure applied in the cell.

17. The drainage valve is opened slightly to release some more air from the specimen and then closed.

18. This pore pressure will stabilize back to the applied back pressure after a certain period of time. The excess pore pressure due to the cell pressure was dissipated due to opening of the drainage valve. Hence, the pressure inside the specimen should be same as the applied back pressure. The cell pressure and back pressures are raised incrementally. The above steps are repeated until either the desired back pressure is applied or the full saturation condition is achieved.

19. Saturation is measured by calculating the Skempton’s B parameter. This parameter is defined as the ratio of the increment in the back pressure to an increase in cell pressure \( \frac{\Delta \pi}{\Delta \sigma} \). If the ratio is equal to or greater than 0.90, then an acceptable saturation is reached (Bishop and Henkel, 1962).

20. Sometimes the specimen is left overnight to ensure complete saturation. The B values of present tests were around 0.93 to 0.97, suggesting that fairly high values of saturation are achieved.

**Testing Procedure**

The following steps are used in the triaxial testing.

1. The back pressure at the end of saturation is decreased or cell pressure is increased in order to reach the desired effective stress.

2. The loading frame assembly is lowered such that the assembly just touches the specimen. The frame is leveled and adjusted with set screws in order to achieve a level top plate.
3. The load ring is zeroed and the LVDT is positioned to measure the displacements.

4. The triaxial cell is then pushed upwards at a constant displacement rate of $0.05 \frac{mm}{mm}$.

5. The displacement, load, and pore pressures are automatically recorded in the data logger at every 5 seconds intervals.

6. The test is stopped when the load on the ring remarks constant for a period of time or it decreases and shows no further signs of decrease.

7. The software CLISP is used to interact with data logger. The raw data are reduced through this software to calculate the axial strain, deviatoric stress and excess pore pressures.

8. These results are stored in Lotus 123 files and the data is used to calculate the needed total and effective stress parameters.

9. These final results are then plotted by using the Grapher software.

10. When the test is completed, the triaxial cell is drained and cleaned for the next test.

Tests are conducted at different relative densities and different confining pressures.

3.2.2.2 Calibration Chamber Testing

i) Specimen Preparation

Cone penetration testing conducted on uncemented sands by several investigators reach the similar conclusion that testing under dry or full saturation has minor or no influence on the cone tip resistance (Schmertmann, 1976; Tunay, 1976; Baldi, 1981; Bellotti, et al., 1985). Hence, uncemented specimens are tested under dry conditions. This allowed the author to reuse the tested sand for a few more tests, provided there was no significant crushing around the cone. Grain size distributions conducted on the sand collected around this miniature cone revealed that there is no significant crushing of the sand grains.
Specimens are prepared by employing the following procedure:

1. The shutter is rotated to a position where the holes are misaligned and the plates prevent pluviation. The dry sand is then placed inside the top container.

2. The top cap with the guiding weight is placed above the sand.

3. The membrane is placed around the bottom plate and O-rings are placed around the specimen to achieve air tight conditions.

4. Vacuum is applied outside the chamber so that the membrane stretches around the chamber walls. This vacuum is maintained throughout specimen preparation.

5. The specimen chamber is then rolled underneath the table. The chamber is adjusted so that it aligns straight below the top two chambers.

6. The diffuser is placed inside the specimen chamber and all four threads or metal cables from the diffuser are connected to the top cap.

7. The shutter is rotated to align the holes and sand is pluviated.

8. During pluviation, the level of the sand in the top chamber goes down and the top plate moves downward. The downward movement of the top plate raises the diffuser by an equivalent distance keeping the falling height constant.

9. When the sand is completely rained out of the top chamber, the specimen chamber is carefully rolled out for weighing.

10. The extra sand on the top is carefully removed by a smooth straight edge.

11. The height of the specimen is measured.

12. The chamber is then weighed to calculate an average relative density for the specimen.

The guidelines suggested by Rad and Tumay (1987) are followed in selecting the pluviation variables. The important variables that affect the specimen relative density in pluviation are shutter porosity, height of sand fall and diffuser sieve...
size. Two shutters of porosities 6 and 15 % and three different sieves (ASTM #’s \( \frac{1}{2} \) in. (12.5 mm), \( \frac{3}{8} \) in. (9.5 mm) and \( \frac{1}{4} \) in. (6.3 mm)) are used. The height of fall is varied between 5 to 15 cm.

For higher relative densities, a shutter porosity of 6.5 % and a sieve opening of 6.3 mm are selected. For 65 to 75 % relative density, the shutter with the 6.5 % porosity and a sieve opening of 9.5 mm are used. For lower relative densities (45 to 55 %), the shutter with the 15 % porosity and sieves with openings of 12.5 mm or 9.5 mm are used.

Several specimens (10) are prepared using the same sieve opening, shutter and height of fall in order to evaluate the repeatability of the specimen preparation. The variation in relative density was found to be ±5 % in the case of 45 to 55 % range. In other cases i.e. when preparing 65 - 75 % or above 85 % relative density specimens, the variation is around ±7 %. This shows that reasonably repeatable specimens can be prepared using this technique.

ii) Testing Procedure

The chamber operation is split into 3 major operations.

1. Specimen transfer and placement

2. Consolidation

3. Cone penetration testing

1. Specimen Transfer and Placement

Specimen transfer and placement procedures are an important component of the testing procedure. Specimens may collapse when the chamber molds are separated. This is due to cohesionless nature of this material which does not exhibit any strength when there is no confinement. The following steps are taken in an attempt to minimize disturbance during specimen transfer and placement.

1. The specimen chamber with the specimen is lifted on to the calibration chamber following pluviation and it is placed carefully on the piston.

2. The top plate is placed over the specimen. O-rings are placed around the top plate covering the membrane.
3. A $\frac{1}{2}$ in. tubing with female end of the quick connector is connected to the top plate on the specimen and the other end is connected to the male end of the tubing from the vacuum pump (Plate 3.4).

4. A suction is applied to the specimen for approximately one hour and then the tube is disconnected from the pump by separating the quick connector.

5. The specimen and the chamber are then carefully lifted without any tilt and it is placed on the piston assembly.

6. The diametrically split molds of the specimen chamber are then removed. The specimen stands firmly without any buckling due to the confinement induced by the suction pressure (Plate 3.5). The inner and outer chambers are immediately lowered and placed over the specimen.

7. The vacuum remains the same without any loss, provided there are no holes in the membrane.

8. The inner chamber is lowered using the manually operated crane. Any slight tilt in the chamber position will strip the O-rings on the top plate causing the vacuum in the specimen to be released and resulting in the collapse of the specimen. These problems are experienced in the first few experiments.

9. Upon placement of the inner and outer cells, the outer most top plate is connected to the top plate via twelve bolts of size 12.5 mm. The outer top plate is later connected to the piston assembly through twelve equally spaced rods as shown in Plate 3.6. These rods are subjected to a torque of 65 kNm. This torque to each rod will ensure that the whole system is tightly and uniformly assembled to avoid any leaks during testing. The leaks in the system can be detected only when the deaired water in outer cell is pressurized during consolidation.

10. The inner and outer cells are filled with deaired water once the specimen is placed and all connections are made. This is accomplished by directing the water from the container by opening the valves on the control board.
Plate 3.4: Applying Vacuum Inside the Specimen
Plate 3.5: Specimen After Unfolding the Split Molds
Plate 3.6: Final Assembly of the Specimen
2. Consolidation

The next step is consolidating the specimen under a given vertical stress. This is accomplished through the data acquisition and reduction software developed by de Lima (1990). This software handles the following steps:

1. The input values like information about the soil sample and the vertical consolidation stress is read.

2. The system acquires data from eight electric transducers via analog to digital conversion and operates two transducers via digital to analog conversion.

3. It also displays graphically the real-time variation of stresses and vertical displacement.

The specimens are consolidated under $K_o$ conditions. The $K_o$ conditions require zero lateral strain. A set of procedure is followed during the process. When the specimen is ready for consolidation, the program CHAMBKO.EXE is to be executed. This can be done by typing CHAMBK0 and entering it. The program takes information about the soil specimen and inquires about the electro-pneumatic transducers (5.6 to 30 psi or 3 to 120 psi) and the pressure transducer ranges (0 to 30 psi or 0 to 100 psi) to be used.

The following steps are then followed in consolidation:

1. File name, details about the specimen, selected ranges of pressure transducers, a YES/NO option for applying equilibrium pressure, selection of electro-pneumatic transducer range and the value of consolidation stress are inputted.

2. The equilibrium pressure is the vertical pressure that has to be applied to the inner piston cell water in order to equilibrate the load generated by the weight of the specimen, specimen bottom plate and membrane. Any pressure higher than this equilibrium pressure will cause the piston move upwards initiating a consolidation. For YES option, a default pressure of 3 psi is applied prior to the consolidation. For the NO option, the user needs to supply this value via the control panel.

3. In the NO option, the program reads the equilibrium pressure via the computer from S4 or S5 transducer. This can be done by directing the compressed air.
flow from the air filter through transducer F1 while keeping valves B13 and B16 open. When the specimen equilibrium pressure is reached, and the specimen touches the top plate, the user should press the keyboard corresponding to answer NO. Subsequently, valves B3, B13 and B16 should be closed, and the compressed air flow from the air filter should be directed to the E/P transducer F1/F2 chosen for testing.

4. At this stage, the program takes all zero readings of the transducers, the tip and friction load cells and the LVDT. Once the valves B10/B11 and B16/B17 are closed and the valves B6 and B13 are opened on the control board, the computer takes zero readings. Immediately after the readings are displayed on the monitor, the operator should open valves B10/B11 and B16/B17 and close valves B6 and B13.

5. Computer flashes on the screen with the graphics featuring vertical deformation with time and vertical stress versus the horizontal stress.

6. The program will then read the final consolidation vertical stress and it will generate the necessary input to apply this stress on the specimen in increments. The first increment is calculated in the program as voltage. The program sends this voltage to the transducer F1/F2 where it is converted to pneumatic pressure. This pressure will make the piston move up thereby exerting a vertical pressure on the specimen.

7. When the vertical stress is applied, the specimen will expand horizontally. This will create a certain pressure in deaired water in the inner cell. The program reads this inner pressure and then sends a signal in volts equivalent to compensate for the inner pressure to the transducer F3/F4.

8. The electro-pneumatic transducer, F3/F4 will convert the voltage into pressure and the pressure will be applied to the deaired water in the outer cell. Since the pressure in the outer and the inner cells are the same, there will be no lateral movement and horizontal strain in the specimen.

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9. The other increments are also applied identically until the final vertical stress is reached. This process is known as $K_o$ consolidation since any lateral movement is not allowed.

The fundamental problems associated with this phase is emergence of leaks in the chamber. These leaks are found due to improper tightening of the rods surrounding the chamber and also due to the improper tightening of the screw bolts on the top plate. Any small gaps around these bolts will leak the water in the inner and the outer cells releasing the horizontal pressure.

3. Cone Penetration Testing

The final phase consists of conducting cone penetration testing at two or three different locations. Cone penetration testing is conducted under zero lateral strain and constant vertical stress. The program, CHAMBC3 is used to initiate the process; however the user has to close valves B6 and B17 on the panel of control before doing so. The following steps are used in this phase:

1. The file name for zero readings input, i.e. the file name for consolidation phase is to be entered,

2. The file name for data storage is to be given,

3. Laboratory testing information needed should be entered,

4. Program provides the option to change the display of the settings for different sounding depths,

5. Pressure transducer range (one used in consolidation phase) is to be selected,

6. The electro-pneumatic transducer used in the consolidation phase is to be selected.

Upon completion of the above steps, the computer screen flashes with the graphics which include the tip resistance, friction resistance, vertical stress, inner and outer horizontal stress versus depth. At this stage, penetration should be initiated. The steps involved in penetration are given below.

1. Prior to executing the program, it is necessary to lift the hydraulic jack with the help of the crane and place it above the top plate.
2. The jack is then clamped to the top plate at one of the four testing holes on the top plate with four equally spaced bolts. The jack which is in the collapse stage is raised to its original position.

3. The miniature cone is placed such that it just touches the specimen and at this position, the cone is fixed to the clamp on the jack.

4. Once the graphics screen is ready and flashing on the computer monitor, the cone is pushed into the specimen at a rate of $2 \text{ cm/sec}$. 

5. The readings are recorded at every $2 \text{ cm}$ intervals in the program. The test can be terminated by pressing any key.

6. When the test is completed, the jack is moved to another location and the above steps are repeated in conducting another test.

7. Upon completion of the tests, the chamber is cleaned and is kept ready for the next test.

The chamber assembly and specimen preparation takes about 60 to 90 minutes. The application of vacuum and lifting the specimen into the chamber takes at least 2 to 2 1/2 hours. Filling the inner and outer cells with deaired water takes around 30 and 40 minutes respectively. Consolidation process requires around 15 minutes. The total time for conducting two penetration tests on the specimen is about 5 hours. Dismantling and cleaning the chamber requires another 2 hours, totaling about 12 hours per test.

The data reduction pertaining to this test consist of two phases. The data from the test are first processed and then plotted on a plotter.

3.2.3 Procedure - Cemented Specimens

3.2.3.1 Triaxial Tests

Specimen Preparation

The specimen preparation procedure is similar to that of uncemented specimens. The only difference is that a cement/sand mixture is used in place of only the sand. The cement used in this study is ordinary portland cement. The cement sand mixture is prepared as follows:
1. The specimen mold made of plexiglas with membrane inside is placed on the bottom plate (Plate 3.7). These molds are 7.11 cm (2.8 in.) in diameter and 15 cm (6 in.) in height.

2. The bottom plate has four equally spaced holes for allowing water into the specimen. These holes are covered with a filter paper prior to pluviation.

3. 1000 g of dry sand is weighed and placed in the bowl.

4. For samples with 1 or 2 % (of the dry weight of sand) cementation, 10 or 20 g of portland cement is weighed respectively.

5. 4 g of water (0.4 % by the dry weight) is sprayed on the sand and is stirred with a hand mixer. This creates a slight moisture on the surface of the sand which allows an even distribution of the cement particles upon mixing.

6. The cement is added gradually during mixing. The mixing is continued for 60 seconds.

7. The mixer is stopped and further mixing is continued with hand. This gives a better coating of the sand particles by cement.

8. This mixture known as cement sand is used in specimen preparation.

9. The specimen is equally distributed in six beakers. The rest of the steps in the specimen preparation are similar to those described in preparation of uncemented specimens.

10. The specimen together with plexiglas sleeve is weighed to calculate the relative density.

11. The specimen in the plexiglas container is then placed on a bed of sand in a bucket.

12. Water is introduced into the bucket so that its level gradually rises (approximately 0.5 mm per minute). The sample is completely submerged underwater in approximately 5 to 6 hours.
Plate 3.7: Plexiglas Molds for Triaxial Specimens
13. The bucket containing the sample is transferred to the humidity room for curing.

A curing period of 7 days is selected in this study. This curing period is enough to form cemented bonds that are required for natural cemented deposits while it also allows preparation of sufficient number of specimens in a short period of time. The scanning electron micrographs of the cemented and uncemented sands are shown in the Plate 3.8. The crystallization process that might have taken place during curing is depicted in this plate.

Testing Procedure

The undrained testing is similar to that of uncemented specimens. The only difference is that higher back pressures are needed during the testing since cemented sands dilate more than uncemented sands. Samples were fully saturated prior to each test. The deviatoric stress, axial strain and the excess pore pressure build up are recorded.

3.2.3.2 Calibration Chamber Testing

Specimen Preparation

The following procedure is used in cemented specimen preparation for the calibration chamber study:

1. 13500 gm of dry sand is weighed and placed in a plastic bucket.
2. 135 (1 % cementation) or 270 (2 %) gm of cement is added to the sand.
3. This dry mixture is placed in a mixer and the container is transferred under the mixer.
4. Prior to mixing, 54 gm of water is added and mixed with hand.
5. The mixing is started and is continued for about 120 seconds.
6. The coated cement on mixer blades are wiped off into the sample and the mixing is continued for another 120 seconds.
7. The cement sand mix is carefully transferred in to the top chamber in the pluviation setup.
Plate 3.8: Scanning Electron Micrographs of Uncemented and Cemented (0 and 2 \%\) sand
8. The above steps are repeated until the top chamber in the pluviation setup is filled.

9. The rest of the process is similar to that used in uncedmented sands.

10. When pluviation is completed, small amount of cement is noticed left in the top specimen chamber. This segregated cement is often 3 to 4 % of the total cement added in each bucket (4 to 6 gm out of 135 gm cement). Hence, it's effect is neglected.

11. The specimen chamber with cememnted sand mix is weighed to calculate an average relative density.

12. The specimen is moved and it is connected to the saturation tank (Plate 3.9).

13. The carbon dioxide is allowed into the specimen from the bottom replacing the air in the specimen.

14. The water from the tank is allowed to enter into the specimen.

15. When the specimen is completely submerged, the specimen is transferred to a humidity room for curing.

The specimens are checked for homogeneity and cementation. It is assumed that homogeneity is achieved since pluviation studies on uncedmented specimens prove that reasonably homogeneous, repeatable specimens can be prepared. However, the main concern is whether the pluviation process preserves the cementation bonds formed at the time of specimen preparation. Hence, samples are collected at four different depths, 10, 25, 40 and 50 cm. The relative densities are calculated and are plotted in Figure 3.5. The specimen is reasonably homogeneous, while standard deviations are of the order of 2 to 3 %. Scanning electron micrographs for these samples suggest that the pluviation process did not result in segregation and the cemented bonds exist between sand grains (Plates 3.10 and 3.11). Cement material around the sand particles can be seen at all depths. Subsequent to curing, the specimens are transferred to the calibration chamber for the testing.
Plate 3.9: Specimen Undergoing Saturation Process
Figure 3.5: Relative Density Versus Depth in Cemented Specimens
Plate 3.10: Scanning Electron Micrographs of Cemented Specimen at 10 and 25 cm Depths
Plate 3.11: Scanning Electron Micrographs of Cemented Specimens at 40 and 50 cm Depths
Testing Procedure

Testing procedure is similar to that described in uncedmented sands. Cem­
tation induces some cohesion. However, this cohesion is not sufficient to provide
the necessary confinement during transfer and placement of inner and outer cells.
Hence, application of vacuum is found necessary before removing the mold. The
vacuum pressure ranged between 60 to 70 \( kPa \). This initial vacuum will induce
a pre-consolidating effect on the specimen; however these specimens are later con­
solidated to stresses higher than 100 \( kPa \) and hence they are considered normally
consolidated.

3.2.3.3 Unconfined Compression Test

Specimen preparation in unconfined compression tests is similar to those prac­
ticed used in preparing triaxial specimens. The only difference is the size of the
specimen. Plexiglas molds of 7.6 \( cm \) diameter and 23 \( cm \) height are used in these
tests.

It is well known that curing period increases the compressive strength. The
effect of curing periods of 3, 7, 14 and 28 days on unconfined compressive strength
is investigated.

3.3 Summary

Specimen preparation and testing procedures are described. Specimen prepara­
tion procedure did not result in excessive segregation of cement and the cem­
ented bonds are observed to be intact. The vacuum application inside the specimen prior
to transferring into the chamber is found necessary and this procedure is necessary
even in the case of cem­mented specimens. Curing period of 7 days and 1 \% to 2 \%
cement contents are probably enough to simulate the very weakly cem­ented natural
deposits.
Chapter 4

ENGINEERING BEHAVIOR OF MONTEREY NO 0/30 SAND

4.1 Introduction

The results of consolidated drained, consolidated undrained triaxial compression tests and unconfined compression tests are presented. Strength-deformation behavior of Monterey No. 0/30 sand is evaluated in comparison with previously reported studies. A theoretical model is used to model the behavior of this sand under drained and undrained testing conditions. These modeling parameters are later used in the modeling of cone penetration testing.

The tests are coded in the following manner: MSC0CU45, where MS stands for Monterey Sand No. 0/30; C0 stands for cementation level or cement content (0, 1 and 2); CU stands for isotropic consolidated undrained triaxial test (CD is consolidated drained test, UC is unconfined compression test) and 45 represents the relative density of the tested specimen. Physical characteristics of the sand and cement are presented in Chapter 3.

The testing program consisted of conducting 30 undrained triaxial tests on un-cemented and artificially cemented sands. Cementation levels of 0, 1 and 2 % and three ranges of relative density (45 to 55, 65 to 75 and above 85 %) are used. The tests are conducted at confining stresses of 100, 200 and 300 kPa. The objectives of this testing program are established as:

1. to define the strength-deformation parameters and to compare them with the results reported by previous studies (Rad, 1984) in an attempt to assess repeatability and to evaluate whether previously reported results can be used in current evaluations,

2. to provide a basis for correlating penetration parameters with the strength parameters,
3. to develop critical state lines for this sand (cemented and uncemented) and to use them in the critical state interpretation of the cone penetration test results,

4. to obtain the necessary theoretical modeling parameters such as dilational angle and shear modulus and to use them in modeling cone penetration with the cavity expansion theory.

A separate study of the effect of curing on unconfined compression strength is also initiated. Unconfined compression strength in cemented sands provides an indirect measure of the cohesion intercept. The cone penetration tests in the calibration chamber are conducted with 7 days cured specimens. Specimen placement and transfer in calibration chambers often took several hours (8 to 10 hrs). The concern then arises that slight variations in curing may affect the strength-deformation behavior and hence the tip and friction resistances. The unconfined compression strength study provides an evaluation of the relative effect of such changes. Furthermore, this study renders a qualitative evaluation of the expected strength increase.

4.2 Strength-Deformation-Pore Pressure Response

The triaxial compression test is widely used to determine the shear strength parameters, the effective angle of internal friction $\phi'$ and the cohesion intercept, $c'$. The shear strength, $\tau$ as per Mohr-Coulomb failure criterion is expressed as

$$\tau = c' + \sigma_n' \tan \phi'$$

where $\sigma_n'$ is the normal effective stress acting on the failure surface. Three different triaxial tests, consolidated drained (CD), consolidated undrained (CU) and unconsolidated undrained (UU) tests are conducted to determine shear strength parameters.

Consolidated undrained tests are conducted on cemented specimens in order to determine total and effective strength parameters. Figures 4.1 to 4.6 shows the stress-strain and excess pore pressure-strain response for un cemented and cemented sand samples. The relative density and confining stresses are depicted in each figure. It is necessary to use back pressure both to fully saturate the specimen and also to measure negative pore pressures. All specimens were saturated using back pressures ranging from 400 to 600 kPa depending upon the density. Denser specimens (85 %}
relative density) necessitated back pressures of up to 600 kPa. One other reason for the development of negative pore pressures was cementation and the increase in dilation with cementation can be seen in Figures 4.3 to 4.6. Rest of the figures are presented in Appendix B.

4.3 Strength Parameters

Ladd (1966) suggested different approaches for obtaining strength parameters from undrained testing: (1) at maximum stress difference, (2) at maximum obliquity, and (3) at the point of tangency to the effective stress path. Third procedure is generally recommended for dilating type materials, hence, present tests are analyzed using this method.

Figures 4.7 to 4.9 show the stress paths in CU tests. The rest is presented in Appendix B. These are used in calculating the peak and residual strength parameters. The stress paths show that the effective stress paths in all tests lie to the right of the total stress path suggesting the development of negative pore pressures.

The effective stress path also shows the initial positive pore pressure distribution and then the negative pore pressure distribution due to dilation. The effective stress paths after reaching peak, move towards left and this is due to the drop in deviatoric stress at the residual strains. The effective stress paths of cemented and uncemented sands are practically parallel, suggesting cementation has minor influence on friction angle (Figure 4.10). Cementation leads to cohesive binding between the grains and cohesion is entirely mobilized at a deviatoric stress lower than peak deviatoric stress. At large strains, disintegration of cemented bonds take place which results in lower residual cohesion values.

Table 4.1 presents effective strength parameters at each cementation level. In all cases, an increase in relative density results in a rise in the friction angle. Similar observations are also made in cemented sands. This is attributed to the higher the density, higher the number of particle contact points between grains hence higher friction values. Cementation does not result in a significant increase in friction angles at the same densities.

Cohesion intercept is zero for uncemented sands. However, a cohesion intercept emerges in cemented sands. This value increases at higher densities at the same
Figure 4.1: Undrained Triaxial Test on Uncemented Monterey No.0/30 Sand ($D_r = 45\%$)
Figure 4.2: Undrained Triaxial Test on Uncemented Monterey No.0/30 Sand ($D_r = 65\%$)
Figure 4.3: Undrained Triaxial Test on Uncemented Monterey No.0/30 Sand ($D_r = 80\%$)
Figure 4.4: Undrained Triaxial Test on Cemented Monterey No.0/30 Sand ($D_r = 45$ and C.C. 1 %)

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Figure 4.5: Undrained Triaxial Test on Cemented Monterey No.0/30 Sand ($D_r = 65$ and C.C. 1 %)
Figure 4.6: Undrained Triaxial Test on Cemented Monterey No.0/30 Sand ($D_r = 80$ and C.C. 1 %)
Figure 4.7: Stress Paths From Triaxial Tests on Uncemented Monterey No.0/30 Sand ($D_r = 80 \%$)

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Figure 4.8: Stress Paths From Triaxial Tests on Cemented Monterey No.0/30 Sand

\[ D_r = 80\%; \ C.C. = 1\% \]
Figure 4.9: Stress Paths From Triaxial Tests on Cemented Monterey No.0/30 Sand
($D_r = 80 \%$; C.C. = 2 \%)
Figure 4.10: Stress Paths of Cemented and Uncemented Sand

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Table 4.1: Effective Strength Parameters of Cemented Sands (Undrained Tests)

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>Cement Content</th>
<th>Cohesion ( (\frac{\text{kN}}{m^2}) )</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>%</td>
<td>peak</td>
<td>res</td>
</tr>
<tr>
<td>45</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>65</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>85</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>45</td>
<td>1</td>
<td>19.7</td>
<td>6.3</td>
</tr>
<tr>
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<td>1</td>
<td>40.6</td>
<td>19.2</td>
</tr>
<tr>
<td>85</td>
<td>1</td>
<td>54.5</td>
<td>32.2</td>
</tr>
<tr>
<td>45</td>
<td>2</td>
<td>25.0</td>
<td>6.12</td>
</tr>
<tr>
<td>65</td>
<td>2</td>
<td>70.5</td>
<td>37.5</td>
</tr>
<tr>
<td>85</td>
<td>2</td>
<td>83.3</td>
<td>42.0</td>
</tr>
</tbody>
</table>

cementation level. At high densities, more contacts exist between the particles and the probability of having more cemented contacts across a plane increases.

Similar observations are noted in the residual strength parameters. Significantly lower cohesion intercept values are obtained at larger strains, this is mainly hypothesized to be due to the breaking of the cemented bonds.

4.4 Comparison with Drained Results

The undrained results are compared with drained results in Table 4.2 (cohesion) and 4.3 (friction angle). Drained test results are obtained from the on-going study by Arslan (1993). 7 days cured specimens of Monterey No. 0/30 sand are tested in this study.

Effective friction angles in drained testing are lower than those in undrained testing. The variation is 5 to 6° in some cases. This variation can be expected when the \( q - p' \) envelopes are used to estimate \( c \) and \( \phi \) values. Similar observations are also noted by Bjerrum (1960). Drained and undrained friction angles are compared from the results conducted on several types of clays (Bjerrum, 1960). The envelopes from stress paths which are equivalent to envelopes at maximum principal effective stress ratios (maximum obliquity) yield different \( c \) and \( \phi' \) values than those obtained from drained tests. Drained friction angles are 3° lower than undrained effective friction angles and in some cases the difference is as high as 7° (Bjerrum, 1960). This variation

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
Table 4.2: A Comparison of Cohesion Values Obtained in Drained and Undrained Tests

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>Cement Content</th>
<th>Cohesion(CD) ($\gamma^c$)</th>
<th>Cohesion(CU) ($\gamma^c$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>peak</td>
<td>res</td>
</tr>
<tr>
<td>45</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>65</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>85</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
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</tr>
<tr>
<td>85</td>
<td>1</td>
<td>15.0</td>
<td>5.0</td>
</tr>
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<td>2</td>
<td>10.0</td>
<td>5.0</td>
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<td>7.0</td>
</tr>
<tr>
<td>85</td>
<td>2</td>
<td>25.0</td>
<td>13.0</td>
</tr>
</tbody>
</table>

NA-Not Available

is possibly due to the differences and the development of pore pressures with strains in undrained testing.

Cohesion values are higher in undrained testing due to the development of negative pore pressures. These pore pressures induce extra confinement on the sample and the final strength will be more affected by slight differences in densities across the specimen. This may be the reason for higher cohesion values. The increase in curing period also contributes to this variation. The influence of curing period on cohesion is separately investigated.

Selection of either drained parameters or undrained parameters depends upon the conditions prevailing around the cone during penetration. If the hydraulic conductivity of the soil is high enough to dissipate the excess pore pressures developed, then drained triaxial parameters are used in interpretation.

4.5 Critical State Diagram

Application of critical state soil mechanics to sands has been less successful than clays due to the difficulty in defining a virgin consolidation line (Been et al., 1991). However with the development of modern laboratory techniques such as undrained triaxial tests, measurement problems were resolved and thereby critical state lines for sands were generated. There has been a lot of discussion on whether critical and
Table 4.3: A Comparison Between Effective Friction Values Obtained in Drained and Undrained Tests

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>Cement Content</th>
<th>Friction(CD)</th>
<th>Friction(CU)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>%</td>
<td>peak</td>
</tr>
<tr>
<td>45</td>
<td>0</td>
<td></td>
<td>36.5</td>
</tr>
<tr>
<td>65</td>
<td>0</td>
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<td>38.7</td>
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<td></td>
<td>40.5</td>
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<tr>
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<td></td>
<td>38.5</td>
</tr>
<tr>
<td>85</td>
<td>2</td>
<td></td>
<td>39.4</td>
</tr>
</tbody>
</table>

NA - Not Available

steady state lines are identical (Been et al, 1991).

There seems to be a difference between these two lines based on the method of measurement. Critical state line is derived from drained, strain-rate-controlled test results on dilatant samples, whereas steady state line is obtained from undrained triaxial tests. Tests conducted on a Erksak sand showed that the critical and steady state lines from drained and undrained tests are identical, implying testing has no influence on the ultimate state (Been et al., 1991).

The state parameter concept developed by Been et al., (1986) provides a reference critical state from which the state parameter (definition of state parameter is presented in Chapter 2) and other sand properties are derived. The state parameter concept is used in interpretation of cone results. Results from undrained triaxial tests are used in estimating critical confining pressures and also in developing the critical state line.

The critical confining pressure is the confining pressure at which there is no volume change. In case of undrained testing where the volume change is constant, critical confining pressure is evaluated at larger or residual strains where there is no significant change in excess pore pressure. The critical confining pressure is calculated for each confining pressure at a particular relative density. The average of the critical confining pressures obtained in the tests is defined as the critical confining pressure.
pressure at that density. The above procedures are repeated for each relative density and cement content. Steady state lines are plotted in Figure 4.11. The figure shows that steady state lines for cemented specimens locate close (slightly to the right) to the uncemented critical state line. The variation between cemented and uncemented values is minor since they are plotted on a logarithmic scale. In order to show the difference, the results are replotted on natural scale (Figure 4.12). Cemented specimens dilate more than uncemented specimens implying development of higher negative excess pore pressures leading to higher effective confining pressures and higher critical octahedral stresses. Since there is no significant change in void ratio in undrained testing on cemented specimens, the steady state line (SSL) for cemented specimens moves to the right and above the SSL corresponding to the uncemented samples. The above SSL are plotted comparatively with the SSL from tests conducted on Erksak sand (Been et al., 1991) and Monterey No. 0/30 sand (Been et al., 1986) in Figure 4.13. Tests on Monterey No. 0/30 sand are conducted at lower confining stresses. The steady state line is generally approximated by a straight line in $e - \log p'$ space and this approximation is valid for sands with sub-angular to sub-rounded particles (Been et al., 1991). However, the shape of the line over a wider stress range is different and it is similar to the shape plotted for Erksak sand in Figure 4.11. Steady state line curves abruptly at a stress level of 1 MPa. This break in the steady state line is indicative of a change in mechanism of shearing at higher levels of stress (Been et al., 1991). It is also hypothesized that at higher confining stress levels, there appears to be some breakage of grains which result in the abrupt change in the slope of the SSL (Been et al., 1991).

Combining the present data on Monterey No. 0/30 sand (conducted at higher confining stresses) with Been's results on Monterey sand (conducted at low confining stresses), a complete steady state line similar to that of Erksak sand is constructed. SSL of Erksak and Monterey sands are parallel to this line and then vary due to different grain sizes and shapes.

This steady state line is used to derive steady state parameter, which is subsequently used in the interpretation of cone penetrometer results.
Figure 4.11: Steady State Line Diagrams for Cemented and Uncemented Sands
Figure 4.12: Steady State Line Diagrams for Cemented and Uncemented Sands
Figure 4.13: Comparisons of SSL
4.6 Modeling Parameters

A simple soil model proposed by Juran and Guermazi (1988) is updated and used to analyze the triaxial test results. This model was essentially developed to simulate soil response in cavity expansion tests. This model assumes soil to be homogeneous and isotropic. A strain-hardening elasto-plastic material behavior with a non-associated flow rule is used. The strain hardening parameter is chosen to be the deviatoric strain ($\gamma$). The stress invariants used in formulations are $q_m$, the deviatoric stress, $(\sigma'_1 - \sigma'_3)$ and $p'_m$ and the effective octahedral normal stress ($\sigma_{oct}$: $(\sigma'_1 + 2\sigma'_3)/3$). The difference between stress invariants, $p'_m$, $q_m$ and the stress path variables, $p$, $p'$ and $q$ should be noted.

4.6.1 Yield Function

A Mohr-Coulomb type yield criterion is considered:

$$f(\sigma_{ij}, \gamma) = \frac{q_m}{p'_m} - h(\gamma)$$ (4.2)

where $\sigma_{ij}$ is the stress tensor, $q_m = (\sigma'_1 - \sigma'_3)$ is the deviatoric stress, $p'_m = (\sigma'_1 + 2\sigma'_3)/3$ is the effective octahedral stress in triaxial testing conditions, $\sigma'_1$, $\sigma'_3$ are, major and minor effective stresses respectively. The $h(\gamma)$ is the strain hardening function relating the actual yield surface to the current strain rate. Shear strain, $\gamma = \varepsilon_1 - \varepsilon_3$ and is assumed to be the strain hardening parameter (Juran and Beech, 1986).

The above yield function is generally used for frictional or cohesionless materials. Since, cemented materials exhibit a cohesion intercept, failure criterion is updated as:

$$q_m = C\phi C + M\phi p'_m$$ (4.3)

where

$$M\phi = \frac{6 \sin \phi}{3 - \sin \phi}$$ (4.4)

$$C\phi = \frac{6 \cos \phi}{3 - \sin \phi}$$ (4.5)

To make analysis simpler, yield function which intersects $q$ (y axis on $q_m - p'_m$ plot) at $C\phi C$ is extended to intersect the $p'_m$ axis. This gives the following form to the yield function.

$$q_m = p'_{um} M\phi = (p'_1 + p'_m) M\phi$$ (4.6)
where \( p'_i = \frac{c G_k}{M_\phi} \) and \( p'_u = p'_i + p'_m. \) Then

\[
f(\sigma_{ij}, \gamma) = \frac{q_m}{p'_u} - h(\gamma) \tag{4.7}
\]

During modeling, the imaginary ordinate, \( p'_i \) are deducted from the computed \( p'_u \) values at each strain level in order to get the correct \( p'_m \) values. The corrected values represent the actual \( p' \) values for the above \( c - \phi \) yield function.

In the analysis, the following assumption is made: If the peak cohesion value is used in the \( p'_i \) values, the corrected \( p' \) values at shear strains corresponding to residual stresses will be lower than actual values. This, coupled with the difficulty of selecting a different cohesion value at each shear strain, \( \gamma \), lead to the use of an average cohesion intercept (average of peak and residual cohesion intercepts) in the analysis. Even though this results in some difference in \( p' \) values, the discrepancy can be disregarded for all practical purposes.

### 4.6.2 Hardening Function

The strain hardening function, \( h(\gamma) \), used in the above equation must be specifically defined for the case of contracting and dilating material. For loose contracting sands, a hyperbolic strain hardening function with two material constants is often used:

\[
h(\gamma) = \frac{\gamma}{a + b \gamma} \tag{4.8}
\]

For a triaxial test, \( a = \frac{\sigma_0}{2G} \), \( b = \frac{1}{M_{\phi,sa}} \) where \( \sigma_0 \) is the initial consolidation stress, \( G \) is the initial shear modulus, and \( \phi_{cv} \) is the friction angle at constant volume.

For dense dilating sands, it is assumed that the hardening function \( h(\gamma) \) is parabolic to hyperbolic and can be written as:

\[
h(\gamma) = \frac{c \gamma(\gamma - a)}{(\gamma + b)^2} \tag{4.9}
\]

where the constants \( a, b \) and \( c \) are determined from the following conditions:

1. the initial tangent modulus of the \( h(\gamma) \) function is equal to \( \frac{\sigma_0}{\sigma_0} \),
2. at peak stress ratio, \( h(\gamma) = M_{\phi} \), and
3. at the critical state, \( h(\gamma) = M_{\phi,cv} \),
these conditions yield:
\[
\begin{align*}
ap &= -2 \frac{\sigma_0 M_\phi}{G M_{\phi,cv}} \\
b &= \frac{\sigma_0}{G} M_\phi \Gamma \\
c &= M_{\phi,cv}
\end{align*}
\] (4.10)-(4.12)
where \(\Gamma = 1 + [1 - (M_{\phi,cv} M_\phi)]^{\frac{1}{2}}\).

### 4.6.3 Flow Function

Adopting Mohr-Coulomb or the more generalized Drucker-Prager yield criteria along with an associated flow rule leads to overestimation of dilatancy. Therefore, a non-associated flow rule must be used (Juran and Beech, 1986). The following flow rule which defines stress ratio-dilatancy relationship is derived by Juran and Beech (1986) based on energy considerations:
\[
\eta = \sin \nu = \frac{d\epsilon_v^p}{d\gamma^p} = \frac{1}{\mu_s} (M_{\phi,cv}) - \frac{q_m}{p_m'}
\] (4.13)
where
\[
\begin{align*}
d\epsilon_v^p &= \text{plastic volumetric strain increment} \\
d\gamma^p &= \text{plastic deviatoric strain increment} \\
\eta &= \text{dilation rate} \\
\nu &= \text{dilation angle}
\end{align*}
\]
and \(\mu_s\) is a correction modulus defined as
\[
\begin{align*}
\mu_s &= \mu_1 \text{ when } \frac{q}{p'} \leq M_{\phi,cv}; \text{contracting behavior} \\
\mu_s &= \mu_2 \text{ when } \frac{q}{p'} \geq M_{\phi,cv}; \text{dilating behavior}
\end{align*}
\] (4.14)

### 4.6.4 Plastic Potential Function

A non-associated plastic potential function \(g(q_m, p'_m) = 0\) can be derived, assuming coincidence of principal axes of stress and plastic strain increments.
\[
g(p'_m, q_m) = \frac{q_m}{p_m'} - \psi(p'_m, M_{\phi,cv})
\] (4.15)
Figure 4.14 shows schematically the strain hardening function, $h(\gamma)$, the associated volumetric response, and the corresponding stress ratio-dilatancy rate function. Maximum dilatancy is equal to the slope of volumetric strain - shear strain curve at the peak of the $h(\gamma)$ function, whereas the dilatancy rate at the critical state is equal to zero (Juran and Mahmoodzadeh, 1989).

4.6.5 Model Results

This soil model requires five parameters: $\frac{G}{\sigma_0}$, $\phi_p$ or $MPHI \frac{6 \sin \phi_p}{3- \sin \phi_p}$, $\phi_{cv}$ or $MPHI_{cv}$, $\mu_1$ and $\mu_2$. Drained results from Arslan (1993) and Rad and Clough (1982) are used to determine these parameters. The following procedure is then adopted in soil modeling.

1. Three programs, written in Fortran are used for soil modeling. The flow charts and source listings of these programs are shown in Appendix B. The first program reads the drained data and calculates stress ratio, $q_m/p_m^\prime$, shear strain, $\gamma$ and dilation rate. From these results, initial values of $\phi_p$, $\phi_{cv}$, $\mu_1$, $\mu_2$ and $G/\sigma_0$ are estimated. These results are calculated for each test at each relative density.

2. These results are then input into a second program which simulates drained triaxial behavior. The output of these results contain $q_m/p_m^\prime$, volumetric strain and shear strain. These results are then compared with experimental drained results. If the simulation is not in agreement with the experimental results, the program is rerun with different parameters, $G/\sigma_0$, $\mu_1$ and $\mu_2$ values. The parameters that rendered best simulations are tabulated in Table 4.4 and 4.5. Some of the results are shown in Figures 4.15, 4.16 and 4.17. Rest of the comparison plots are presented in Appendix B. The peak and residual (constant volume) $\phi$ values are very close to the experimental results suggesting that best simulations obtained.

Figures 4.18 and 4.19 shows the comparison between volumetric strain and shear strain. Rest of the comparison plots are presented in Appendix B. Comparisons show that $\mu_1$ value is 0.55 in the contraction zone and $\mu_2$ can vary from 0.20 to 0.6 in the expansion zone, based upon the relative density. Lower $\mu_2$ values yield higher dilational strains which are represented by negative values.
Figure 4.14: Assumed Constitutive Equations and Related Soil Parameters (Juran and Beech, 1986)
Figure 4.20 shows the influence of varying $G/\sigma_0$ on prediction of drained strength-deformation behavior. It is observed that the variation is minor within the range of changes observed in this study (100 to 200 kPa).

Table 4.4: Modelling Parameters

<table>
<thead>
<tr>
<th>C.C.</th>
<th>$D_r$</th>
<th>$G/\sigma_0$</th>
<th>$\phi_p$</th>
<th>$\phi_cv$</th>
<th>$c_{av}$</th>
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<tbody>
<tr>
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<td>%</td>
<td>%</td>
<td>%</td>
<td>%</td>
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</tr>
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<td>40.0</td>
<td>37.0</td>
<td>25.0</td>
</tr>
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</table>

3. The third program simulates undrained triaxial behavior. The parameters that best simulated drained behavior are used in this program. The output from this program consists of $q_m/p_m'$, excess pore pressure and shear strain. These are compared with experimental results of the present investigation. Figures 4.21, 4.22 show these comparisons. Comparisons show that simulations from the model closely approximate the experimental results.

Table 4.5: Modelling Parameters

<table>
<thead>
<tr>
<th>$D_r$</th>
<th>$\sigma_v$</th>
<th>$\mu_1$</th>
<th>$\mu_2$</th>
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<tbody>
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<tr>
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Figure 4.15: Comparisons Between Predicted and Experimental Drained Triaxial Tests

\[ G/\sigma_0 = 115.0; \sigma_v = 0.0 \]
\[ \phi_p = 40.5; \phi_v = 36.0 \]

Comparisons (MS-C0-DR85)
COMPARISONS (MS-C1-DR85)

G/SIG0 = 120.0; Cav = 6.2
φp = 39.0; φcv = 36.4

Figure 4.16: Comparisons Between Predicted and Experimental Drained Triaxial Tests

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Figure 4.17: Comparisons Between Predicted and Experimental Drained Triaxial Tests

G/SIg0 = 170.0; C_{sv} = 25.0
\varphi_p = 40.0; \varphi_{cv} = 37.0

- Experimental (100 kPa)
- Experimental (200 kPa)
- Experimental (300 kPa)
- Theoretical

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Figure 4.18: Comparisons Between Predicted and Experimental Drained Triaxial Tests (Volumetric Strains)
Figure 4.19: Comparisons Between Predicted and Experimental Drained Triaxial Tests (Volumetric Strains)
Figure 4.20: Influence of $G/\sigma_0$ on Stress Ratio

$\phi_p = 35.3; \ \phi_{cv} = 34.1$

$C_{av} = 0.0$

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Figure 4.21: Comparisons Between Predicted and Experimental Undrained Triaxial Tests

G/SIG0 = 115.0
\( \varphi_p = 39.1; \varphi_{cv} = 36.0 \)
Figure 4.22: Comparisons Between Predicted and Experimental Undrained Triaxial Tests
Dilational angles calculated from drained results from program 1 are shown in Figure 4.23. Other model parameters, \( G/\sigma_0 \), \( \phi_p \) and \( \phi_{cv} \) versus relative density and cement content are presented in Table 4.4. These results along with dilational angle are later used in estimating the limiting pressures by the spherical cavity expansion theory. This limiting pressure can be correlated with tip resistances.

### 4.7 Unconfined Compression Tests

In an attempt to shorten the time of testing, it was decided to test calibration chamber specimens after 7 days of curing. Therefore, a curing time of 7 days was taken as reference all throughout the study. The transfer and placement of the specimen in the calibration chamber along with cone penetration testing often took between 10 to 12 hours. The question then arose whether this excess time of curing would affect the strength significantly. If this change is significant, then any correlations made with using the strength-deformation behavior of the 7 day cured specimens would have involved an error.

Unconfined compression tests are conducted on 1 and 2 % cemented specimens. Specimens are prepared at three ranges of relative densities and they are cured at 3, 7, 14 and 28 days.

The test results are shown in Figures 4.24 to 4.25. Figure shows that there is an increase in the unconfined compression strength with curing period, however the strength increase beyond 7 day curing period is insignificant. These plots are used to prepare Figure 4.26 showing \( dq/dt \) versus time, \( t \) for the 1 % cemented specimen. The slope, \( dq/dt \) decreases significantly with time, particularly after 14 days, implying curing may not affect results significantly beyond 14 days.

The 7-day curing period is chosen for convenience and all comparisons are made at the same curing period.

The effect of temperature on unconfined compression strength of cement stabilized materials was conducted by Dumbleton (1962). The rate of increase of strength increased with an increase in temperature, hence all the tests, triaxial, unconfined compression and calibration chamber tests are conducted at room temperature.
Figure 4.23: Dilational Angles of Uncemented and Cemented Sand
Figure 4.24: Influence of Curing Time on Unconfined Compression Strength of 2% Cemented Specimen
Figure 4.25: Influence of Curing Time on Unconfined Compression Strength of 1% Cemented Specimen

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Figure 4.26: $dq/dt$ versus Time on 1% Cemented Specimens

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4.8 Summary

Several triaxial and unconfined compression tests conducted on cemented and un cemented Monterey No. 0/30 sand reconfirmed the following observations.

- Cementation induces cohesion intercept in sandy soils.
- Cementation has minor influence on friction angle.
- Cementation, as like relative density, increases the dilational behavior.
- Cementation increases initial shear modulus.
- Drained parameters are lower than the undrained effective stress parameters. This is due to the procedures used (envelopes drawn in stress paths) in analyzing the test data. Similar procedures used by Bjerrum (1960) also observed higher undrained friction angles and cohesion values than in the drained testing.
- Steady state line for Monterey No. 0/30 sand is similar in shape to that of Erksak sand (Been et al., 1991) and the variation in magnitudes of $e$ and $p'$ are due to the different size and shape of the aggregates and their dilational behavior during shearing.

The above observations signify the importance of cementation. The curing period increases unconfined compression strength. However, this increase beyond 7 day curing period is insignificant. In theoretical calculations in the interpretation of cone penetration testing results, the strength parameters are taken exactly at the same curing period that is used for curing chamber specimen on which cone test is conducted.

A simple elasto-plastic soil model is used to simulate drained and undrained behavior. Yield function in terms of cohesion and friction is developed and a non-associated flow rule is adapted in the modeling. Drained test results are first programmed to evaluate the necessary approximate parameters. These are then used in the second and third programs to simulate drained and undrained behavior. The parameters obtained in drained simulations are well matched with experimental results. Undrained simulations also closer to experimental results. These model parameters are...
will be used in the interpretation of cone penetration results by spherical cavity expansion theory.
Chapter 5

CONE PENETRATION TESTING

The difficulty in undisturbed sampling of natural cohesionless deposits, the cost and complications of any envisioned field-scale calibration attempts prompt investigators to use laboratory scale experimental models. Calibration chambers constitute such experimental models used in calibrating insitu testing devices.

An experimental model should ideally replicate the geometrical considerations and boundary conditions representative of field conditions. Cone penetration in a homogeneous deposit under field conditions constitutes an axisymmetric, semi-infinite, steady-state penetration problem. It is a simple task to down-scale the axi-symmetric nature of the problem into the laboratory by constructing cylindrical chambers and conducting experiments at the center. However, the major concern arises while down-scaling the semi-infinite boundary to finite boundary. The question then arises; what the ratio of the diameter of the chamber to the cone diameter should be so that the results are not affected by the finite boundary conditions. A systematic study is not yet available; however the data generated by the increasing number of chambers of different sizes provide some statistical trends.

There are two philosophical approaches to the study of the geotechnical characteristics of deposits by insitu testing devices in an attempt to offer empirical correlations or to assess the validity of the theoretical predictions; 1) conducting field testing followed by comprehensive laboratory testing and evaluation of soil characteristics, composition, environmental variables, 2) conducting experimental model tests in the laboratory under controlled compositional and environmental variables.

Both philosophies have their advantages and disadvantages. In the case of penetration in cohesive soils, the first approach is generally followed. This is possibly due to the relative ease in retrieving undisturbed cohesive samples and also the time constraints associated with reconstituting and consolidating large-scale fine-grained specimens in the laboratory. However, calibration chamber testing on reconstituted specimens has been favored for coarse grained deposits possibly due to the extreme difficulty encountered in sampling such deposits. The disturbance concern is particu-
larly true in case of un cemented or lowly cemented deposits since changes in fabric or breaking of the cementation bonds will strongly affect the results. Consequently, it seems more prudent and cost-effective to study cone penetration in cemented sands in calibration chambers under specified composition, fabric and environmental factors. It is envisioned that complications arising from changes in fabric and gradation are introduced and evaluated sequentially.

In line with the above presented reasoning and philosophy, the testing reported in this study is conducted in a calibration chamber on reconstituted, un cemented and artificially cemented specimens of Monterey No. 0/30 sand. This chapter presents the results obtained in these tests together with a comparative discussion of the influence of various factors like relative density, cement content, testing location on the results.

5.1 Testing Program

The tests are conducted in a double-walled flexible cylindrical calibration chamber on specimens of 53 cm diameter and 79 cm in height. A miniature quasi-static cone penetrometer (MQSC) of 1.2 cm in diameter is used, rendering a diameter ratio of 42. The cone used was a friction cone. The details of the chamber, cone and testing procedures are presented in Chapter 3. Monterey sand No. 0/30, a sand commonly used in laboratory experiments, is used in tests. Ordinary portland cement is used as cementing agent in cemented specimen preparation. Pluviation technique is employed in specimen preparation. Test results are discussed in the following sections.

5.1.1 Uncemented Specimens

The testing program for this study involved a total of 14 tests conducted on three different ranges of relative densities (Table 5.1). Six of them are prepared in the range of above 85% relative density, four of them in the range of 65-75% and the remaining in the range of 45-55%. The first few tests with relative density of above 85% are conducted to ensure repeatability and to investigate the appropriate testing procedure. Specimen No. 12 could not be tested because of leaks in the inner and the outer cells. In another sample (No. 14 in Table 5.1), erroneous results were
obtained because the electrical wire connections in the cone tip and friction sleeves were twisted resulting in the loss of tip data while testing.

The details of the tests during consolidation are shown in Table 5.1. Figure 5.1 presents a comparative plot of the $K_o$ values obtained in the calibration chamber testing with respect to the ranges of the values that should be obtained with the Jaky's relationship ($1 - \sin \phi$). The figure shows that $K_o$ values of 0.37 to 0.54 are obtained for above 85% relative density; 0.42 to 0.47 for 65 - 75 percent relative density and 0.35 to 0.62 for 45 - 55% relative density specimens. $K_o$ values obtained from the testing are not always consistent with the Jaky's relationship ($1 - \sin \phi$). Possible reasons are: 1) leaks between the inner and outer cells which may result in lower horizontal stresses thereby lower $K_o$ values, 2) leaks between inner and piston cells result in a horizontal stress value equivalent to the vertical stress (piston cell water pressure), resulting in higher $K_o$ values.

Figures 5.2 to 5.4 show some of the penetration results in the tests conducted. The rest is presented in Appendix C. Tip and friction resistances are depicted in these figures. The tip resistance and sleeve friction resistance readings are taken at around mid-depth (between 30 to 35 cm) in the specimens. In some cases, values that show minor variation along the depth of the specimen are taken as the reading. Table 5.2 summarizes the results.

5.1.2 Cemented Specimens

A total of 20 cemented specimens are prepared and tested. Eleven of the specimens are prepared at 2% cementation and the rest at 1% cementation. Tables 5.3 and 5.4 show the characteristics of the tests and the testing program.

A test with 2% cementation was used as a trial test to check whether vacuum is needed inside the specimen while transferring the specimen into the chamber. Cementation of 2% was not enough to provide the sufficient confinement (cohesion) to avoid collapse of the specimen. Once the outer molds were separated, specimen collapsed (No. 11 in Table 5.4). Hence, it was decided to apply vacuum to all cemented specimens during transportation and before confinement.

Initially, it was planned to carry out the tests with a piezocone which measures excess pore pressures at the cone tip. It is hypothesized that cementation
Figure 5.1: Comparison of $K_o$ values with Jaky’s relationships
Figure 5.2: Cone Penetration Test Results on a Specimen ($D_r = 71.9 \%$; $e_{max} = 0.85$; $e_{min} = 0.56$ and 100 kPa)

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Figure 5.3: Cone Penetration Test Results on a Specimen ($D_r = 68.7\%$; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$ and 200 kPa)
Figure 5.4: Cone Penetration Test Results on a Specimen ($D_r = 71.1\%$; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$ and 300 kPa)
may decrease hydraulic conductivity resulting in undrained conditions during cone penetration. It is envisioned that this excess pore pressure could assist in developing a classification scheme. One test conducted at the center of a specimen (C.C. 2% and $D_r$ 65%) using a dual piezocone of 10 cm² area demonstrated that excess pore pressures are not developed (Figure 5.5). The pore pressure developed is immediately dissipated during penetration implying that the reduction in hydraulic conductivity may not be enough to cause undrained conditions during cone penetration. Consequently, all the tests were conducted with the available miniature friction cone.

The consolidation characteristics of all specimens at cementation levels of 1 and 2% are presented in Tables 5.3 and 5.4. $K_o$ values obtained in these consolidations are compared with Jaky’s relationships (Figure 5.6 and 5.7). Specimen numbered 2 (Table 5.3) has a $K_o$ value of 1 due to the leaks between the inner cell and the piston cell resulting in same stresses. Similar findings as noted in uncemented sand test results are observed.

Cone penetration test results are shown in Table 5.5 and Table 5.6. Cone test results conducted on specimens of 65% relative density and for both cement contents of 1 and 2 percents are shown in Figures 5.8 to 5.13. Rest of them are presented in

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$D_r$ (m)</th>
<th>$H$ (m)</th>
<th>$\sigma_v$ (kN/m²)</th>
<th>$\sigma_{hi/ho}$ (kN/m²)</th>
<th>$K_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>90.0</td>
<td>0.71</td>
<td>100</td>
<td>54.1/51.3</td>
<td>0.54</td>
</tr>
<tr>
<td>2</td>
<td>90.0</td>
<td>0.71</td>
<td>200</td>
<td>87.1/89.3</td>
<td>0.44</td>
</tr>
<tr>
<td>3</td>
<td>88.0</td>
<td>0.66</td>
<td>300</td>
<td>120.4/123.0</td>
<td>0.40</td>
</tr>
<tr>
<td>4</td>
<td>71.9</td>
<td>0.73</td>
<td>96.4</td>
<td>41.8/41.8</td>
<td>0.42</td>
</tr>
<tr>
<td>5</td>
<td>68.7</td>
<td>0.71</td>
<td>204</td>
<td>95.4/95.7</td>
<td>0.48</td>
</tr>
<tr>
<td>6</td>
<td>71.1</td>
<td>0.74</td>
<td>300</td>
<td>121.44/120.3</td>
<td>0.40</td>
</tr>
<tr>
<td>7</td>
<td>48.7</td>
<td>0.75</td>
<td>100</td>
<td>43.8/44.1</td>
<td>0.44</td>
</tr>
<tr>
<td>8</td>
<td>56.4</td>
<td>0.75</td>
<td>200</td>
<td>76.9/70.0</td>
<td>0.35</td>
</tr>
<tr>
<td>9</td>
<td>54.8</td>
<td>0.73</td>
<td>300</td>
<td>185.6/174.2</td>
<td>0.62</td>
</tr>
<tr>
<td>10</td>
<td>86.0</td>
<td>0.71</td>
<td>100</td>
<td>37.1/35.2</td>
<td>0.37</td>
</tr>
<tr>
<td>11</td>
<td>84.0</td>
<td>0.72</td>
<td>100</td>
<td>35.3/34.6</td>
<td>0.35</td>
</tr>
<tr>
<td>12</td>
<td>82.8</td>
<td>0.69</td>
<td>100</td>
<td>0.0/0.0 (leak)</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>65.3</td>
<td>0.71</td>
<td>100</td>
<td>41.3/41.8</td>
<td>0.41</td>
</tr>
<tr>
<td>14</td>
<td>82.3</td>
<td>0.68</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 5.5: Pore Pressures During Piezocone Penetration of Cemented Specimen (C.C. 2 %)
Figure 5.6: Comparison of $K_o$ Values with Jaky's Relationships (C.C. 1 %)
Figure 5.7: Comparison of $K_o$ Values with Jaky's Relationships (C.C. 2 %)

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Table 5.2: Penetration Results (Uncemented)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$D_r$</th>
<th>$\sigma_u$ $(kN/m^2)$</th>
<th>$K_o$</th>
<th>$q_c$ $(ksc)$</th>
<th>$f_s$ $(ksc)$</th>
<th>f.r.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>90.0</td>
<td>100</td>
<td>0.54</td>
<td>122</td>
<td>0.34</td>
<td>0.49</td>
</tr>
<tr>
<td>2</td>
<td>90.0</td>
<td>200</td>
<td>0.44</td>
<td>218</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>3</td>
<td>88.0</td>
<td>300</td>
<td>0.40</td>
<td>350</td>
<td>1.73</td>
<td>0.49</td>
</tr>
<tr>
<td>4</td>
<td>71.9</td>
<td>96</td>
<td>0.42</td>
<td>84</td>
<td>0.20</td>
<td>0.48</td>
</tr>
<tr>
<td>5</td>
<td>68.7</td>
<td>204</td>
<td>0.48</td>
<td>176</td>
<td>0.79</td>
<td>0.45</td>
</tr>
<tr>
<td>6</td>
<td>71.1</td>
<td>300</td>
<td>0.40</td>
<td>203</td>
<td>1.31</td>
<td>0.69</td>
</tr>
<tr>
<td>7</td>
<td>48.7</td>
<td>100</td>
<td>0.44</td>
<td>60</td>
<td>0.20</td>
<td>0.33</td>
</tr>
<tr>
<td>8</td>
<td>56.4</td>
<td>200</td>
<td>0.35</td>
<td>75</td>
<td>0.45</td>
<td>0.60</td>
</tr>
<tr>
<td>9</td>
<td>54.8</td>
<td>300</td>
<td>0.62</td>
<td>115</td>
<td>0.73</td>
<td>0.63</td>
</tr>
<tr>
<td>10</td>
<td>86.0</td>
<td>100</td>
<td>0.37</td>
<td>103</td>
<td>0.30</td>
<td>0.28</td>
</tr>
<tr>
<td>11</td>
<td>84.0</td>
<td>100</td>
<td>0.35</td>
<td>125</td>
<td>0.80</td>
<td>0.64</td>
</tr>
<tr>
<td>12</td>
<td>82.8</td>
<td>100</td>
<td>0.32</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
<td>65.3</td>
<td>100</td>
<td>0.41</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>82.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: $1 ksc = 1 kg/cm^2 = 100 kPa$

Appendix C. These results along with unceemented cone test results are used to study the influence of relative density, cementation and testing location on cone resistance parameters.

5.2 Performance Assessment

In performance assessment of the tests conducted in this calibration chamber using the MQSC, repeatability and accuracy of the results are considered.

5.2.1 Repeatability

In an attempt to evaluate repeatability and precision, tests on unceemented specimens with similar relative densities are compared. The tip resistance profiles for both specimens are presented in the Figure 5.14. There is little difference in the tip resistances recorded in the two tests; however, there is significant variation in friction resistance ($0.34$ to $0.80 kg/cm^2$). The test conducted on 84% relative density specimen is affected by the erroneous sleeve calibrations and also due to mishandling and not cleaning the sleeve portion. This has happened in the beginning phase of
Figure 5.8: Cone Penetration Test Results on a Specimen ($D_r = 68.4\%; e_{\text{max}} = 0.85; e_{\text{min}} = 0.56; C.C. 1\%$ and $100\, kPa$)
Figure 5.9: Cone Penetration Test Results on a Specimen ($D_r = 66.4\%$; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$; C.C. 1% and 200 kPa)

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Figure 5.10: Cone Penetration Test Results on a Specimen ($D_r = 70.2\%$; $e_{\max} = 0.85$; $e_{\min} = 0.56$; C.C. 1\% and 300 kPa)
Figure 5.11: Cone Penetration Test Results on a Specimen ($D_r = 69.6 \%$; $e_{\max} = 0.85$; $e_{\min} = 0.56$; C.C. 2\% and 100 kPa)

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Figure 5.12: Cone Penetration Test Results on a Specimen ($D_r = 69.2\%$; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$; C.C. 2\% and 200 kPa)
Figure 5.13: Cone Penetration Test Results on a Specimen ($D_r = 72.1\%$; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$; C.C. 2\% and 300 kPa)
Table 5.3: Characteristics of Tests (C.C. 1 %)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$D_r$ (m)</th>
<th>Height (m)</th>
<th>$\sigma_v$ ($kN/m^2$)</th>
<th>$\sigma_{hi/ho}$ ($kN/m^2$)</th>
<th>$K_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>86.0</td>
<td>0.71</td>
<td>100</td>
<td>43.5/46.0</td>
<td>0.54</td>
</tr>
<tr>
<td>2*</td>
<td>84.6</td>
<td>0.71</td>
<td>200</td>
<td>200/200</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>89.2</td>
<td>0.70</td>
<td>300</td>
<td>149.4/152.1</td>
<td>0.49</td>
</tr>
<tr>
<td>4</td>
<td>68.4</td>
<td>0.72</td>
<td>100</td>
<td>37.3/34.45</td>
<td>0.37</td>
</tr>
<tr>
<td>5</td>
<td>66.4</td>
<td>0.71</td>
<td>200</td>
<td>125.9/116.2</td>
<td>0.63</td>
</tr>
<tr>
<td>6</td>
<td>70.2</td>
<td>0.74</td>
<td>300</td>
<td>148.15/142.8</td>
<td>0.49</td>
</tr>
<tr>
<td>7</td>
<td>48.8</td>
<td>0.71</td>
<td>100</td>
<td>35.7/30.7</td>
<td>0.36</td>
</tr>
<tr>
<td>8</td>
<td>46.6</td>
<td>0.704</td>
<td>200</td>
<td>82.0/93.1</td>
<td>0.41</td>
</tr>
<tr>
<td>9</td>
<td>53.2</td>
<td>0.72</td>
<td>300</td>
<td>171.6/173.2</td>
<td>0.57</td>
</tr>
</tbody>
</table>

Note: * - Leak Between Piston and Inner Cell

this testing. These mistakes are rectified in the subsequent tests by recalibrating the cone and cleaning the sleeve portion prior to each test. Another test conducted on a similar specimen for studying the location influence yielded a value of 0.30 kg/cm² which is close to 0.34 kg/cm². Tests conducted for studying the influence of boundary condition (presented in the next chapter) also showed that both tip and friction resistances are repeatable.

5.2.2 Accuracy

Accuracy requires testing a material with known tip resistance and sleeve friction values in the present test setup. There is not yet such defined material in calibration chamber testing. Furthermore, a round robin testing of a specific sand in different calibration chambers and under selected boundary conditions is not yet available. However, it is possible to compare the present test results with the previous results reported on the same sand. Such a comparison is presented in the next chapter.

5.3 Influence of Testing Variables on Cone Test Results

5.3.1 Uncemented Specimen Results

Figures 5.15 and 5.16 shows the tip and friction resistance versus effective vertical confining stress at different relative densities of uncemented specimens. Both tip and friction resistances increase with the effective vertical stress. Higher vertical
Figure 5.14: Repeatability of the Results
**Table 5.4: Characteristics of Tests (C.C. 2 %)**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$D_r$</th>
<th>height (m)</th>
<th>$\sigma_v$ (kN/m$^2$)</th>
<th>$\sigma_{hi/ho}$ (kN/m$^2$)</th>
<th>$K_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>88.2</td>
<td>0.70</td>
<td>100</td>
<td>54.12/51.3</td>
<td>0.54</td>
</tr>
<tr>
<td>2</td>
<td>86.3</td>
<td>0.71</td>
<td>200</td>
<td>87.1/89.3</td>
<td>0.44</td>
</tr>
<tr>
<td>3</td>
<td>84.2</td>
<td>0.70</td>
<td>300</td>
<td>120.4/123.02</td>
<td>0.40</td>
</tr>
<tr>
<td>4</td>
<td>69.6</td>
<td>0.71</td>
<td>100</td>
<td>41.8/41.8</td>
<td>0.42</td>
</tr>
<tr>
<td>5</td>
<td>69.2</td>
<td>0.71</td>
<td>200</td>
<td>95.4/95.7</td>
<td>0.48</td>
</tr>
<tr>
<td>6</td>
<td>72.1</td>
<td>0.72</td>
<td>300</td>
<td>121.44/120.3</td>
<td>0.40</td>
</tr>
<tr>
<td>7</td>
<td>47.2</td>
<td>0.69</td>
<td>100</td>
<td>43.84/44.1</td>
<td>0.44</td>
</tr>
<tr>
<td>8</td>
<td>54.4</td>
<td>0.73</td>
<td>200</td>
<td>76.9/70.0</td>
<td>0.35</td>
</tr>
<tr>
<td>9</td>
<td>52.0</td>
<td>0.71</td>
<td>300</td>
<td>185.6/174.2</td>
<td>0.62</td>
</tr>
<tr>
<td>10</td>
<td>86.2</td>
<td>0.72</td>
<td>65</td>
<td>65/65</td>
<td>1.00</td>
</tr>
<tr>
<td>11</td>
<td>84.4</td>
<td>0.71</td>
<td>-</td>
<td>-</td>
<td>(collapse)</td>
</tr>
</tbody>
</table>

Stress in the specimen cause the specimen to offer more resistance to the penetration of the cone, resulting in higher tip resistances. The effect of relative density on tip resistance can also be deduced from the same figure. The higher the relative density, the closer the sand packing and the contact points between the grains which result in higher tip resistances. Similar observations are noted in the case of friction resistance.

Most of the tests are conducted at location A, the center of the specimen unless specified. In an attempt to study the influence of location, an uncedemented specimen of relative density of 86 % is prepared. The test was conducted at location B which is closer to the edge. The test results are compared with test results on a specimen of 90 percent relative density which are conducted at location A. Both the specimens are consolidated under an effective vertical stress of 100 kPa and tested under boundary condition 3.

Figure 5.17 compares the results conducted at these two locations. It is interesting to note that location has minor influence on the final tip and friction resistances; however, at the shallow penetration depths along the specimen, there is a marked difference. The variation is negligible at higher penetration depths. These minor differences are possibly due to both relative density and also coupling of the tip resistance with sleeve friction. The soil pluviating near the edge of the sieve may result in a different density than in the middle. Boundary effects may have also led
Table 5.5: Penetration Results (C.C. 1 %)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$D_r$</th>
<th>$\sigma_v$ ($kN/m^2$)</th>
<th>$K_o$</th>
<th>$q_c$ ($ksc$)</th>
<th>$f_s$ ($ksc$)</th>
<th>f.r.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>86.0</td>
<td>100</td>
<td>0.54</td>
<td>168.0</td>
<td>0.95</td>
<td>0.56</td>
</tr>
<tr>
<td>2</td>
<td>84.6</td>
<td>200</td>
<td>1.00</td>
<td>254.0</td>
<td>1.55</td>
<td>0.64</td>
</tr>
<tr>
<td>3</td>
<td>89.2</td>
<td>300</td>
<td>0.49</td>
<td>338.0</td>
<td>1.98</td>
<td>0.58</td>
</tr>
<tr>
<td>4</td>
<td>68.4</td>
<td>100</td>
<td>0.37</td>
<td>131.0</td>
<td>0.74</td>
<td>0.56</td>
</tr>
<tr>
<td>5</td>
<td>66.4</td>
<td>200</td>
<td>0.63</td>
<td>191.0</td>
<td>0.86</td>
<td>0.45</td>
</tr>
<tr>
<td>6</td>
<td>70.2</td>
<td>300</td>
<td>0.49</td>
<td>246.0</td>
<td>1.21</td>
<td>0.49</td>
</tr>
<tr>
<td>7</td>
<td>48.8</td>
<td>100</td>
<td>0.36</td>
<td>68.0</td>
<td>0.20</td>
<td>0.29</td>
</tr>
<tr>
<td>8</td>
<td>46.6</td>
<td>200</td>
<td>0.41</td>
<td>97.0</td>
<td>0.72</td>
<td>0.72</td>
</tr>
<tr>
<td>9</td>
<td>53.2</td>
<td>300</td>
<td>0.57</td>
<td>104.0</td>
<td>1.00</td>
<td>0.97</td>
</tr>
</tbody>
</table>

Note: $1ksc - 1kg/cm^2 = 100 kPa$

Table 5.6: Penetration Results (C.C. 2 %)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$D_r$</th>
<th>$\sigma_v$ ($kN/m^2$)</th>
<th>$K_o$</th>
<th>$q_c$ ($ksc$)</th>
<th>$f_s$ ($ksc$)</th>
<th>f.r.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>88.2</td>
<td>100</td>
<td>0.54</td>
<td>202.0</td>
<td>1.10</td>
<td>0.54</td>
</tr>
<tr>
<td>2</td>
<td>86.3</td>
<td>200</td>
<td>0.44</td>
<td>295.0</td>
<td>1.75</td>
<td>0.59</td>
</tr>
<tr>
<td>3 *</td>
<td>84.2</td>
<td>300</td>
<td>0.40</td>
<td>350.0</td>
<td>2.11</td>
<td>0.60</td>
</tr>
<tr>
<td>4</td>
<td>69.6</td>
<td>100</td>
<td>0.42</td>
<td>143.0</td>
<td>0.80</td>
<td>0.55</td>
</tr>
<tr>
<td>5</td>
<td>69.2</td>
<td>200</td>
<td>0.48</td>
<td>220.0</td>
<td>1.20</td>
<td>0.54</td>
</tr>
<tr>
<td>6</td>
<td>72.1</td>
<td>300</td>
<td>0.40</td>
<td>307.0</td>
<td>1.31</td>
<td>0.43</td>
</tr>
<tr>
<td>7</td>
<td>47.2</td>
<td>100</td>
<td>0.44</td>
<td>80.0</td>
<td>0.40</td>
<td>0.50</td>
</tr>
<tr>
<td>8</td>
<td>54.4</td>
<td>200</td>
<td>0.35</td>
<td>120.0</td>
<td>0.75</td>
<td>0.62</td>
</tr>
<tr>
<td>9</td>
<td>52.0</td>
<td>300</td>
<td>0.62</td>
<td>150.0</td>
<td>1.01</td>
<td>0.67</td>
</tr>
<tr>
<td>10</td>
<td>86.2</td>
<td>65</td>
<td>1.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: $1 ksc - 1 kg/cm^2 = 100 kPa$

* - Test Stopped
Figure 5.15: Tip Resistance vs Effective Vertical Stress for Various Relative Densities

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Figure 5.16: Friction Resistance vs Effective Vertical Stress for Various Relative Densities

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Figure 5.17: Influence of Location on Test Results
to this difference in sleeve friction. At a confining pressure of 100 \textit{kPa}, a change of 2 degrees in the internal friction angle will result in approximately a difference of 25 \textit{kPa} in sleeve friction (0.25 \textit{kg/cm}^2).

5.3.2 Cemented Specimens

Figures 5.18 and 5.19 present the influence of relative density on tip resistance and friction resistance of cemented specimens (1 \%). Similar to uncemented sands, an increase in relative density results in an increase in both tip and friction resistances.

Figures 5.20 and 5.21 compare the tip and friction resistances of cemented and uncemented sands. Cemented sands induce higher tip and friction resistances, attributed to the cementation bonding. These bonds bring about a cohesive property to the sand which significantly increases its strength. The higher the strength of these cemented sands the higher will be the tip and friction resistances.

The increase in cementation results in a drop in friction ratio (Table 5.5 and 5.6) due to the more pronounced increase in tip resistance.

5.4 Summary

A total of thirty four (14 uncemented and 20 cemented specimens) calibration chamber tests were conducted using a miniature cone penetrometer. These results will be used in the subsequent chapters in preparing empirical and semi-empirical correlations.

Piezocone penetration test on a specimen of 2 \% cementation showed that there was no excess pore pressure measurements. This implies drained conditions still prevail in cemented specimens used in this study and the decrease in hydraulic conductivity is not sufficient to cause undrained conditions during cone penetration.

Tip and friction resistances increase with effective vertical and horizontal confining stresses and also with relative densities. The tip and sleeve friction resistance are found to be repeatable. Testing location has minor influence on tip resistance.

Increase in cementation results in an increase in tip and friction resistance and a decrease in friction ratio. The increase is due to evolution of cementation bonds between grains, resulting in cohesion which in turn increases the strength of the specimens. This increase in strength offers more resistance to the penetrating cone,
Figure 5.18: Influence of Relative Density on Tip Resistance of Cemented Specimen (1 %)

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Figure 5.19: Influence of Relative Density on Friction Resistance of Cemented Specimen (1 %)

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Figure 5.20: Influence of Cementation on Tip Resistance

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Figure 5.21: Influence of Cementation on Friction Resistance

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hence higher tip and friction values are observed. Increase in tip resistance is more pronounced than friction resistance, resulting in lower friction ratios.
Chapter 6

FACTORS INFLUENCING TEST RESULTS

6.1 Introduction

At the present state of the art and despite significant achievements in calibration chamber research, there are number of concerns associated with engineering interpretation of insitu tests and the use of chambers in developing an understanding of insitu testing devices (Ghionna and Jamiołkowski, 1991). Generally, chamber tests are conducted on specimens of freshly reconstituted specimens whose fabric is different from those of the natural soil deposits. The structure of a natural deposit is highly developed as a result of cementation, drained creep and diagenesis (Ghionna and Jamiołkowski, 1991). Hence interpretations developed based only on tests conducted on clean sands may not be valid for certain natural deposits, specifically when there is cementation.

Another concern arises when the various factors that influence chamber test results are considered; soil characteristics like grain size, compressibility and relative density and also chamber parameters such as diameter ratio, boundary conditions and the size of the cone significantly affect the test results. Several studies have been attempted to understand the above problems without reaching definitive conclusions (Eid, 1987; Ghionna and Jamiołkowski, 1991).

Present study was conducted on both cemented and uncemented sand specimens. A diameter ratio of 42 was achieved by using the miniature cone (MQSC). These results are used in providing an assessment of the effect of different variables. Uncemented sand results are first compared with various relative density-cone resistance curves. A chart for estimating the relative density for Monterey No. 0/30 sand is also presented. Various factors affecting chamber results are evaluated. Data from seven different calibration chamber investigations along with present test results are used in this study.

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6.2 Relative Density

The test results of this study are compared with various relative density charts proposed by different investigators. The findings and observations from these comparisons are discussed in the following sections.

6.2.1 Comparison With Results Reported by Villet and Mitchell (1981)

In an attempt to investigate the variability in results from one calibration chamber to another, a preliminary study is conducted. Villet and Mitchell (1981), Schmertmann (1978) and Baldi (1981) have reported extensive studies with the calibration chamber. Comparisons are made with V & M's results since they have used Monterey No. 0 sand. The results of this study at relative densities of 45 - 55, 65 - 75 and above 80 % are plotted on the V & M's relative density prediction chart for comparison purposes (Figure 6.1).

V & M tests are conducted in a pressure chamber of 80 cm (32 in.) in height and 76 cm (30 in.) in diameter. A cone penetrometer of 10 cm² tip area (3.56 cm diameter) and a base apex angle of 60 degrees is used. Their diameter ratio is 20. Specimens are consolidated under $K_o$ conditions. A fixed $K_o$ value of 0.5 was used for their tests. Horizontal stress of magnitude equal to half of the vertical stress was manually applied to the specimen, thereby rendering a $K_o$ value of 0.5.

The results obtained in this study are systematically lower than those reported by V & M. The author offers the following reasons for this difference:

1. Tests by V & M are conducted under constant stress boundary conditions. The $K_o$ values used by Villet and Mitchell were around 0.5, substantially larger than the $K_o$ values obtained in this study. Note that horizontal stresses (equivalent of $K_o$ conditions) are manually applied in V & M study, unlike the ones obtained in the present study, which are measured for zero lateral strain conditions.

2. V & M used a conventional cone of 3.56 cm in diameter while this study used a 1.27 cm diameter cone. It is not well established, how this factor will individually affect the results. Studies conducted by Eid (1987) show that there is a scale effect in the cone test results based on the size of the cone used. Therefore, the results from other chambers cannot be used as a yardstick in
Figure 6.1: Comparisons with V and M's Results

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evaluating the variations in the present study due to the differences in testing and chamber parameters.

3. The diameter ratio in Villet and Mitchell is 20, significantly smaller than the diameter ratio of 42 used in this study. In dense sands, $q_c$ increases substantially with the decrease in diameter ratio. Therefore, the results reported by V & M may have been significantly influenced by the boundaries.

6.2.2 Comparisons With Results Reported for Other Sands

Comprehensive calibration chamber studies with a sand are also reported by Schmertmann (1978) and Baldi (1981). The sands used in these studies are different and are classified as low to medium compressible. Schmertmann’s results are stated to be valid for the Fugro cone in normally consolidated, saturated, recent, unce­mented, fine SP sands. The sand is defined to be subrounded to angular. Baldi’s results are valid for Ticino sand (Italy). This sand is sub-angular to angular in shape. The $K_o$ values of Baldi’s tests range from 0.37 to 0.46. Dry pluviation method was used in both studies for specimen preparation. Both studies were conducted at a diameter ratio of 34.

The comparisons are shown in Figures 6.2 and 6.3. The uncemented test results obtained in this study are plotted on Baldi’s and Schmertmann’s results from the relative density-tip resistance-vertical stress curves for comparison purposes. It is noted that the results reported by Schmertmann deviate more at higher confining stresses (greater than 150 kPa). The boundary conditions, the physical characteristics of the sand in Schmertmann’s results are significantly different from those of this study. In addition, the lower diameter ratio (34 versus 42 used in this study) also influences these results. The comparisons with Baldi’s chart yield the following conclusions. In case of loose specimens, the results seem to be closer, however there is some variation at higher densities. It is interesting to note that in the case of lower densities, the results are not as affected as those at higher densities by the diameter ratio, boundary conditions and probably the grain size and shapes. Diameter ratio of Baldi’s test results are lower than the present test results (34 versus 42) and also these tests are conducted at constant stress boundary conditions (BC1). These vari­ations, according to previous investigations (Been et al., 1986) will influence cone
Figure 6.2: Comparisons with Baldi's Chart
Figure 6.3: Comparisons with Schmertmann’s Chart

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test results at higher densities. Another factor that might have affected the results is the shape of the grain. Ticino sand of Baldi’s study is more angular than Monterey No. 0/30 sand which is subrounded to subangular in shape.

The results obtained indicate that it is quite difficult to compare the penetration results obtained in one chamber with one type of sand with another due to the effect of boundary conditions, diameter ratio, differences in grain shape, size and soil composition and specimen preparation. The data obtained in this study seems to be within the variability of the results reported by the two previous studies (Baldi, 1981; Schmertmann, 1978).

It is clear that tip resistance is influenced by several factors. Sands of different grain size and shape behave differently when tested under identical conditions. Additional testing variables like chamber size and boundary conditions will influence the results. A further attempt is made to collect the penetration data on the sands and study the influence of the above mentioned factors.

6.3 Factors Influencing Penetration Resistance in Calibration Chamber

The cone results reported by different investigators are compiled in Appendix B. These results are used to conduct an assessment of the complex problems facing calibration chamber testing. In an attempt to bring a formalism to the effect of different calibration chamber test results, available data are compiled in groups. In analysis of the effect of one variable, care was given to select the data which were obtained under similar conditions and compositional variables. For example, data obtained from the sands of similar size and shape, compressibility and tested under same boundary condition are used to analyze the influence of chamber size effects.

6.3.1 Chamber Size

Since the early eighties, the influence of the chamber dimensions and configuration on penetration results are regarded as the fundamental variables affecting the
results (Ghionna and Jamiolkowski, 1991). Holden (1971) demonstrated that the
camber size significantly affects the results in dense sands.

The effect of chamber size on the measured results is later addressed by other
investigators (Parkin and Lunne, 1982; Jamiolkowski et.al., 1985; Bellotti et. al.,
that a diameter ratio of 50 will be needed when dense sands are tested. He also
found out that for loose sands, even at low diameter ratios (around 21), tip resistance
results are independent of boundary conditions. Although, a systematic study is not
yet available, higher diameter ratio will be required testing dense sands in calibration
chambers. This will be specifically true at lower confining pressures when the sand
could dilate during shearing.

Mayne (1991) observed that the cone data tested under flexible wall calibration
chambers are less than they would be in an infinite medium. Mayne (1991) suggests
that yielding which occurs while testing under flexible wall calibration chambers may
be the reason for this decrease (Mayne, 1991). The author notes that cementation
may also be another factor. Mayne (1991) further suggests that tip resistances
measured in chambers need to be corrected for the chamber effects. Mayne (1991)
proposes a correction factor that takes care of such chamber effects. This factor was
derived based upon examination of six available test data on sands and is purely of
empirical nature.

\[ q_{c, corr} = q_{c, meas} \left( \frac{D_c}{D_{05}} - 1 \right) \left( \frac{D_{05}}{B} \right)^{0.5} \]  

The correction factor, defined as the ratio of corrected to measured tip resistances
ranges from 0.4 to 0.9 for diameter ratios of 15 to 40 and for relative densities of
20 to 100 %. For denser specimens (above 80 %), the correction factors are 0.4 to
0.6 which implies that the measured resistance need to be almost doubled to get the
corrected value.

The accuracy of these results is debatable and the validity of the above equation
raises several following concerns. The results used in the above study are not nor-
malized with respect to horizontal and vertical stresses at which they are obtained.
Instead, they are reported as values for an average effective vertical stress of 150 kPa
and a \( K_o \) value of 0.43, thereby neglecting the influence of these stresses on the tip
resistances.
This type of analysis clearly ignores the influence of effective vertical and horizontal stresses on the results. Since, tip resistance increases with both horizontal and vertical effective stresses, the tip resistance of each test needs to be normalized with both vertical and horizontal stresses. These normalized values have to be used to study the influence of chamber size effects. The influence of horizontal stress on $q_c$ is similar to vertical stress (Figure 6.4). Hence, normalization has to be done by taking into account the horizontal effective stress. Influence of horizontal and vertical effective stresses is incorporated by normalizing with respect to octahedral stress, $p_{oct}$. Tip resistance is normalized as the ratio of $(q_c - p_{oct})$ to $\sigma_{atm}$, where $q_c$ is the measured cone resistance; $p_{oct}$ is total octahedral stress $(\frac{\sigma_1 + 2\sigma_3}{3})$ and $\sigma_{atm}$ is atmospheric pressure. Effective octahedral stress, $p'_{oct}$ is normalized by taking the ratio of $p'_{oct}$ to $\sigma_{atm}$. Then, those normalized values can be correlated in the following form,

$$\frac{q_c - p_{oct}}{\sigma_{atm}} = \alpha_o \left( \frac{p'_{oct}}{\sigma_{atm}} \right)^{n_o} \quad (6.2)$$

where $\alpha_o$ and $n_o$ are the parameters that include the influence of relative density, grain size, shape, fabric and composition (soil parameters), diameter ratio and boundary conditions (chamber parameters). The subscript 'o' denotes the octahedral stress normalization. The following procedure is used in evaluating the effect of different parameters on $\alpha_o$ and $n_o$.

1. Eight different results reported for sands, including the present test data are used. Physical properties of these sands and the chamber parameters in which they are tested are presented previously in Chapter 2. The tip resistance values are normalized with respect to $\frac{q_c - p_{oct}}{\sigma_{atm}}$.

2. Similarly, normalized effective octahedral stress, $\left( \frac{p'_{oct}}{\sigma_{atm}} \right)$ values are calculated in all the tests,

3. Logarithm of these normalized values are computed,

4. The logarithmic values are correlated with each other,

5. A best fit straight line is passed through each set of relative density range,
Figure 6.4: Tip Resistance Versus Horizontal Effective Stress (Present Study)
6. The y-axis intercept, \( \log \alpha_o \) and the slope of the line, \( n \), are calculated, and the values \( \alpha \) (or \( \alpha_o \)) and \( n \) (or \( n_o \)) thus obtained are tabulated in Table 5.3.

Figures 6.5, 6.6, 6.7 and 6.8 show such plots for various investigations. From each figure, \( \alpha \) and \( n \) are estimated. These are reported along with other details of these studies in Table 6.1. The mean and standard deviation of the \( n \) values are calculated. The mean value varies between 0.55 to 0.92 with a standard deviation value of 0.03 to 0.22. Most of these investigations have low standard deviation values which implies that \( n \) may be regarded constant at all densities for a particular type of sand. The \( n \) value seems to be a property of the soil at a state at which there is no volume change (critical state). This may be the reason for \( n \) being independent of relative density. After calculating the average and standard deviation values of \( n \), lines are passed through the same data points in these figures at slopes equivalent to \( n_{\text{mean}} \pm 1 \) standard deviation. The new \( \alpha \) values (denoted as \( \alpha_1 \) and \( \alpha_2 \) in Table 6.1) are determined and the average of these values are taken as the corrected \( \alpha \) value. These corrected values are determined for each density in all investigations and they are reported in the same table.

Influence of chamber size is studied by plotting the mean \( \alpha \) values on the y-axis and diameter ratio on the x-axis (Figure 6.9). For each set of relative density, a best fit curve is obtained. This figure signifies the importance of chamber size dimensions on chamber test results. The influence of chamber size (diameter ratio) on \( \alpha \) is not as evident at lower densities as in the case of higher densities. The diameter ratio influences the results on denser specimens and influence decreases with a decrease in relative density. Hence, it is recommended to use higher diameter ratios for calibration chamber tests on dense sands. A diameter ratio of above 40 is needed to reduce the chamber size effects on dense specimens (above 80 %). For the specimens of above 90 % relative density, diameter ratio of above 50 is recommended.

### 6.3.2 Compressibility and Crushability of the Sand Tested

Bellotti et al. (1991) shows that significant crushing takes place in cone penetration tests performed in calibration chambers which in turn affect the CPT results. This problem will be more evident, if the sand used in the chamber has low com-
Table 6.1: The $\alpha$ and $n$ for Various Investigations

<table>
<thead>
<tr>
<th>Study (BC)</th>
<th>Sand $d_{50}$ (mm)</th>
<th>$D_r$</th>
<th>D.R.</th>
<th>$\alpha$</th>
<th>$n$</th>
<th>$n_m (\sigma)$</th>
<th>$\alpha_1 (n_m+\sigma)$</th>
<th>$\alpha_2 (n_m-\sigma)$</th>
<th>$\alpha (\alpha_1+\alpha_2)/2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>EID (1987) (BC1)</td>
<td>0.45</td>
<td>25</td>
<td>17</td>
<td>98</td>
<td>0.90</td>
<td>97.9</td>
<td>94.0</td>
<td>95.9</td>
<td></td>
</tr>
<tr>
<td>SR-SA</td>
<td>25</td>
<td>42</td>
<td>70.8</td>
<td>0.79</td>
<td>0.75</td>
<td>71.8</td>
<td>69.2</td>
<td>70.5</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>34</td>
<td>141</td>
<td>0.48</td>
<td>(0.16)</td>
<td>128.7</td>
<td>137.9</td>
<td>133.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>42</td>
<td>229</td>
<td>0.72</td>
<td>219.7</td>
<td>235.5</td>
<td>227.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>64</td>
<td>302</td>
<td>0.87</td>
<td>298.9</td>
<td>320.4</td>
<td>309.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Baldi (1981) (BC1)</td>
<td>0.39</td>
<td>45</td>
<td>34.2</td>
<td>88.7</td>
<td>0.57</td>
<td>88.7</td>
<td>88.7</td>
<td>88.7</td>
<td></td>
</tr>
<tr>
<td>SA-A</td>
<td>75</td>
<td>34.2</td>
<td>166.3</td>
<td>0.57</td>
<td>0.55</td>
<td>166.3</td>
<td>166.3</td>
<td>166.3</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>34.2</td>
<td>253.5</td>
<td>0.52</td>
<td>(0.03)</td>
<td>250.2</td>
<td>253.5</td>
<td>251.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fior (1991) (BC1)</td>
<td>0.16</td>
<td>45</td>
<td>60</td>
<td>70.5</td>
<td>0.54</td>
<td>92.8</td>
<td>71.3</td>
<td>82.1</td>
<td></td>
</tr>
<tr>
<td>SA</td>
<td>70</td>
<td>33.4</td>
<td>190.5</td>
<td>0.83</td>
<td>0.78</td>
<td>195.0</td>
<td>183.5</td>
<td>189.3</td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>60</td>
<td>242.0</td>
<td>0.98</td>
<td>(0.22)</td>
<td>242.0</td>
<td>235.9</td>
<td>238.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pupp (1993) (BC3)</td>
<td>0.45</td>
<td>45</td>
<td>42</td>
<td>61.7</td>
<td>0.70</td>
<td>59.1</td>
<td>61.5</td>
<td>60.3</td>
<td></td>
</tr>
<tr>
<td>SR-SA</td>
<td>65</td>
<td>42</td>
<td>128.8</td>
<td>0.88</td>
<td>0.84</td>
<td>127.5</td>
<td>131.3</td>
<td>129.4</td>
<td></td>
</tr>
<tr>
<td>87</td>
<td>42</td>
<td>186.2</td>
<td>0.95</td>
<td>(0.12)</td>
<td>186.4</td>
<td>180.6</td>
<td>183.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VM (1981) (BC1)</td>
<td>0.45</td>
<td>30</td>
<td>20</td>
<td>70.8</td>
<td>0.95</td>
<td>70.4</td>
<td>73.5</td>
<td>71.9</td>
<td></td>
</tr>
<tr>
<td>SR-SA</td>
<td>60</td>
<td>20</td>
<td>121.3</td>
<td>0.89</td>
<td>(0.04)</td>
<td>119.7</td>
<td>121.6</td>
<td>120.6</td>
<td></td>
</tr>
<tr>
<td>Lhuer (1976) (BC1)</td>
<td>0.30</td>
<td>30</td>
<td>34.2</td>
<td>37.1</td>
<td>0.79</td>
<td>37.1</td>
<td>36.7</td>
<td>36.8</td>
<td></td>
</tr>
<tr>
<td>65</td>
<td>34.2</td>
<td>125.8</td>
<td>0.67</td>
<td>0.72</td>
<td>126.7</td>
<td>125.7</td>
<td>126.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>34.2</td>
<td>154.8</td>
<td>0.71</td>
<td>(0.06)</td>
<td>156.3</td>
<td>153.7</td>
<td>155.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Harm (1976) (BC1)</td>
<td>0.30</td>
<td>34.2</td>
<td>40.6</td>
<td>0.82</td>
<td>40.7</td>
<td>39.6</td>
<td>40.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA</td>
<td>65</td>
<td>34.2</td>
<td>138.4</td>
<td>0.80</td>
<td>0.76</td>
<td>139.5</td>
<td>134.6</td>
<td>137.1</td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>34.2</td>
<td>200.9</td>
<td>0.68</td>
<td>(0.07)</td>
<td>211.6</td>
<td>201.6</td>
<td>206.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nutt (1991)</td>
<td>0.24</td>
<td>30</td>
<td>34.2</td>
<td>39</td>
<td>0.66</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: $\sigma$ - Standard Deviation

$n_m$ - Average $n$; SR - Subrounded; SA - Subangular; A - Angular
Figure 6.5: Normalized Tip Resistance Versus Normalized Effective Octahedral Stresses
Figure 6.6: Normalized Tip Resistance Versus Normalized Effective Octahedral Stresses
Figure 6.7: Normalized Tip Resistance Versus Normalized Effective Octahedral Stresses
Figure 6.8: Normalized Tip Resistance Versus Normalized Effective Octahedral Stresses

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Figure 6.9: Influence of Diameter Ratio on $\alpha$ for Various Densities
pressibility. The crushing of the sand around the cone may have more pronounced effect on friction resistance as this value will be measured after crushing takes place. Several investigators noted the importance of particle crushing which results in an increase in compressibility (Bellotti et al., 1991). The other important effect is a more pronounced curvature of the failure envelope up to an ultimate value of friction angle which would be unaffected by further crushing. The combined effect of all these factors is a reduction in tip and friction resistance.

Bellotti et al. (1991) tested four different sands and evaluated their crushability indexes from the grain size distributions of the sands tested at different pressures in one-dimensional compression tests. Crushability indexes are proposed in various forms by various investigators (Bellotti et al., 1991). These indexes express the amount of crushing of the aggregates that took place during cone penetration. This is done by calculating the ratio of the amount of the tested sand passing in a particular standard sieve to that of untested sand. Crushing measurements in CPT exhibited a well defined correlation between crushing amount and cone resistance (Bellotti et al., 1991). These correlations are independent of consolidation stress, relative density and overconsolidation ratio (Bellotti et al., 1991).

The sand used in the present study can be regarded incompressible even though it contains some mica (less than 1 %). The present results are compared with the results obtained in Ticino sand to assess the influence of compressibility. Since the crushing of sand particles is observed in dense specimens (above 84 %), only dense specimens are considered for this analysis.

The sand around a radial distance of 3.5 cm around the cone was collected subsequent to penetration and a grain size distribution analysis was conducted. A comparison of the grain size distributions is presented in Figure 6.10. The sand tested under 200 kPa and 300 kPa shows some variation in gradation. The results also demonstrate that there is substantial crushing of the material retained on ASTM sieves No. 30, 40 and 50.

The author also observed crushed sand powder all along the penetration profile. The sand collected from under the cone was passed through an ASTM No. 200 sieve. A coefficient named as crushability under cone (CUC) is used to measure the crushability. CUC is defined as the ratio of the weight of sand passed through a No. 200 sieve of a penetration tested sand per 1000 gm to that of an untested sand.
Figure 6.10: Grainsize Distributions of the Crushed Sand Around the Cone
A CUC value of less than 5 implies sand is moderately compressible, between 5 and 10 is known as of medium compressibility and above 10 it is termed as highly compressible. CUC depends upon the influence zone from which the penetrated sand is collected. In the present study, sand is collected from a zone of three times the penetrometer diameter around the tested location. The CUC values obtained from the sand collected in tests under vertical consolidation stress of 200 and 300 kPa are 2.8 and 3.8, respectively. The low CUC values suggest that the sand has moderate compressibility; therefore the crushing around the cone will decay significantly as one moves away from the cone.

The tip resistances measured under these tests were plotted in the Figure 6.11 along with the data of Quiou sand, a subrounded to subangular sand (Almeida et al., 1991). These results, although few, suggest that crushing does not significantly affect the tip resistance around the cone in Monterey No. 0/30 sand. However, there is significant crushing taking place in the case of Quiou sand. A possible reason for this difference is the size of the cone (3.57 cm cone was used in the study on Quiou sand).

6.3.3 Boundary Conditions

The four traditional boundary conditions used in chamber testing were described earlier. Parkin and Lunne (1982) state that the penetration resistance in the field will lie between a CC test with a zero lateral strain boundary (BC3) and CC test with a constant lateral stress boundary (BC1). The zero strain boundary condition overestimates \( q_c \), as higher stresses will exist at chamber boundary than in the field at an equivalent distance from the cone. Conversely, the constant stress condition underestimates the \( q_c \) as higher stresses may develop in the field at an equal distance from the cone (Been et al., 1988).

Figure 6.12 shows the zero strain and constant stress data converging when the chamber diameter is greater than 50 times the cone diameter. Boundary conditions have little influence on the results in loose sands. The present test results which are conducted under boundary condition 3 are used to discuss the boundary condition effects. The results from two other studies, Villet and Mitchell (1981) and Eid (1981) are also used. These tests were conducted under boundary condition 1. The change
Figure 6.11: Influence of Crushing on Tip Resistance
in $\alpha$ with relative density is presented for these two studies in Figure 6.13. BC1 induces higher $\alpha$ values than BC3; tip resistances being higher in BC3. Higher octahedral stresses are observed in BC3 than in BC1. Zero lateral strain boundary induces higher lateral stresses resulting in higher octahedral stresses thereby reducing $\alpha$ values. These investigations have different diameter ratios and their influence on the above results are not accounted for in the above discussion.

The influence of boundary condition on test results is investigated by conducting cone tests on two identical cemented specimens subjected to different boundary conditions; namely the constant stress (BC1) and zero lateral strain (BC3) conditions. Specimens are prepared at 45 to 55 % relative density range at a consolidation pressure of $300 \, kPa$. Tip and friction resistance results are shown in Figure 6.14. The two tests produce similar results, verifying the indifference in tip resistance to boundary conditions at higher diameter ratios (42) in the present tests. The larger diameter ratio implies the cone test results are least affected by the boundary conditions. This implies that the testing rendered results similar to that of field conditions of semi-infinite medium.

6.3.4 Influence of Soil Particle Size and Shape

It is an established fact that sands from different locations will not behave identically when tested under similar conditions. This is attributed to the size and shape of the particles which lead to changes in dilational characteristics. It was suggested by Ghionna and Jamiolkowski (1991) that significant studies have to be done to understand the effect of these parameters on penetration. An attempt is made in this section to evaluate the various test results conducted on different types of sands. Figure 6.15 was prepared by plotting $n$ on the y-axis and the ratio of cone diameter to $d_{50}$ of the aggregate on the x-axis. The $n$ value varies between 0.60 to 0.95 without exhibiting any trend.

The $n$ value seems to be more influenced by the size of the aggregates and the diameter of the cone. An increase in the ratio of the diameter of cone to $d_{50}$ results in an increase of $n$. It can also be concluded that the $n$ value is not influenced by the relative density and the stresses that the specimen is subjected to. The effect of grain shape on $n$ can not be deduced since sands used in this analysis are
Figure 6.12: Boundary Condition Influence on CC Results (Parkin, 1985)
Figure 6.13: Boundary Condition Influence on CC Results

- □□□□ D/d = 42, Monterey No. 0/30
- □□□□ D/d = 40, Ticino Sand
- □□□□ BC1, This Study
- □□□□ BC3, Baldi (1981)
Figure 6.14: Influence of Boundary Condition on Cone Test Results

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subrounded-subangular to angular in shape.

### 6.3.5 Cementation

Few studies in the literature reported testing cemented sands (Rad and Tumay, 1986; Akili and Nabil, 1988). The available studies were conducted in rigid chambers at low confining pressures (less than 5 kPa). In order to study the influence of cementation, tests have been conducted on artificially cemented specimens at various confining pressures. These results have been reviewed in Chapter 5.

The influence of cementation is investigated by determining $\alpha$ and $n$ values in the test results obtained in cemented specimens (Table 6.2). Figures 6.16 and 6.17 present the normalized results at both cementation levels. The $\alpha$ and $n$ are calculated from these figures and are shown in Table 6.2.

These results are plotted comparatively with those of un cemented results in Figures 6.18. The $\alpha$ value clearly increases with cementation at each relative density. This increase is due to the increase in tip resistance with cementation. However, cementation influence on $n$ is not clear except that $n$ decreases with cementation at each relative density. In brief, cementation reduces the relationship between normalized tip and octahedral stresses into non-linear form. The following non-linear relationship is obtained between cement content and $n$ (Figure 6.18).

$$n = 0.84 - 0.90 \ln \left(1 + 0.56 (C.C.)^{0.13}\right)$$ (6.3)

This relationship demonstrates that results will be strongly affected by slight increases in cementation levels. The author notes that although there is an increase
Figure 6.15: Influence of Cone Diameter to \(d_{50}\) ratio on \(n\)
Figure 6.16: Normalized Results on Cemented Specimen (C.C. 1 percent)
Figure 6.17: Normalized Results on Cemented Specimen (C.C. 2 percent)
Figure 6.18: Influence of Cementation on $\alpha$ and $n$
in $n$ with an increase in relative density at 0 %, a decrease is observed at 1 and 2 %. When the standard deviation in $n$ in all the studies presented in Table 6.1 is considered, it is concluded that $n$ may best be considered to be constant for one type of soil and testing equipment. Therefore mean values are used as $n$ values in the analysis.

6.4 Summary

The two important parameters that affect cone test results are chamber size and boundary conditions. From comparisons with previously reported results, it is concluded that the test results obtained in this study are not significantly influenced by the chamber size and boundary condition effects. Diameter ratio of 42 is found to be sufficient to reduce the chamber size effects, particularly at lower densities (less than 50 %). Boundary conditions have minor influence when the tests are conducted at this diameter ratio. Crushability tests indicate that crushing under the cone is insignificant and can be disregarded. The tip and friction resistances reported are normalized and the effect of different factors are evaluated.
Chapter 7

ANALYSIS OF TEST RESULTS:
Theoretical and Empirical Methods

7.1 Introduction

The current state-of-the-art in the analysis of cone penetration testing depends largely on empirical or theoretical correlations developed through an evaluation of theoretical and experimental models. The theoretical models of penetration are based upon the following approaches: bearing capacity theories which assume rigid plastic material behavior, simulation of the penetration mechanism by the cavity expansion theory which allows easier incorporation of plasticity models but simplify the geometry of the problem, and the strain path method which assumes that the strain field can be estimated independent of the stress field. Some of the above theories have been successfully used in past studies to interpret cone penetration test results, yet improvements are still needed.

In this chapter, two bearing capacity theories and two cavity expansion theories are used to predict the measured cone tip resistance and these predictions are compared with experimental results. Friction resistance is also evaluated. Based upon these prediction schemes, a methodology is developed to estimate the strength parameters.

Another methodology based on empirical correlations is also presented. This approach is derived based on an assessment of the experimental and also theoretical tip and sleeve friction values. Predictions from this methodology are compared with experimental results. The second empirical method which is based on steady state line is also presented in another section. The final section presents a discussion on various classification charts and their use in the identification of cemented sands.
7.2 Theoretical Method

7.2.1 Tip Resistance - Bearing Capacity Theories

Several investigators have analyzed cone penetration as a bearing capacity problem (Meyerhof, 1961; Durgunoglu and Mitchell, 1973; Baligh, 1975; Janbu and Senneset, 1974). These theories assume different failure mechanisms which are then used to calculate ultimate bearing capacities using limit equilibrium approach. When penetration is treated with the conventional bearing capacity theories, soil is often assumed to behave as a rigid-perfectly plastic material. Therefore, the elastic strains and compressibility of the material are neglected. Bearing capacity theories which include compressibility have also been proposed (Vesic, 1973).

7.2.1.1 D & M Theory

Tip resistance is calculated by determining $N_c$ and $N_{c^*}$ presented in Equation 2.5 from the charts provided by Durgunoglu and Mitchell (1973) (Figure 2.14) and the strength properties reported in Tables 4.2 and 4.3 (Chapter 4). Peak values of friction angle and cohesion intercept are taken in calculating the tip resistance.

Figures 7.1, 7.2 and 7.3 presents the measured tip resistances with predicted tip resistances of cementations 0, 1 and 2 % respectively. The results of a previous study conducted at much lower confining stresses than this study are also presented (Figure 2.18). The theoretical predictions correlate excellently with measured resistances. The D & M theory assumes rigid-plastic behavior for the medium. The stress-strain behavior of cemented sands can be considered close to rigid-plastic, particularly at peak strength. This may be the reason behind the good correlations.

7.2.1.2 J & S Theory

The bearing capacity given in equation 2.5 is modified for drained conditions as follows:

$$q_v + a = N_q \cdot (\sigma'_{vo} + a) + \frac{1}{2} \cdot \gamma \cdot B \cdot N_q$$

(7.1)

The contribution of the $N_q$ to the overall resistance is relatively insignificant. Therefore, this term is neglected to obtain:

$$q_T = q_v = N_q \cdot (\sigma'_{vo} + a) - a$$

(7.2)
Figure 7.1: Comparison Between Theoretical and Measured Tip Resistances for C.C. 0 % Specimens (D & M Theory)
Figure 7.2: Comparison Between Theoretical and Measured Tip Resistances for C.C. 1% Specimens (D & M Theory)
Figure 7.3: Comparison Between Theoretical and Measured Tip Resistances for C.C. 2% Specimens (D & M Theory)
The strength parameters 'a' and 'ϕ' are taken from the Tables 4.1 and 4.2. \( N_q \) value is a function of angle of plastification, \( β \). \( β \) values ranging from -5 to -35 degrees are generally chosen for dilating sands (Janbu and Senneset, 1974). The value of this angle mainly depends upon relative density and confining pressure. Currently, selection of the \( β \) value is mostly based on experience. In order to formalize this selection, the following procedure is adopted. Based on the good predictions by D & M theory, it is assumed that J & S theory will also provide good predictions. The \( β \) value that will give the measured tip resistance is first determined. The calculated \( β \) values range from -5 to -25 degrees for low relative densities (45 to 55 %); -15 to -33 for medium relative densities (between 65 to 75 %) and -20 to -34 for higher relative densities (above 85 %). Then these \( β \) values are correlated to the dilation angle, \( \nu \) (Figure 7.4). The following best fit line is obtained from the figure.

\[
β = 1.43 \nu - 3.88 \quad (7.3)
\]

It should be noted that the dilation angle depends on the relative density and the confining pressure. Figure 4.23 can be used for estimating the dilation angle.

Using this equation, tip resistances are predicted and compared with measured tip resistances in Figures 7.5, 7.6 and 7.7. There is very good agreement between the measured and calculated resistances.

Figure 7.4 can be used in determining the plastification angle if dilation angle is known. Dilation behavior of other sands will be different for each sand, hence this figure cannot be used for sands other than Monterey No. 0/30 sand.

### 7.2.2 Tip Resistance - Cavity Expansion Theories

Cavity expansion theories have been applied to practical problems such as interpretation of pressuremeter tests (Hughes et al., 1977; Juran and Mahmoodzadegan, 1989) and cone penetration tests (Greeuw et al., 1988). Numerical solutions are required when large strain deformations are considered in the analysis. The analysis can be simplified by considering small strain deformations, and several closed form solutions were presented for such analysis (Carter et al., 1986; Vesic, 1972; Ladanyi, 1963).

The present work attempts to study two procedures in cavity expansion theories and their applications with regards to simulating cone penetration mechanism. The
Figure 7.4: Dilation Angles Versus Plastification Angles Used in J & S Theory
Figure 7.5: Comparison Between Theoretical and Measured Tip Resistances for C.C. 0 % Specimens (J & S Theory)
Figure 7.6: Comparison Between Theoretical and Measured Tip Resistances for C.C. 1% Specimens (J & S Theory)
Figure 7.7: Comparison Between Theoretical and Measured Tip Resistances for C.C. 2% Specimens (J & S Theory)
The first procedure is presented in the closed form solutions given by Carter et al., 1986. This solution is for small strains and assumes a constant dilation angle. The second procedure investigated by the author is the modeling used in cylindrical cavity expansion suggested by Juran and Mahmoodzadeh, 1989. The author developed a spherical cavity expansion solution using this procedure and updated the formulation for the effect of cementation. Predictions using both the constant dilation angle solution (Carter et al., 1986) and the newly developed spherical cavity expansion solution are presented.

7.2.2.1 Cavity Expansion Theory - Procedure 1

The first theory is proposed by Carter et al. (1986). Closed form solutions are presented for the small strain expansion of cylindrical and spherical cavities in an ideal cohesive frictional soil (Carter, et al., 1986). These solutions predict the limiting pressure necessary in expansion of the cavity. The limiting pressure, \( p_l \), is the pressure in the cavity which corresponds to a continuous deformation without pressure increase. Mohr-Coulomb yield function and an elasto-perfectly plastic soil behavior are used in deriving the following expression for limiting pressure (Carter et al., 1986):

\[
\frac{2G}{p_o + ccot \phi} = \frac{N - 1}{N + k} \left[ T \left( \frac{p_L + c \cot \phi}{\sigma_R + c \cot \phi} \right) \right]^{\gamma} - Z \left( \frac{p_L + c \cot \phi}{\sigma_R + c \cot \phi} \right)
\]

where

\[ T = (k + 1) \left( 1 + \frac{kx}{\alpha + \beta} \right) \]

\[ Z = (k + 1) \frac{kx}{\alpha + \beta} \]

\[ \sigma_R = \frac{1 + k}{N + k} N p_o \]

\[ \alpha = \frac{k}{M} \]

\[ \beta = 1 - k \left( \frac{N - 1}{N} \right) \]

\[ \gamma = \frac{1 + \alpha}{1 - \beta} \]

\[ M = \frac{1 + \sin \nu}{1 - \sin \nu} \]
\[ N = \frac{1 + \sin\phi}{1 - \sin\phi} \]  
\[ x = \frac{k(1 - \mu) - k\mu(M + N) + [(k - 2)\mu + 1]MN}{[(k - 1)\mu + 1]MN} \]

and \( k = 1 \) (for cylindrical cavity) and \( k = 2 \) (for spherical cavity); \( \phi \) is the friction angle; \( \nu \) is the dilation angle; \( \mu \) is the poisson ratio; \( p_o \) is the initial octahedral stress; \( G \) is the shear modulus and \( p_L \) is the limiting pressure.

An assumption is made with regards to isotropic pressure, \( p_o \).

\[ p_o = \frac{1}{3} (1 + 2K_o) \sigma_v \]  

The following steps are used in the analysis:

1. Modeling parameters, \( G, \phi, \) dilation angles are earlier determined for various cement contents and relative densities from the drained triaxial tests. These parameters are obtained by modeling drained triaxial tests using the Juran-Guermazi model. It is to be noted that the dilation angles are computed by taking the average of the incrementally computed dilation angles over the dilation region of the volumetric strain - axial strain curve. When negligible dilation angles are calculated (within the zone where volumetric strain remains constant), this averaging is terminated. This procedure is used since the dilation angle used in the cavity expansion model is defined as the slope of the expansion portion of the curve.

2. An iterative program is written in Fortran to estimate the limiting pressure. The listing of this program is presented in Appendix D. Modeling parameters obtained above are the input parameters to this program.

3. Once the limiting pressure is computed, the ratio of this limiting pressure to measured tip resistance is calculated. These ratios are then plotted versus relative density for cement contents 0, 1 and 2 % in Figures 7.8, 7.9 and 7.10.

\[ \frac{q_c}{p_L} = (1.2 - 0.2\, C.C.) \quad \text{Spherical Cavity} \]  
\[ \frac{q_c}{p_L} = f(D_r, C.C.) > 3.5 \quad \text{Cylindrical Cavity} \]
Figure 7.8: Ratio of Tip Resistance to Limiting Pressure (C.C. 0 %)

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Figure 7.9: Ratio of Tip Resistance to Limiting Pressure (C.C. 1 %)

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Figure 7.10: Ratio of Tip Resistance to Limiting Pressure (C.C. 2 %)
The above ratio indirectly explains the variation between the actual and the idealized expansions under the cone. The idealized expansion can be spherical or cylindrical in shape. When these ratios are closer to 1, the idealized expansion can be assumed to be close to real expansion. Ratios from both cavity simulations clearly show that spherical cavity simulations using Carter et al. (1986) formulation yield limiting pressures closer to tip resistances.

However, there is a wider variation in the ratios obtained from cylindrical cavity expansion. At present, the author does not have any explanations for this, except that the cavity under the cone is possibly far from being cylindrical in shape. Ratios from spherical cavity expansion from this study are compared with the same from another study conducted on uniform fine quartz sand, Oosterschelde sand (Greeuw et al., 1988) (Figure 7.11). A trend similar to the decrease in ratio with increase in relative density is observed in both results. However, the ratio is higher in the results obtained in Oosterschelde sand. This may due to the size of the cone used in the respective studies. A 36 mm diameter cone is used in the tests on Oosterschelde sand whereas a 12.7 mm diameter cone is used in the present study. This ratio is strongly affected by the dilation angle. Furthermore, in the study by Greeuw et al. (1988), the following equation (7.17) is used for determining dilation angles instead of measuring this angle using the volumetric strain data obtained from drained triaxial tests.

\[
\sin \nu = \frac{\sin \phi - \sin \phi_{cv}}{1 - \sin \phi \cdot \sin \phi_{cv}}
\]  

(7.18)

This equation underpredicts the dilation angle and also it does not account for the influence of confining stress. It gives the same dilation angle for all confining stresses. Hence in that study, the calculated limiting pressure is lower.

The major drawback of the analysis by Greeuw, et al., 1988 lies in the model assumed for the soil (Juran and Mahmoodzadegan, 1989). The soil (sand) is assumed as linearly elastic - perfectly plastic and the plastic flow is defined by constant rate of dilation. It is known that dense sands undergo contraction and strain hardening prior to peak principal stress ratio and then followed by a post strain softening behavior (Juran and Mahmoodzadegan, 1989). The dilation is maximum at the peak and is minimum or close to zero at the critical state or constant volume. The closed form solution presented by Carter et al. (1986) is derived based on a constant dilation an-
Figure 7.11: Comparison of Ratios

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gle at the peak and it does not account for the strain softening behavior. It is known that post peak strain softening can influence the response of insitu soil significantly, hence the above solutions appear to be restrictive for proper interpretations (Juran and Mahmoodzadegan, 1986).

7.2.2.2 Cavity Expansion Theory - Procedure 2

The analytical procedure proposed by the author is based on the soil model proposed by Juran and Beech (1986). The soil is assumed as homogeneous, isotropic, and strain hardening elasto-plastic material with a non-associated flow rule. A model is described in Chapter 4, and the modeling parameters are presented in Tables 4.4 and 4.5. This model uses octahedral stress variables (q and p) where as the present soil model is based on deviatoric stress \( t = \frac{\sigma_i - \sigma_n}{2} \) and average effective stress \( s = \frac{\sigma_i + \sigma_n}{2} \). However, soil modeling parameters do not change when the modeling parameters, \( q - p' \) or \( t - s' \) are used. For convenience, the yield function, hardening function and plastic potential functions are hereby presented in terms of \( t \) and \( s' \).

A Mohr-Coulomb type yield criterion is considered:

\[
f(\sigma_{ij}, \gamma) = \frac{t}{s'} - h(\gamma)
\]

where \( \sigma_{ij} \) is the stress tensor. The \( h(\gamma) \) is the strain hardening function relating the actual yield surface to the current strain rate. The strain hardening function used in the above equation must be specifically defined for the case of contracting and dilating material. For dense dilating sands, it is assumed that the hardening function \( h(\gamma) \) is parabolic to hyperbolic and can be written as:

\[
h(\gamma) = \frac{c \gamma (\gamma - a)}{(\gamma + b)^2}
\]

where

\[
a = -4 \frac{\sigma_0}{G} \frac{\sin^2 \phi_p}{\sin \phi_{cv}} \Gamma^2
\]

\[
b = 2 \frac{\sigma_0}{G} \sin \phi_p \Gamma
\]

\[
c = \sin \phi_{cv}
\]

and \( \Gamma = 1 + [1 - (\sin \phi_{cv} \sin \phi_p)]^{\frac{1}{2}} \).

The difference between the above \( a, b \) and \( c \) values and those of octahedral stress modeling should be noted.
The following flow rule which defines stress ratio-dilatancy relationship is derived based on energy considerations (Juran and Beech, 1986):

\[
\eta = \sin \nu = \frac{dc_p^e}{d\gamma^p} = \frac{1}{\mu_s} \sin \phi_{cv} - \frac{t}{s} \tag{7.24}
\]

where

\[
dc_p^e = \text{plastic volumetric strain increment}
\]

\[
d\gamma^p = \text{plastic deviatoric strain increment}
\]

\[
\eta = \text{dilation rate}
\]

\[
\nu = \text{dilation angle}
\]

and \(\mu_s\) is a correction modulus defined as

\[
\mu_s = \mu_1 \text{ when } \frac{t}{s} \leq \sin \phi_{cv}; \quad \text{contracting behavior}
\]

\[
\mu_s = \mu_2 \text{ when } \frac{t}{s} \geq \sin \phi_{cv}; \quad \text{dilating behavior}
\]

Using this soil model, the following procedure is adopted to obtain a spherical cavity solution. Spherical symmetry, radial equilibrium and compatibility conditions are used in the analysis.

The strains in the expansion are given by:

\[
\epsilon_r = -\frac{dx}{dr}; \quad \epsilon_\theta = \frac{x}{r}; \quad \epsilon_\phi = \frac{x}{r} \tag{7.25}
\]

where \(\epsilon_r, \epsilon_\theta\) and \(\epsilon_\phi\) = radial, circumferential and spherical strains; \(x\) = the radial displacement on the face of the cavity; \(r\) = radius of the cavity.

\(N\) is defined as the ratio of volumetric strain to the deviatoric strain:

\[
N = \sin \psi = \frac{\epsilon_v}{\gamma} = \frac{\epsilon_r + \epsilon_\theta + \epsilon_\phi}{\epsilon_r - \epsilon_\theta} \tag{7.26}
\]

From the above expression, the following relationship is obtained:

\[
\frac{\epsilon_r}{\epsilon_\theta} = \frac{2 + \sin \psi}{1 - \sin \psi} \tag{7.27}
\]

if elastic strains are neglected, Equation 7.27 can be expressed in the following form:

\[
d\gamma = -3 \frac{d\epsilon_\theta}{1 - \sin \nu} \tag{7.28}
\]
By differentiating Equation 7.26 with respect to $\gamma$, the following relation between the dilation angle, $\nu$, and $N$ is obtained:

$$\sin \nu = N + \gamma \frac{\partial N}{\partial \gamma}$$

(7.29)

The strain compatibility condition is given by:

$$\epsilon_r = \epsilon_\theta + r \frac{d \epsilon_\theta}{d \gamma}$$

(7.30)

The radial equilibrium equation is given by:

$$\frac{\partial \sigma_r}{\partial r} + 2 \frac{\sigma_r - \sigma_\theta}{\rho} = 0$$

(7.31)

Combining the above equations 7.28, 7.29 and 7.30 along with the radial equilibrium equation 7.31, the following equation for incremental lateral confining pressure is obtained:

$$\Delta \sigma_c = \frac{4}{3} \frac{h(\gamma)}{1 + h(\gamma)} \frac{1 - \epsilon_\theta}{1 + \epsilon_\theta} \frac{\Delta \epsilon_\theta}{\epsilon_\theta} \sigma_c$$

(7.32)

If the sand exhibits cohesion intercept, the following addition has to be made to the incremental pressure:

$$\Delta \sigma_{cc} = \frac{1 - \epsilon_\theta}{1 + \epsilon_\theta} \frac{\Delta \epsilon_\theta}{\epsilon_\theta} c \cos \phi$$

(7.33)

The following incremental procedure is used to compute the limiting pressure.

1. A small displacement, $x$ is assumed. Strains, dilation angle, $\nu$, and the deviatoric strain, $d\gamma$, are determined.

2. Using the $d\gamma$, the strain hardening function is calculated.

3. The next step is estimating the increment in lateral confining pressure by using equation 7.31. This is added to the initial lateral confining pressure to obtain the current lateral pressure.

4. The above steps are repeated until the incremental lateral pressure computed is relatively insignificant. The lateral pressure at the end of this final step is taken as the limiting pressure.
A Fortran program is written for the above model (Appendix D) and limiting pressures are estimated for different densities and cement contents. The limiting pressures are computed and tabulated along with tip resistance in Table 7.1. The ratios of tip resistance to limiting pressures are plotted versus vertical confining pressure and these figures are presented in Appendix D.

This procedure estimates lower limiting pressures than the procedure given by Carter et al., 1986. The first procedure assumed a constant dilation angle. The validity of this assumption is dubious since the dilation angle at higher strains is zero. As a result, the procedure proposed by Carter et al., 1986 clearly overestimates the limiting pressure.

The ratio of tip resistance to limiting pressure range from 2.2 to 3.5 at densities of 45%, 5.5 to 7.5 at densities of 65% and around $8$ for densities at 85%. This ratio increases with increase in density. A ratio called the 'form factor' is given between tip resistance and limiting pressure by Vesic (1977).

$$q_c = N_f \cdot p_L$$

$$q_c = \tan^2\left(\frac{\pi}{4} + \frac{\phi_{ev}}{2}\right) \cdot (1 + \sin \phi_{ev})^{-1} \cdot \exp\left[\frac{\pi}{2} - \phi_{ev} \cdot \tan \phi_{ev}\right] \cdot p_L$$

The actual shape under the cone is different from the idealized spherical shape. The $N_f$ which takes care of the shape difference as a function of friction angle at constant volume. The factors are calculated for the present test results and are included in the same table. The fact that the same form factor proposed by Vesic (1977) is of the similar order with the $\frac{q_c}{p_L}$ values suggests that the assumption on the cavity shape may be the reason behind high $\frac{q_c}{p_L}$ ratios.

Cavity expansion theories and the bearing capacity theories seem to provide good predictions of the measured tip resistance.

### 7.2.3 Friction Resistance

The theoretical prediction of friction resistance assumes that sleeve friction is only due to shear resistance. The following formula can be used to calculate the friction resistance:

$$f_s = S_s \cdot (\sigma_{vo} + a)$$  \hspace{1cm} (7.36)

in which
Table 7.1: Comparison of Cavity Expansion Theories

<table>
<thead>
<tr>
<th></th>
<th>$D_r$ 45%</th>
<th></th>
<th>$D_r$ 65%</th>
<th></th>
<th>$D_r$ 85%</th>
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<td>200</td>
<td>300</td>
<td>100</td>
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<td>300</td>
</tr>
</tbody>
</table>
| Units: $\sigma_v$ - kPa; $q_c$ and $p_L$ - MPa

<p>| | | | | | | |</p>
<table>
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<tr>
<th></th>
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<th></th>
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<th></th>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>C.C 0%</td>
<td>$p_L$</td>
<td>$q_c$</td>
<td>$\varepsilon_L$</td>
<td>Carter (1986)</td>
<td>$N_f$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.2</td>
<td>6.0</td>
<td>2.7</td>
<td>1.46</td>
<td>3.0 - 4.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.9</td>
<td>7.5</td>
<td>2.6</td>
<td>1.21</td>
<td>4.0 - 5.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.5</td>
<td>11.5</td>
<td>3.3</td>
<td>1.32</td>
<td>5.0 - 6.5</td>
<td></td>
</tr>
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<td>8.4</td>
<td>3.5</td>
<td>1.07</td>
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<td></td>
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<td>17.6</td>
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<td>4.3</td>
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<tr>
<td></td>
<td>4.1</td>
<td>35.0</td>
<td>8.5</td>
<td>1.43</td>
<td></td>
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</tr>
<tr>
<td>C.C 1%</td>
<td>$p_L$</td>
<td>$q_c$</td>
<td>$\varepsilon_L$</td>
<td>Carter (1986)</td>
<td>$N_f$</td>
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<td>1.02</td>
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</tr>
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<tr>
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<td></td>
</tr>
<tr>
<td>C.C 2%</td>
<td>$p_L$</td>
<td>$q_c$</td>
<td>$\varepsilon_L$</td>
<td>Carter (1986)</td>
<td>$N_f$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>8.0</td>
<td>3.2</td>
<td>0.95</td>
<td>3.0 - 4.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.2</td>
<td>12.0</td>
<td>3.8</td>
<td>1.04</td>
<td>4.0 - 5.0</td>
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</tr>
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<tr>
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<td>20.2</td>
<td>7.0</td>
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</tr>
<tr>
<td></td>
<td>3.7</td>
<td>29.5</td>
<td>8.0</td>
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<td></td>
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</tr>
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<td>8.0</td>
<td>0.74</td>
<td></td>
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</tr>
</tbody>
</table>
\[ f_s = \text{shear or friction resistance} \]
\[ S_s = |r| \cdot \tan \phi \cdot K \]
\[ |r| = \frac{\tan \delta}{\tan \phi} \]

The earth pressure coefficient \( K \) is generally taken as the \( K_o \) in the pile friction capacity calculations below a critical depth (Poulos, 1976). Figure 7.12 shows the ratios of \( K/K_o \) values for various densities at each cement content. These figures depict that \( K/K_o \) values are closer to 1 for lower densities (around 45\%) and increase with higher densities. In soils with high dilation characteristics such as cemented sands, and at higher densities, using \( K_o \) in the equation will predict lower friction resistance. The restrained dilatancy will increase the confinement on the interface resulting in higher \( K \) values.

Figure 7.13 presents the influence of dilation angle on \( K/K_o \). Since dilation angle depends on the relative density and confining stress, similar observations are noted. Results from Baldi (1981) are also plotted in this figure. These results were obtained on a normally consolidated Ticino sand. This variation between both results is due to the difference between dilation characteristics of the two sands.

### 7.2.4 Approach for Estimating Cohesion and Friction Angles

A chart for estimating relative densities and cohesion was proposed by Puppala et al., 1993. This chart was based on tests conducted by Rad and Tumay (1986) using cemented sands. These results were obtained in a rigid PVC chamber and at very low confining stresses (less than 5 kPa). Furthermore, the influence of rigid boundary conditions, chamber size effects and high stresses were not evaluated. The present study gives a way to incorporate the effect of all these factors.

Present results were conducted in a flexible double-walled calibration chambers and were reported for three confining stresses, relative densities and cement contents. Different bearing capacity theories were used to predict these measured values. These theories predicted tip resistance reasonably well. Hence, the prediction of the bearing capacity theories are used in the following semi-empirical approach developed.

Cone penetration test results obtained in calibration chambers demonstrate that tip resistance increases nonlinearly with the increase in vertical effective stress both

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Figure 7.12: $K/K_o$ Versus Relative Density

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Figure 7.13: Influence of Dilation Angle on $K/K_0$. 

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due to changes in dilational characteristics and peak friction angle with the increase in confinement and also due to grain crushing at higher confinement. Therefore, the empirical and semi-empirical correlations of tip resistance employ a non-dimensional tip resistance \( \frac{Q_{t}}{\sigma_{atm}} \left( \frac{z}{\sigma_{atm}} \right)^{n} \), where \( \sigma_{atm} \) is atmospheric pressure and \( n \) varies between 0.55 to 0.84. Similar normalization was found necessary in relating cone penetration resistance to low strain dynamic properties by other investigators (Rix and Stokoe, 1991). This implies \( \alpha \) value is a function of the initial state of the soil.

Tip resistances predicted by D & M theory showed good agreement with measured values up to a confining stress of 350 kPa. Hence, non-linearity may be neglected at vertical stress less than 350 kPa and for relative densities less than 80% (Villet and Mitchell, 1981). Similar findings were observed in other studies where bearing capacity theories were used (Baldi, et al., 1981). Possible reason for this linear relationship between \( q_{c} \) and \( \sigma_{v} \) was due to the non-linearity of the strength envelop (Baldi, et al., 1991); i.e. \( \phi' \) value decreases as confinement increases. Bearing capacity theory predictions of the tip resistances showed quite good agreement in this study. This implies, a linear relationship can be assumed between the tip resistance and vertical effective stress when bearing capacity formulations are used. Hence, normalized tip resistance, \( \frac{Q_{t}}{\sigma_{v}} \) eliminates the influence of the vertical effective stress.

The following methodology is used in preparing the semi-empirical approach. Cement content and relative density are expressed in terms of cohesion and friction angle. Tip, friction resistances and friction ratios are calculated for different cohesion and friction angles. Using these values, a chart is prepared.

Figure 7.14 compares the normalized tip resistance, \( \frac{Q_{t}}{\sigma_{v}} \) with respect to friction ratio for both bearing capacity theories. These theories provide similar predictions of normalized cohesion intercept, \( \frac{\phi'_{c}}{\sigma_{v}} \) from a knowledge of friction ratio and tip resistance.

Figure 7.15 and 7.16 provide charts which can be used to estimate the normalized cohesion intercept and the relative density when the friction ratio and normalized tip resistance are known. The first chart is prepared for \( \frac{\phi'_{c}}{\sigma_{v}} \) ranging from 0 to 1. The second figure is for \( \frac{\phi'_{c}}{\sigma_{v}} \) values of 0 to 5. Present testing data is depicted in this figure for comparisons. This data shows that reasonable estimates of \( \frac{\phi'_{c}}{\sigma_{v}} \) and \( D_{r} \) are possible by this chart. Once the range of relative density is known, the friction
Figure 7.14: The Normalized Tip Resistance Versus Friction Ratio for Various $\frac{d_r}{\sigma_v}$ Values
angles can be estimated by using Figure 7.17. Although the above procedures provide good estimates of the strength parameters, insitu data is still necessary to verify and improve the prediction scheme.

7.3 Empirical Method

Two empirical methods are presented in this section. The first method is based on the octahedral stress normalization which is described in Chapter 6. This approach is later modified such that it can be used in the interpretation of insitu tests. The second method is based on the state parameter concept. This method requires a classification chart for identifying cemented deposits.

7.3.1 Empirical Method Based on $\alpha$ Approach

Existing relative density charts are developed from chamber test results on a particular type of sand. These charts are generally valid for that particular type of sand. An attempt is made here to normalize various cone results and use them in formulating a new approach for estimating the relative density. This method is also extended to estimate cementation levels.

7.3.1.1 Tip Resistance

Figure 6.9 (Chapter 6) which depicts the diameter ratio influence on $\alpha$ for various densities can be used for estimating relative density. The $\alpha$ values for different relative densities at a certain diameter ratio are taken and are replotted (Figure 7.18). The best fit regression lines equations for each diameter ratio are presented as follows:

$$\frac{q_c - p_{oct}}{\sigma_{atm}} = (3.6D_r - 54.0) \left( \frac{p_{oct}}{\sigma_{atm}} \right)^n, \quad D.R. = 60 \tag{7.37}$$

$$\frac{q_c - p_{oct}}{\sigma_{atm}} = (3.2D_r - 43.0) \left( \frac{p_{oct}}{\sigma_{atm}} \right)^n, \quad D.R. = 50 \tag{7.38}$$

$$\frac{q_c - p_{oct}}{\sigma_{atm}} = (3.0D_r - 39.0) \left( \frac{p_{oct}}{\sigma_{atm}} \right)^n, \quad D.R. = 42 \tag{7.39}$$

While the above equations generalize the change in tip resistance with confinement, $n$ values are considered specific to the study. For uncemented, clean, normally con-
Figure 7.15: A Chart for Estimating Cohesion Intercept and Relative Density
Figure 7.16: A Chart for Estimating Cohesion Intercept and Relative Density
Figure 7.17: Relative Density Versus Friction Angles
Figure 7.18: $\alpha$ Versus Relative Density for Various Diameter Ratios
solidated sands, the author finds that \( n \) values obtained are affected by the particle shape (Table 7.2).

The above equations, 7.37, 7.38 and 7.39 are used to produce the following equation which depicts the influence of chamber diameter ratio. This equation is valid for chamber diameter ratio values up to 60.

\[
\frac{q_c - p_{oct}}{\sigma_{atm}} = [(0.03 \, D.R. + 1.75)D_r - (0.84 \, D.R. + 2.6)] \left( \frac{p'_{oct}}{\sigma_{atm}} \right)^n
\]  

(7.40)

All the above equations are derived based on the results for clean, uncemented sands. The effect of cementation which is valid up to a chamber diameter ratio of 42 is included below.

The \( \alpha \) values of 1 and 2 % cement contents are comparatively plotted along with the uncemented results (Figure 7.19). The following equations are obtained for the cement contents at 1 and 2 %.

\[
\frac{q_c - p_{oct}}{\sigma_{atm}} = (3.08 \, D_r - 40.8) \left( \frac{p'_{oct}}{\sigma_{atm}} \right)^n, \quad C.C. = 1\% 
\]  

(7.41)

\[
\frac{q_c - p_{oct}}{\sigma_{atm}} = (3.64 \, D_r - 47.4) \left( \frac{p'_{oct}}{\sigma_{atm}} \right)^n, \quad C.C. = 2\% 
\]  

(7.42)

Combining equations 7.39, 7.41 and 7.42 yield

\[
\frac{q_c - p_{oct}}{\sigma_{atm}} = [(2.9 + 0.3 \, C.C.)D_r + (4.0 \, C.C. + 38.33)] \left( \frac{p'_{oct}}{\sigma_{atm}} \right)^n
\]  

(7.43)

where \( n \) value is:

\[
n = 0.84 - 0.9 \ln \left( 1 + 0.56(C.C.)^{0.15} \right)
\]  

(7.44)

This equation includes the effect of both cementation and relative density. The only disadvantage in this approach is the use of octahedral stress variation. In field tests, it is difficult to estimate the lateral stresses, thereby octahedral stresses. An attempt is made to make this approach applicable to field tests by replacing...
Figure 7.19: $\alpha$ Versus Relative Density for Various Cement Contents
Table 7.3: The $\alpha_v$ and $n_v$ for Various Investigations

<table>
<thead>
<tr>
<th>C.C</th>
<th>$D_r$</th>
<th>$\log \alpha_v$</th>
<th>$n_v$</th>
<th>$n_{\text{mean}}$ (Std. Dev.)</th>
<th>$\alpha_v$</th>
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<tr>
<td>0%</td>
<td>45</td>
<td>1.72</td>
<td>0.98</td>
<td></td>
<td>53</td>
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<tr>
<td></td>
<td>65</td>
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<td>0.99</td>
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<tr>
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<td>85</td>
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<td>(0.01)</td>
<td>112</td>
</tr>
<tr>
<td>1%</td>
<td>45</td>
<td>1.80</td>
<td>0.85</td>
<td></td>
<td>63</td>
</tr>
<tr>
<td></td>
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<td>0.88</td>
<td>97</td>
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<tr>
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<td>0.92</td>
<td>(0.03)</td>
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<tr>
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<tr>
<td></td>
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<td>2.31</td>
<td>0.83</td>
<td>(0.06)</td>
<td>204</td>
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</table>

Octahedral stresses with vertical stresses. Vertical stress normalization is performed on present results and the variables in these are defined as $\alpha_v$ and $n_v$. The $\alpha_v$ and $n_v$ values are estimated (Figure 7.20) and are presented in Table 7.3. Tip and sleeve friction values used in this normalization are taken from experimental investigations. Some of the values which differ significantly from theoretically computed values are replaced with theoretically computed values. This is done to improve the empirical correlations.

The following empirical equation is obtained for the relationship between normalized tip resistance and vertical effective stress.

$$\frac{q_c - \sigma_v}{\sigma_{\text{atm}}} = [(1.71 + 0.99 \text{C.C.})D_r - (41.9 \text{C.C.} + 34.3)] \left(\frac{\sigma_v}{\sigma_{\text{atm}}}\right)^{n_v}$$

(7.45)

where

$$n_v = 0.99 - 0.1(C.C.)$$

(7.46)

This linear equation is valid up to 2% cementation level.

### 7.3.1.2 Friction Resistance

The friction resistance in test results can be expressed in the following form.

$$\frac{f_s}{\sigma_{\text{atm}}} = \beta \left(\frac{\sigma_v}{\sigma_{\text{atm}}}\right)^{n_1}$$

(7.47)
Figure 7.20: Normalized Plots of Data to Determine $\alpha_v$ and $n_v$ (Present Test Results: C.C. 0 and 2 %)
Figure 7.21: $\beta$ and $n_1$ from Present Test Results
Figure 7.22: $\beta$ and $n_1$ from Present Test Results
Table 7.4: The $\beta$ and $n_1$ for Various Cementations

<table>
<thead>
<tr>
<th>C.C.</th>
<th>$D_r$</th>
<th>$\beta$</th>
<th>$n_1$</th>
<th>$n_{1,m}$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>45</td>
<td>0.23</td>
<td>1.14</td>
<td>1.16 (0.08)</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>0.40</td>
<td>1.09</td>
<td>1.09</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>85</td>
<td>0.42</td>
<td>1.25</td>
<td>1.25</td>
<td>0.45</td>
</tr>
<tr>
<td>1</td>
<td>45</td>
<td>0.60</td>
<td>0.46</td>
<td>0.51 (0.10)</td>
<td>0.58</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>0.74</td>
<td>0.44</td>
<td>0.87</td>
<td>0.74</td>
</tr>
<tr>
<td></td>
<td>85</td>
<td>0.97</td>
<td>0.63</td>
<td>0.91</td>
<td>0.91</td>
</tr>
<tr>
<td>2</td>
<td>45</td>
<td>0.41</td>
<td>0.85</td>
<td>0.64 (0.19)</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>0.83</td>
<td>0.46</td>
<td>0.74</td>
<td>0.74</td>
</tr>
<tr>
<td></td>
<td>85</td>
<td>0.91</td>
<td>0.60</td>
<td>1.07</td>
<td>1.07</td>
</tr>
</tbody>
</table>

In order to determine $\beta$ and $n_1$, friction resistance results of this study are normalized and plotted (Figure 7.21 and 7.22). The $\beta$ and $n_1$ are estimated in each figure and tabulated (Table 7.4).

After performing similar analysis as in the case of tip resistance, friction resistance can be expressed in the following form:

$$
\frac{f_s}{\sigma_{atm}} = [(0.002 \text{C.C.} + 0.006) D_r + (0.27 \text{C.C.} - 0.02)] \left( \frac{\sigma_y}{\sigma_{atm}} \right)^{n_1} \tag{7.48}
$$

where $n_1 = 1.16 - 1.37 \ln(1 + 0.56(\text{C.C.})^{0.13})$.

Tip and friction resistances are defined empirically in the equations 7.45 and 7.48. These equations are used in the following two procedures proposed in determining geotechnical characteristics of deposits from cone penetration tests. These procedures are presented in the following subsections as Approach 1 and 2. These empirical procedures are only proposals and they have to be validated in the field.

### 7.3.1.3 Approach 1

This approach can be used provided unconfined compression strength, $q_f$ is known. Hence, this approach requires block sampling of the specimens from the testing location or some other method of estimating the unconfined compression strength or the cohesion intercept of the deposit.

The expression for cement content as a function of $q_f$ and $D_r$ is necessary in this procedure. The unconfined compression test results conducted on cemented specimens of 1 and 2 % at a curing period of 7 days are considered. $q_f$ is correlated
with relative density for both cement contents using the following equations (Figure 7.23).

\[
q_f = 0.44 D_r + 5.6 \quad C.C. = 1\%
\]

\[
q_f = 0.33 D_r + 19.5 \quad C.C. = 2\%
\]

(7.49)

(7.50)

These equations are combined to produce the following equation for \( q_f \) which is a function of relative density and cement content,

\[
q_f = (0.165 C.C. + 0.09) D_r + (9.7 C.C. - 1.37).
\]

(7.51)

From this, cement content expression is derived,

\[
C.C. = \left[ \frac{(q_f + 1.37 - 0.09 D_r)}{(0.165 D_r + 9.7)} \right]
\]

(7.52)

The above equation 7.52 along with equation 7.45 are used in preparing the following charts for estimation of relative densities. Cemented deposits are defined according to their unconfined compression strength (Rad, 1984). Low cemented deposits are defined to have a \( q_f \) of 100 kPa. However, cement contents of 1 and 2\% which are used to simulate these low cemented deposits exhibited unconfined compression strengths of up to 50 kPa (Rad, 1984). The above equation is used along with tip resistance expression in preparing charts for various \( q_f \) values. These charts (Figures 7.24, 7.25 and 7.26) are used for estimating relative densities for \( q_f \) values of 0, 20 and 40 kPa respectively. Remaining plots for other unconfined compression strengths are presented in Appendix D.

This method can not be extended to higher unconfined compression strengths unless further cemented specimens (higher C.C.) are tested. A \( q_f \) value of 50 kPa represents a cement content of above 3\% and the equations derived for \( n \), \( q_c \) and C.C. will not be valid at these cement contents. Further research is necessary to establish the relationships at higher cement contents.

### 7.3.1.4 Approach 2

In the event the block sampling is not available, Approach 2 is proposed. Both tip and friction resistance expressions along with cement content equation 7.52 are used in preparing the Figures 7.27, 7.28 and 7.29 to estimate both \( q_f \) and \( D_r \). Each
Figure 7.23: The $q_f$ Versus Relative Density for C.C. 1 and 2 %
Figure 7.24: Chart for Estimating Relative Density ($q_f = 0$ kPa) (Approach 1)
Figure 7.25: Chart for Estimating Relative Density \((q_f = 20 \text{ kPa})\) (Approach 1)
Figure 7.26: Chart for Estimating Relative Density \( q_f = 40 \text{ kPa} \) (Approach 1)
Figure 7.27: Chart for Estimating \( D_r \) and \( q_f \) for \( \frac{\sigma_c}{\sigma_{atm}} = 1 \). (Approach 2)
Figure 7.28: Chart for Estimating $D_r$ and $q_f$ for $\frac{\sigma_v}{\sigma_{atm}} = 2$. (Approach 2)
Figure 7.29: Chart for Estimating $D_r$ and $q_f$ for $\frac{\sigma_v'}{\sigma_{atm}} = 3$. (Approach 2)
figure is valid for a certain vertical effective stress. For stresses other than those given, charts are to be interpolated.

The above approaches not only identify the cemented deposits but also give the strength properties. However, at present these methodologies are applicable only for subrounded to subangular materials like Monterey sands. Generalization of this approach could not be done due to the lack of cemented cone test results on other sands. Furthermore, the validity of both approaches are to be investigated for field test results.

7.3.2 Empirical Method Based on State Parameter Approach

In this approach, the first step is to identify whether the sand deposits are cemented or uncemented. Hence, a classification chart is developed by plotting friction ratio on the x-axis and normalized tip resistance on the y-axis (Figure 7.30). The friction ratio is calculated by taking the ratio of friction resistance \( \frac{r}{\sigma_{tip}} \) to tip resistance \( \frac{q_c-\sigma_v}{\sigma_{atm}} \). This is expressed in %. There is a marked zone in the figure which represents the possible cemented deposits. Any cone results of the sand that lie in this zone can be expected to have cementation.

Another empirical approach generally used in the estimation of relative densities is based on the concept of state parameter. The definitions of the state parameters are presented earlier in Chapter 4. State parameters are calculated by subtracting the void ratio at the steady state from the natural void ratio. These are calculated for both cemented and uncemented test results. Void ratio at steady state is same for all confining pressures up to 300 kPa used in the triaxial test. This is because the SSL at these confining pressures is still a line parallel to the x-axis. There is no variation between state parameters of sands at different cement contents since critical state lines at different cementation levels lie close to each other (Figure 4.11). Table 7.5 presents the steady state parameters.

Figure 7.31 (uncemented) and 7.32 (cemented) present \( q_c-\sigma_v \) on x-axis and \( \sigma'_v \) on y-axis. Each curve in the figure represents a particular state parameter, \( \psi \). Vertical stresses are used in this approach since it is difficult to estimate the lateral stresses in the field. Once state parameter is known, strength parameters can be obtained from the \( \psi-\phi \) and \( \psi \)-cohesion correlations (Figure 7.33). Table 7.6 also presents the
Figure 7.30: Classification Chart for Estimating Cemented Deposits
Table 7.5: The Steady State Parameters

<table>
<thead>
<tr>
<th>$D_r$</th>
<th>$\sigma'_y$</th>
<th>$e$</th>
<th>$e_{ssl}$</th>
<th>$\psi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>100</td>
<td>0.59</td>
<td>0.77</td>
<td>-0.18</td>
</tr>
<tr>
<td>90</td>
<td>200</td>
<td>0.59</td>
<td>0.77</td>
<td>-0.18</td>
</tr>
<tr>
<td>88</td>
<td>300</td>
<td>0.59</td>
<td>0.76</td>
<td>-0.17</td>
</tr>
<tr>
<td>72</td>
<td>100</td>
<td>0.63</td>
<td>0.77</td>
<td>-0.14</td>
</tr>
<tr>
<td>69</td>
<td>200</td>
<td>0.64</td>
<td>0.77</td>
<td>-0.13</td>
</tr>
<tr>
<td>71</td>
<td>300</td>
<td>0.63</td>
<td>0.76</td>
<td>-0.13</td>
</tr>
<tr>
<td>49</td>
<td>100</td>
<td>0.69</td>
<td>0.77</td>
<td>-0.08</td>
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<tr>
<td>56</td>
<td>200</td>
<td>0.67</td>
<td>0.77</td>
<td>-0.10</td>
</tr>
<tr>
<td>55</td>
<td>300</td>
<td>0.67</td>
<td>0.76</td>
<td>-0.10</td>
</tr>
</tbody>
</table>

Table 7.6: The Strength Parameters

<table>
<thead>
<tr>
<th>C.C.</th>
<th>$\psi$</th>
<th>$D_r$</th>
<th>$\phi_{peak}$</th>
<th>$\phi_{res}$</th>
<th>$c_{peak}$</th>
<th>$c_{res}$</th>
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<tbody>
<tr>
<td>0</td>
<td>-0.18</td>
<td>85</td>
<td>39.0</td>
<td>35.0</td>
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<td>0</td>
</tr>
<tr>
<td></td>
<td>-0.14</td>
<td>65</td>
<td>36.5</td>
<td>34.5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>-0.09</td>
<td>45</td>
<td>35.0</td>
<td>34.0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>-0.18</td>
<td>85</td>
<td>38.0</td>
<td>36.0</td>
<td>14.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>-0.14</td>
<td>65</td>
<td>36.5</td>
<td>35.5</td>
<td>11.5</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>-0.09</td>
<td>45</td>
<td>35.0</td>
<td>35.0</td>
<td>9.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>-0.18</td>
<td>85</td>
<td>39.0</td>
<td>36.0</td>
<td>30.0</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>-0.14</td>
<td>65</td>
<td>37.5</td>
<td>35.5</td>
<td>25.0</td>
<td>15.5</td>
</tr>
<tr>
<td></td>
<td>-0.09</td>
<td>45</td>
<td>36.0</td>
<td>35.0</td>
<td>20.0</td>
<td>12.0</td>
</tr>
</tbody>
</table>

Note: Results are extrapolated for the above relative densities from Rad (1984)

strength parameters for various $\psi$ values of Monterey No. 0 sand. Since figures 7.31 and 7.32 are presented for various cement contents, the first step is to identify the amount of cementation. Figure 7.30 can be used for that purpose.

Insitu data are still necessary to verify and improve this scheme. Above approaches are empirical and better correlations can be obtained by increasing the number of test results and including field correlations.

Selection of the method (empirical or semi-empirical) will not affect the interpretations since both methods provide similar predictions. However, the empirical scheme can be improved by including more cemented specimen results.
Figure 7.31: Chart for Estimating $\psi$ for Uncemented Sand

$q_e - \sigma_v \ (kg/cm^2)$

Uncemented Sands

$\psi = -0.175$
$\psi = -0.135$
$\psi = -0.090$
Figure 7.32: Chart for Estimating $\psi$ for Cemented Sands
Figure 7.33: $\psi$ Versus Cohesion and Friction Angles

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7.4 Influence of Vertical Confining Pressure on Cementation

Tip resistance increases with an increase in cementation. This increase can be observed significantly at shallow depths (small overburden pressures). However, after certain depth, the influence of cementation on tip resistance can be disregarded since its contribution to tip resistance is less than 4%. To investigate this aspect, the bearing capacity results at various depths for different cementations are considered.

Figures 7.34 (cementation 1%) and 7.35 (cementation 2%) are plotted with the ratio of the difference between the cemented tip resistance and the uncemented tip resistance as a percentage of the uncemented tip resistance, \( \frac{Q_{c,cem} - Q_{c,uncem}}{Q_{c,uncem}} \) on x-axis and vertical confining pressure on y-axis. This ratio decreases with the increase in confining pressure. The ratio is closer to 0.04 at confining pressures of 600 kPa, suggesting that the cementation contribution is insignificant. This implies that for confining pressures of magnitude 600 kPa (equivalent to 60 m of dry soil or 120 m of saturated soil), semi-empirical or empirical charts for uncemented sands can be used for interpretations.

7.5 Discussion

An attempt is made in this section to compare the results of this study with the various classification charts proposed by different investigators. Classification charts by Schmertmann (1978), Douglas and Olsen (1981), Tumay (1985) and Robertson and Campanella (1985) are used. Figures 7.36, 7.37, 7.38 and 7.39 present these charts. Schmertmann (1978) charts is in agreement with present results. Chart by Tumay (1985) classify dense cemented results close to the present test results, however it classifies the medium to loose cemented sands as loose sands. Similar observations are noted in Campanella's chart. Cemented sand results overlap the zones 1, 6, 7, 8 and 9 in this chart (Robertson and Campanella, 1985). The interesting aspect of this chart is that the cemented sands lie in a zone of possible liquefiable soils which signifies the importance of identifying cemented deposits.

Figure 7.37 shows a relatively more comprehensive chart developed by Douglas and Olsen (1981). This chart not only uses the normalized parameters but also
Figure 7.34: Influence of Cementation (1 %) and Confining Pressure on Tip Resistance

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Figure 7.35: Influence of Cementation (2 %) and Confining Pressure on Tip Resistance
recognizes the change in n value with the type of soil. Cementation is not included in this chart. The author updated that region of the chart as given in Figure 7.37.

It was earlier discussed that $D_r$ interpretation charts can not be valid for cemented sands since cementation has pronounced effect on cone resistance. In addition, charts are not developed for determining unconfined compression strength and cohesion intercept of cemented sands. This research has accomplished that goal by providing empirical and theoretical evaluation methods. These methods can be validated and also updated by conducting field tests on cemented deposits.

7.6 Summary

Theoretical schemes such as bearing capacity theories and cavity expansion theory are used to predict tip resistance. Bearing capacity theories proposed by Dur- gunoglu and Mitchell (1973) and Janbu and Senneset (1974) provided reasonable estimates of tip resistances. Two procedures are used for simulating cavity expansion. The first procedure of cavity expansion theory predicts limiting pressure which can be correlated with measured tip resistances. Spherical cavity expansion predicted a limiting pressure which is closer to tip resistance than the cylindrical cavity expansion. This theory still needs the actual expansion that takes place under the cone. The second procedure of cavity expansion theory is based on a soil model proposed by Juran and Beech (1987) and it predicts lower limiting pressures than the first theory. Differences are attributed to the assumptions involving the dilation angles. The $K$ values required for measured friction resistances show that they are greater than $K_o$. This is due to the dilation characteristics of the sand.

A new chart is developed for estimating the strength properties using bearing capacity theories. Comparisons with present results show good predictions of strength properties. However, this method still needs to be checked in the field.

Two empirical methods are also presented for estimating the relative density and unconfined compression strength in cemented deposits. The first method is based on the $\alpha$ method and can be used in two approaches. Charts for various confining pressures are presented in the first approach. Block sampling and unconfined compression testing are required for this approach. When block sampling is not feasible, the second approach can be used. The second empirical method is based on
Figure 7.36: Classification Chart (Schmertmann, 1978)
Figure 7.37: Classification Chart (Douglas and Olsen, 1981)
Figure 7.38: Classification Chart (Tumay, 1985)
**PRESENT STUDY**

<table>
<thead>
<tr>
<th>ZONE</th>
<th>$q_{T/N}$</th>
<th>SOIL BEHAVIOUR TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>SENSITIVE FINE GRAINED</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>ORGANIC MATERIAL</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>CLAY</td>
</tr>
<tr>
<td>4</td>
<td>1.5</td>
<td>SILTY CLAY TO CLAY</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>CLAYEY SILT TO SILTY CLAY</td>
</tr>
<tr>
<td>6</td>
<td>2.5</td>
<td>SANDY SILT TO CLAYEY SILT</td>
</tr>
<tr>
<td>7</td>
<td>3</td>
<td>SILTY SAND TO SANDY SILT</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>SAND TO SILTY SAND</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>SAND</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>GRAVELLY SAND TO SAND</td>
</tr>
<tr>
<td>11</td>
<td>1</td>
<td>VERY STIFF FINE GRAINED</td>
</tr>
<tr>
<td>12</td>
<td>2</td>
<td>SAND TO CLAYEY SAND</td>
</tr>
</tbody>
</table>

--- Liquefiable Zone

Figure 7.39: Classification Chart (Robertson and Campanella, 1986)

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the steady state concept. This approach requires a prior knowledge of cementation. Hence, a classification chart which provides a zone for cemented materials is prepared. Any cone test result that lie within the zone defined by the chart is likely to indicate cementation. Once cementation is known, $\psi$ can be determined by using the proper charts. The $\psi$-strength correlations provides strength parameters. Both these methods are need to be assessed in the field. The existing classification charts are also updated to include cemented sand zones.
Chapter 8

SUMMARY AND CONCLUSIONS

8.1 Summary

Natural cemented deposits are very common throughout different states in US and also different parts of the world. Generally, cementation effect on strength properties of sand is neglected since cementation often improves them. However, recent studies indicate that neglecting cementation, particularly the smaller degree of cementation bonds results in overestimation of the liquefaction resistance and underestimation of the stability of slopes and strength properties. This signifies the importance of identifying cementation in the natural deposits. Difficulty in sampling naturally cemented deposits prompts the need of using insitu testing methods. Cone penetration testing is one such method which is gaining wide acceptance and use in the USA and the world due to its repeatability, economy and capability to provide accurate, repeatable vertical soil profiles and pertinent engineering parameters related to the sounded deposits.

Preliminary studies indicated that cementation resulted in an increase of tip and friction resistance. The existing interpretations which are developed for clean sands would be invalid for cemented sands. A calibration chamber study was conducted.

Monterey No. 0/30 sand and ordinary portland cement (Type I) were used in specimen preparation. Pluviation method was adopted since it best simulates the natural cemented specimen structure. Strength tests (triaxial and unconfined compression tests) and calibration chamber tests were conducted. Triaxial tests showed that cementation induces cohesion intercept, thereby increasing the overall strength. Unconfined compression tests showed that denser cemented specimens display a maximum $q_f$ value of 50 kPa. These results are later used in interpreting cone penetration test results.

The cone tests were conducted in a large scale calibration chamber, which can house a sample of 0.53 m in diameter and 0.79 m in height. Specimens were prepared at three ranges of relative densities (45 - 55, 65 - 75 and above 80 %) and tested at
three confining stresses (100, 200 and 300 kPa). A miniature quasi-static penetrometer of 1.27 cm in diameter was used in the tests to give a diameter ratio of 42. For cemented specimens, a curing period of 7 days was adopted.

Test results on uncemented sands were first assessed for repeatability and accuracy. Then, these results were compared with three previous investigations conducted on similar type of sand. These comparisons showed that boundary conditions, chamber size effects, cementation, grain size and shape influence the test results.

An empirical interpretation scheme was suggested by normalizing the cone test results with respect to octahedral stresses. This approach was then used in preparing two prediction methods; one in which unconfined compression strength of the tested sand is known through block sampling and the second when such sampling procedures are not available. These procedures need to be validated by field tests. The parameters used in the above normalization, \( \alpha \) and \( n \) were also investigated to assess the influence of various parameters on cone test results.

Two bearing capacity theories and small strain cavity expansion theory were used in predicting tip resistances. Durgunoglu and Mitchell (1973) and Janbu and Senneset (1974) bearing capacity theories predicted measured tip resistances quite closely. The limiting pressure predicted by spherical cavity expansion was close to tip resistance. Cylindrical cavity expansion predicted limiting pressures lower than measured tip resistance. The cavity under the cone which is different from the idealized cavity may result in this difference. The \( K \) values back calculated from sleeve friction values in testing were generally greater than \( K_o \). The trend displays \( K \) being closer to \( K_o \) at lower densities. Bearing capacity theories were used in formulating an approach to predict the strength parameters of cemented sands.

Critical state concept was used in preparing another empirical approach in prediction. A classification chart is provided which gives cementation value when cone test results are known. Once it is established that there is cementation, proper figures have to be used for estimating the Been's parameter, \( \psi \). The \( \psi \) value provides strength parameters from the correlations.
8.2 Conclusions

The following conclusions are drawn from this study.

1. Pluviation was found to be the best process in specimen preparation of both cemented and uncemented specimens. Pluviation or raining process showed that the sieve sizes, shutter porosities and the height of fall (distance between the end of the sieve to the top of deposited sand) affect the relative densities of the specimens. Specimens of 40 to 90 % relative density were prepared by varying these variables. Density tests along the depth of the specimen indicated that uniform specimens can be prepared with the pluviation process. Scanning microscope photographs of the cemented samples at different depths of the specimen showed that cement bonding developed all along the depth. The segregation of cement during pluviation was insignificant.

2. Tests conducted with a piezocone in a specimen prepared at 85 % relative density and 2 % cement content showed that there is no excess pore pressure developed during penetration. This implied that the reduction in hydraulic conductivity due to cementation will not result in undrained conditions during cone penetration.

3. Accuracy was assessed by comparing the uncemented results with three other investigations (Villet and Mitchell, 1981; Schmertmann, 1978; Baldi, 1981). Variations between the test results of this study and those of other investigations are attributed to the boundary conditions, $K_o$ values, diameter ratio, size and shape of the aggregates and specimen preparation procedures.

4. Two specimens of similar density were tested under two different boundary conditions 1 (constant stress) and 3 (zero lateral displacement). Both produced similar results and this was attributed to the diameter ratio in the present tests which is around 42. A study on the influence of diameter ratio on various cone test results showed that diameter ratio is less important when the relative density is lower than 40%. However, diameter ratios of 40 and above 50 are needed while testing specimens with 80 and 90 % relative density respectively.
5. Cementation increased tip resistance due to development of cohesion and friction resistance due to dilation.

6. In the theoretical approaches, two bearing capacity theories were used. Dur-gunoglu 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. 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 maximum dilation angle. This theory also rendered quite good comparisons.

7. Back calculations from sleeve friction values showed that a $K$ value higher than $K_o$ is needed to match the measured friction values. During cone penetration, the dilation around the cone is restrained. As a consequence, the confinement at the interface increases resulting in an increase in the measured $K$ value.

8. Bearing capacity theories and sleeve friction predictions are used in formulating a semi-empirical approach to predict cemented soil characteristics. This approach proposes a method to predict 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.

9. Two cavity expansion theories with different soil models are used. The first theory assumes the soil medium to be elastic - perfectly plastic. The second theory assumes elasto - plastic soil behavior. Dilation angles are properly represented in the second theory whereas the first theory assumes a constant dilation angle. Therefore, lower limiting pressures are predicted in the second theory. The ratio of tip resistances to limiting pressures are compared with the form factor suggested by Vesic (1977). There seems to be reasonable agreement between these values at relative densities of 45 - 55 % and 65 - 75 % range. However, there is variation at higher relative densities (above 80 %) and the reasons for this are well established.
10. An empirical approach based on the normalization of both tip and friction resistances is proposed. Normalized parameters, \( \alpha \) and \( \beta \) increase with both cementation and relative density. Other parameters, \( n \) and \( n_1 \) are considered as constants for a particular sand. Comparison of the results with various sands showed that \( n \) and \( n_1 \) values depend on the angularity of the sand particles and decrease with cement content.

### 8.3 Recommendations for Future Studies

The following topics are recommended for future studies in this area.

1. The accuracy of the proposed schemes can be improved if tests are conducted at higher cementation levels (4 and 6%), lower confining stresses (less than 100 kPa).

2. It seems that there is very little study on sleeve friction. It is necessary to better understand the development of sleeve friction.

3. This study shows that the effect of cementation on cone penetration testing can be predicted reasonably well with proposed theoretical models. However, field tests are necessary to validate the findings of this study.
References


- Dario de Lima (1989), Personal Communication.


• Kiousis, P. D., "Large Strain Theory as Applied to Penetration Mechanism in Soils," Ph.D. Dissertation, Civil Engineering department, Louisiana State University, Baton Rouge, August, 1985, 97 pages.


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• Sandven, R., “Strength and Deformation Properties of Fine Grained Soils Obtained From Piezocone Tests,” University of Trondheim, Trondheim, Norway, 1990.


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• Tumay, M.T., "Field Calibration of Electric Cone Penetrometer in Soft Soil - Executive Summary," Published by Louisiana Transportation Research Center, 1985.


Appendix A

A.1 Literature on Cemented Sands

In this section, various investigations and their findings are presented. Table 1 presents the various variables studied in each investigation. Table 2 presents the summary of conclusions of each investigation.
Table A.1. A Summary of Geotechnical Studies Conducted on Cemented Sands

<table>
<thead>
<tr>
<th>Reference</th>
<th>Type of Sand</th>
<th>Cementing Agent</th>
<th>Specimen Preparation/Retrieval</th>
<th>Curing Period</th>
<th>Types of Tests Conducted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wissa, et al. (1964, 1965)</td>
<td>Coarse Ottawa uniform sand and medium Ottawa sand</td>
<td>Portland cement</td>
<td>dry pluviation</td>
<td>3 days</td>
<td>CU and CD triaxial tests</td>
</tr>
<tr>
<td>Chiang and Chae (1972)</td>
<td>Uniform sand and silty clay</td>
<td>Cement, lime, fly ash</td>
<td>Compaction</td>
<td>14 days</td>
<td>Rasonant column tests</td>
</tr>
<tr>
<td>Harrel (1973)</td>
<td>Natural soils</td>
<td>Calcium carbonates</td>
<td>--</td>
<td>--</td>
<td>Direct shear tests</td>
</tr>
<tr>
<td>Mitchell (1976)</td>
<td>Monterey No. 0</td>
<td>Portland Cement</td>
<td>Compaction</td>
<td>NA</td>
<td>Indirect tension flexure</td>
</tr>
<tr>
<td>Yamanouchi, et al. (1977)</td>
<td>Natural</td>
<td>Thermal welding</td>
<td>Shirasu cutter (5 cm diameter tubes)</td>
<td>--</td>
<td>Several types</td>
</tr>
<tr>
<td>Salomone, et al. (1978)</td>
<td>Natural</td>
<td>Carbonates</td>
<td>76 m Denison sampler</td>
<td>--</td>
<td>Undrained triaxial tests, cyclic triaxial tests</td>
</tr>
<tr>
<td>Saxena and Lastrico (1978)</td>
<td>Natural cemented sand near Vincetion, New Jersey</td>
<td>Carbonates, calcite cement</td>
<td>NA</td>
<td>--</td>
<td>Isotropic consolidated triaxial tests, stress controlled cyclic triaxial tests</td>
</tr>
<tr>
<td>Dupas and Pecker (1979)</td>
<td>Medium-dense sands</td>
<td>Portland Cement</td>
<td>Compaction</td>
<td>7 days</td>
<td>CD triaxial tests, dynamic triaxial tests, longitudinal forced vibration studies</td>
</tr>
<tr>
<td>Frydman, et al. (1980)</td>
<td>Kurkar deposits near Israel</td>
<td>Calcareous materials</td>
<td>Block samples, freezing</td>
<td>--</td>
<td>(1) SPT (2) Cyclic triaxial tests</td>
</tr>
<tr>
<td>Poulos (1980)</td>
<td>Natural</td>
<td>Carbonates</td>
<td>Different methods</td>
<td>--</td>
<td>Strength tests</td>
</tr>
<tr>
<td>Clough, et al. (1981)</td>
<td>Monterey #0 Monterey #20</td>
<td>Silicates, ironoscles and Portland Cement</td>
<td>Compaction</td>
<td>14 days</td>
<td>Unconfined compression and drained triaxial compression tests</td>
</tr>
<tr>
<td>Reference</td>
<td>Type of Sand</td>
<td>Cementing Agent</td>
<td>Specimen Preparation/ Retrieval</td>
<td>Curing Period</td>
<td>Types of Tests Conducted</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>------------------------------------</td>
<td>-----------------</td>
<td>---------------------------------</td>
<td>---------------</td>
<td>---------------------------------------------------------------</td>
</tr>
<tr>
<td>Datta, et al. (1982)</td>
<td>Natural</td>
<td>Carbonates</td>
<td>NA</td>
<td>--</td>
<td>Isotropic compression tests, drained triaxial shear tests</td>
</tr>
<tr>
<td>Beringen, Kolk and Windle (1982)</td>
<td>Calcareous sediments at several off-shore locations</td>
<td>Calcareous material</td>
<td>Coring</td>
<td>--</td>
<td>(1) Cone penetration testing (2) Direct shear tests (3) Particle crushing tests</td>
</tr>
<tr>
<td>McKown and Ladd (1982)</td>
<td>Pierry shales in Nebraska</td>
<td>Calcium carbonate</td>
<td>Core drilling</td>
<td>--</td>
<td>(1) Leaching tests (2) Consolidation tests</td>
</tr>
<tr>
<td>Rad and Clough (1982)</td>
<td>Monterey #0 and natural deposits</td>
<td>Carbonates (natural) Portland Cement</td>
<td>Pluviation Compaction</td>
<td>14 days</td>
<td>Drained and undrained triaxial tests</td>
</tr>
<tr>
<td>Rad and Turnay (1984)</td>
<td>Monterey #0</td>
<td>Portland cement</td>
<td>Pluviation</td>
<td>7 to 14 days</td>
<td>Static penetration tests</td>
</tr>
<tr>
<td>Avramidis and Saxena (1985)</td>
<td>Monterey #0</td>
<td>Portland cement</td>
<td>Under compaction</td>
<td>15 days to 6 months</td>
<td>(1) Drained triaxial tests (2) Brazilian tests (3) Unconfined compression tests (4) Resonant column tests</td>
</tr>
<tr>
<td>Acar and Tahir (1986)</td>
<td>Monterey #0</td>
<td>Portland cement</td>
<td>Pluviation</td>
<td>14 days</td>
<td>Resonant column tests</td>
</tr>
<tr>
<td>Chang (1986)</td>
<td>Ottawa 20-30 Muskegon sand, mortar sand, medium sand</td>
<td>Sodium silicate, Portland cement, fly ash, lime</td>
<td>Injection and mix compaction methods</td>
<td>--</td>
<td>NA</td>
</tr>
<tr>
<td>Mitchell and Stone (1986)</td>
<td>Mortar sand</td>
<td>Cement</td>
<td>NA</td>
<td>14 days</td>
<td>Pullout tests</td>
</tr>
<tr>
<td>Loretta Li and Mitchell (1987)</td>
<td>Fine to medium sand</td>
<td>Portland cement</td>
<td>Mechanical mixer</td>
<td>14 days</td>
<td>Plane strain tests</td>
</tr>
<tr>
<td>Reference</td>
<td>Type of Sand</td>
<td>Cementing Agent</td>
<td>Specimen Preparation/Retrieval</td>
<td>Curing Period</td>
<td>Types of Tests Conducted</td>
</tr>
<tr>
<td>----------------------------</td>
<td>---------------------------------------------</td>
<td>-----------------</td>
<td>---------------------------------</td>
<td>--------------</td>
<td>---------------------------------------------</td>
</tr>
<tr>
<td>Riccobono (1987)</td>
<td>Louisiana river sand</td>
<td>Portland cement</td>
<td>Compaction</td>
<td>21 days</td>
<td>Triaxial</td>
</tr>
<tr>
<td>O'Rourke and Crespo (1988)</td>
<td>Moderately cement fine sand and silt sized particles. Natural near the Andes of Ecuador and Columbia</td>
<td>Amorphous silicate</td>
<td>NA</td>
<td>NA</td>
<td>Uniaxial and triaxial compressive strengths, Brasilian tensile strength tests</td>
</tr>
<tr>
<td>Saxena, et al. (1988)</td>
<td>Artificial Monterey #0</td>
<td>Portland Cement</td>
<td>Under compaction</td>
<td>15 to 60 days</td>
<td>(1) Cyclic triaxial tests</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(2) Resonant column tests</td>
</tr>
</tbody>
</table>
Table A.2. Synthesis of Data Reported in Studies Investigating Cemented Sands

<table>
<thead>
<tr>
<th>Reference</th>
<th>Parameters Studied</th>
<th>Conclusions from the Studies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wissa, et al. (1964, 1965)</td>
<td>D, C.C., t&lt;sub&gt;c&lt;/sub&gt;, shear resistance</td>
<td>(1) At low strains, the shearing resistance was due to cementation between grains.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2) At higher strains, cementation between grains were completely destroyed and effective stress-strength curve converge towards the origin on p vs. q plot.</td>
</tr>
<tr>
<td>Chiang and Chae (1972)</td>
<td>Cement content, confining pressure, shear strain amplitude, and moisture content</td>
<td>(1) Dynamic shear modulus and damping can be greatly increased by adding a small amount of cement.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2) Effect of cement content is more pronounced in cement treated cohesionless soils.</td>
</tr>
<tr>
<td>Hamel (1976)</td>
<td>Shear strength parameters</td>
<td>The in situ peak strength parameters of this desert alluvium probably lie between the peak values determined for granular specimens and specimens containing cemented lumps.</td>
</tr>
<tr>
<td>Sexena and Lastrico (1978)</td>
<td>n, γ&lt;sub&gt;u&lt;/sub&gt;, φ, σ&lt;sub&gt;c&lt;/sub&gt;, S&lt;sub&gt;u&lt;/sub&gt;</td>
<td>(1) The stress-strain behavior, the pore pressure response, the stress paths versus strain plots, the relation between undrained strength versus the consolidating confining pressures indicate that the strength behavior of the natural cemented soils are strain dependent.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2) At lower strains, the cohesion caused by the calcite cement bonding between particles is the major component of strength. At higher axial strains (around 1%), the cohesive strength is destroyed and then the frictional strength is predominant.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3) It was also observed that a high hydrostatic confining pressure can destroy cementation.</td>
</tr>
<tr>
<td>Dupas and Pecker (1979)</td>
<td>C.C., t&lt;sub&gt;c&lt;/sub&gt;, E, G, C, K, cyclic strength</td>
<td>(1) Investigated on the minimum amount of cement content required for a particular type of sand so that it will be in stable under both static and dynamic loading.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2) Another observation of this work was that only a small amount of cement is required to prevent liquefaction.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3) Methods have been developed for the interpretation of static and dynamic moduli from the static and dynamic triaxial tests.</td>
</tr>
</tbody>
</table>

Footnote: S - static tests; D - dynamic tests; D<sup>r</sup> - relative density; C.C. - cement content; t<sub>c</sub> - curing period; σ<sub>c</sub> - confining pressure; M<sub>t</sub> - deformation modulus; φ' - effective friction angle; S<sub>u</sub> - undrained shear strength; K - permeability; E - Young's modulus; G - shear modulus; γ<sub>d</sub> - dry density; ε<sub>v</sub> - volume change; q<sub>c</sub> - unconfined compressive strength; q<sub>e</sub> - cone resistance; f<sub>r</sub> - frictional resistance; f<sub>f</sub> - friction ratio; e - void ratio
### Table A.2 (continued)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Parameters Studied</th>
<th>Conclusions from the Studies</th>
</tr>
</thead>
</table>
| Frydman, et al. (1980)     | $\sigma_c$, cyclic strength parameters, stress ratio, $\gamma$ | (1) The cyclic strength of intact specimens may be similar to that of reconstituted specimens compacted to the same inferred relative density calculated from the in situ blow counts using the Gibbs and Holtz correlations.  
(2) The block sampling and freezing techniques were found to be satisfactory methods for preparing intact specimens of granular soils. |
| Clough, et al. (1981)      | C.C., $D_c$, grain arrangement, strength of deformation parameters | (1) A weakly cemented sand shows a brittle failure mode at low confining pressures with a transition to ductile failure at higher confining pressures.  
(2) Volumetric strain increases during shear occur at a faster rate and at a smaller strain for cemented sand than uncemented sands.  
(3) The residual strength of a cemented sand is close to that of an uncemented sand, although some degree of residual cohesion was observed for all the cemented sands investigated.  
(4) The tensile strength of a cemented sand is about 10% of the unconfined compressive strength. |
| Beringen, et al. (1982)    | $w_c$, $I_p$, carbonate content, $c_u$, shearing resistance, $q_u$, $f_u$, $f$ | The study which conducted both cone penetration testing and lab testing in marine calcareous sediments revealed that the in situ testing (cone penetration testing) can dramatically improve the soil classification. The tests also showed that the cone penetration test results from the cemented (carbonate) soils can be interpreted using the principles established for noncarbonate soils. Many examples are quoted in this study to show the importance of performing cone penetration testing when engineering strength parameters are needed for design. |
| Datta, et al. (1982)       | $\alpha_s$, $\phi$, crushing coefficient | This study of engineering behavior of carbonate soils of India reveals that the crushing of carbonate particles and cementation by carbonate materials are the two most dominating factors which influence the engineering behavior of carbonate soils. |
| McKown and Ladd (1982)     | $e$-$\log p$ curves, calcium carbonate content | The following are some conclusions from the consolidation and leaching tests performed on undisturbed specimens from a deposit of Pierre shale located in Northeast Nebraska:  
(1) The results support that natural cementation can have a significant effect on the apparent maximum past pressure.  
(2) Reduction of $CaCO_3$ due to leaching caused an increase in compressibility during recompression and a lower measured apparent maximum past pressure. |

Footnote: S - static tests; D - dynamic tests; $D_r$ - relative density; C.C. - cement content; $I_p$ - curing period; $\sigma_c$ - confining pressure; $M_d$ - deformation modulus; $\phi'$ - effective friction angle; $S_u$ - undrained shear strength; $K$ - permeability; $E$ - Young's modulus; $G$ - shear modulus; $\gamma_d$ - dry density; $\varepsilon_v$ - volume change; $q_u$ - unconfined compressive strength; $q_c$ - cone resistance; $f_s$ - frictional resistance; $f_r$ - friction ratio; $e$ - void ratio
### Table A.2 (continued)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Parameters Studied</th>
<th>Conclusions from the Studies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rad and Clough</td>
<td>C.C. M, e, consolidation pressure, strength</td>
<td>(1) The response of cemented sands to load is a function of the level of cementation, relative density and confining pressure.</td>
</tr>
<tr>
<td>(1982)</td>
<td>parameters, liquefaction, O.C.R.</td>
<td>(2) Increasing the cement content increases the cohesion intercept, while it has little effect on the friction angle.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3) Cyclic shear resistance curves for cemented sands have essentially the same form as for uncemented sand.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(4) Available models to predict pore-pressure build-up in pure sands under repeated loading may need to be modified for cemented sands.</td>
</tr>
<tr>
<td></td>
<td>C.C., D, q, t, f, σ, strength components</td>
<td></td>
</tr>
</tbody>
</table>

This investigation provides an insight into the effect of cementation on the cone penetration resistance of sands. The major conclusions from this study are:

1. Cementation has a pronounced effect on the 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 uncemented 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 resistances and sleeve frictions and lower friction ratios.

3. The effect of cementation on the cone penetration resistance of sand is similar to that of relative density. Utilizing the available correlations 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.

**Footnote:** S - static tests; D - dynamic tests; D_r - relative density; C.C. - cement content; t_e - curing period; α_p - confining pressure; M_t - deformation modulus; \( \phi' \) - effective friction angle; S_u - undrained shear strength; K - permeability; E - Young's modulus; G - shear modulus; \( \gamma_d \) - dry density; \( \epsilon_v \) - volume change; \( q_u \) - unconfined compressive strength; \( q_c \) - cone resistance; \( f_t \) - frictional resistance; \( f_f \) - friction ratio; e - void ratio
<table>
<thead>
<tr>
<th>Reference</th>
<th>Parameters Studied</th>
<th>Conclusions from the Studies</th>
</tr>
</thead>
</table>
| Acar and Tahir (1986)           | C.C., D, Gmax, q, stiffness ratio                      | (1) The tamping method used in specimen preparation scheme leads to higher unconfined strength than in the tapping scheme.  
(2) The increase in dynamic shear modulus of artificially cemented specimens at low levels of cementation is due to an increase in stiffness coefficient.  
(3) Cementation leads to a decrease in damping ratio at all levels of strain.  
(4) It is determined that an increase in the degree of cementation leads to a rapid decay of modulus with increasing strains. It is observed that this modulus decay is more dominant in specimens with high stiffness ratios.  
(5) The relative increase in the stiffness coefficient with cementation could be expressed with stiffness ratio, R. This ratio is nonlinearly related to both the degree of cementation and void ratio. |
| Avramidis and Saxena (1986)     | Strength parameters, dilatancy, pore pressure, cyclic strength | (1) Cohesion, angle of internal friction increases with the increase in cement content.  
(2) For cemented sands, the peak strength is reached when the summation of all strength components reaches its maximum whereas for uncemented sands, the peak strength is reached when the rate of dilatancy is maximum.  
(3) Small amount of cement increases significantly the cyclic strength of uncemented sands. This cyclic strength increases with relative density and curing period.  
(4) For cemented sands before an pore water pressure generation, the cementation bond has to break. This requires a certain number of loading cycles. |
| Chang (1986)                    | G, C.C., damping ratio                                  | (Could not locate the original paper.)                                                                 |
| Mitchell and Stone (1986)       | C.C., q, φ                                            | In this work, the use of reinforcements in cemented fill to reduce the cement usage is studied. It is found that strong cemented layers at typical spacings of about 3 m in a low cement content bulk fill can reinforce the fill and reduce the overall cement usage. |
| Li and Mitchell (1987)          | C.C., stress-strain relationships, alignment or orientations of mesh elements | This study investigated the role of mesh element reinforcements and the anchored reinforcements in increasing the strength and ductility of sandfills. The major conclusions from this study are: The reinforcements are effective in increasing the strength and ductility of the cemented sand fills. But the other type of reinforcements, the smooth and deformed failure reinforcements were not as effective. |

Footnote:  S - static tests; D - dynamic tests; D, - relative density; C.C. - cement content; t, - curing period; σc - confining pressure; M - deformation modulus; φ - effective friction angle; S, - undrained shear strength; K - permeability; E - Young's modulus; G - shear modulus; γd - dry density; εv - volume change; q - unconfined compressive strength; q, - cone resistance; f, - frictional resistance; f - friction ratio; εv - void ratio
Table A.2 (continued)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Parameters Studied</th>
<th>Conclusions from the Studies</th>
</tr>
</thead>
</table>
| Riccobono (1987) | C.C., stress-strain curves | (1) At low axial strain, the cementation results in an elastic behavior. Cementation also induces an apparent cohesion of approximately 120 kPa and a slight increase in internal friction angle.  
(2) At large strains, the cementation between grains is completely destroyed and therefore the residual shearing resistance of the cemented sand approaches that of sand.  
(3) As a result of coupled reinforcement and drainage effect, the cementation reduces the settlement of the reinforced soft soil by about 50% |
| O'Rourke and Crespo (1988) | Index properties uniaxial compressive strength, Brazil tensile strength, peak and residual strengths, $S_u$, $M_s$, principal stress-strain curves | The study which has focused on the geotechnical properties of volcanielastic formation has yielded the following conclusions:  
(1) The Brazil tensile strength is usually high and is 18 to 29% of the uniaxial compressive strength.  
(2) This formation exhibits brittle failure mode at low confining pressure with a transition to ductile at high confining stresses.  
(3) Increasing degrees of saturation cause a shift from brittle to ductile failure at constant confining pressure.  
(4) Material properties such as tensile strength and fracture toughness play an important role in explaining and evaluating slope failures in the cemented formations found in the Andes of Ecuador and Colombia. |
| Saxena, et al. (1988) | $D_o$, C.C., $t_o$, $\sigma_v$, stress ratio, $e$, dynamic modulus, cyclic strength | This study is devoted to discussing the factors affecting liquefaction resistance and to investigate the correlation between the dynamic moduli and cyclic strength of cemented sands.  
(1) Small amount of cement increases significantly the cyclic strength of uncemented sands. The cyclic strength and the pore pressure development curves are corresponding to cemented sands are similar to those for uncemented sands. The cyclic strength in cemented sands increases with relative density and curing periods.  
(2) Nondimensional empirical relationships are developed for dynamic shear and Young's modulus and damping ratio.  
(3) Damping ratio initially increase and then decrease as cement content increases from zero percent to eight percent. |

Footnote:  S - static tests; D - dynamic tests; $D_o$ - relative density; C.C. - cement content; $t_o$ - curing period; $\sigma_v$ - confining pressure; $M_s$ - deformation modulus; $\phi'$ - effective friction angle; $S_u$ - undrained shear strength; K - permeability; E - Young's modulus; G - shear modulus; $\gamma_d$ - dry density; $e_v$ - volume change; $u_c$ - unconfined compressive strength; $q_c$ - cone resistance; $f_r$ - frictional resistance; $f_f$ - friction ratio; $e$ - void ratio
A.2 Dynamic Properties ($G_{max}$)

Figure A.1 and A.2 presents the variation of maximum shear modulus versus confining stress for cemented specimens of relative densities 35 and 80 % respectively.

Figure A.1: The Variation of $G_{max}$ Versus Confining Stress for Cemented Specimens

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Figure A.2: The Variation of $G_{max}$ Versus Confining Stress for Cemented Specimens

Acar and El-Tahir (1986)
### A.3  Cone Test Results on Other Sands

In this section, various cone test results of the investigations presented in section 2.3.3 are presented.

#### Table A.3: Test Results - Eid (1987)

<table>
<thead>
<tr>
<th></th>
<th>( D_r )</th>
<th>( \sigma_v )</th>
<th>( q_c )</th>
<th>( f_s )</th>
<th>D.R.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>64.2</td>
<td>100</td>
<td>19.19</td>
<td>1.24</td>
<td>65</td>
</tr>
<tr>
<td>2</td>
<td>64.1</td>
<td>100</td>
<td>19.55</td>
<td>1.22</td>
<td>65</td>
</tr>
<tr>
<td>3</td>
<td>61.4</td>
<td>100</td>
<td>18.89</td>
<td>1.45</td>
<td>65</td>
</tr>
<tr>
<td>4</td>
<td>65.1</td>
<td>100</td>
<td>19.01</td>
<td>1.22</td>
<td>65</td>
</tr>
<tr>
<td>5</td>
<td>60.0</td>
<td>200</td>
<td>35.08</td>
<td>2.07</td>
<td>65</td>
</tr>
<tr>
<td>6</td>
<td>60.9</td>
<td>100</td>
<td>15.62</td>
<td>0.94</td>
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Table A.5: Test Results - Villet and Mitchell (1981)

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Table A.9: Test Results - Nutt and Houlsby (1992)

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Table A.10: Test Results - Lhuer (1976)

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Appendix B

B.1 CU Tests

The results of consolidation undrained triaxial tests are presented in this section. Cemented results (C.C. 2%) are reported in this section.

Figure B.1: Undrained Triaxial Test on Cemented Monterey No. 0/30 Sand ($D_r = 50\%$; C.C. 2%)
Figure B.2: Undrained Triaxial Test on Cemented Monterey No. 0/30 Sand ($D_r = 65\%$; C.C. 2\%)

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Figure B.3: Undrained Triaxial Test on Cemented Monterey No. 0/30 Sand ($D_r = 85\%$; C.C. 2\%)
B.2 Stress Paths

Stress paths of the undrained test results are shown in the figures in this section.

Figure B.4: Stress Paths of CU Test ($D_r = 45 \%$; C.C. 0 \%)

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Figure B.5: Stress Paths of CU Test ($D_r = 45\%$; C.C. 1%)
Figure B.6: Stress Paths of CU Test ($D_r = 45 \%$; C.C. 2 %)
Figure B.7: Stress Paths of CU Test ($D_r = 65\%$; C.C. 0\%)

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Figure B.8: Stress Paths of CU Test ($D_r = 65\%$; C.C. $1\%$)

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Figure B.9: Stress Paths of CU Test ($D_r = 65 \%$; C.C. 2 \% )
B.3 Flow Charts and Listings

The flow chart and listing of the three programs used in the chapter 4 are presented in this section. The first program reads the drained data and calculates stress ratio and dilation angle. The second program simulates the drained triaxial behavior. The third program simulates undrained triaxial behavior.

Flow Chart: Program 1

Figure B.10: Flow Chart of Program 1
Listing Of Program 1

C*************************************************************************
C Program 1  (Anand Puppala)
C*************************************************************************
DIMENSION EPS1(20),RATSIG(20),EPSV(20),SIG1(20),Q(20),PE(20),
  & GAMA(20),EPS3(20),A1(20),A2(20),A3(20),
  & DEPSV(20),DGAMA(20),DPE(20),DQ(20),DEPSVE(20),DGAMAE(20),
  & DEPSVP(20),DGAMAP(20),A4(20),A5(20)
OPEN(8,FILE='GEOIP.OUT',STATUS='NEW')
WRITE(8)
15    FORMAT(5X,'THE FOLLOWING ARE DRAINED TEST RESULTS',
      & 'I',4X,'SIG1(I)',9X,'Q(I)',10X,
      & 'GAMA(I)',8X,/)  
   eps1(1)=0.5  
    eps1(2)=1.  
     eps1(3)=1.75  
      eps1(4)=3.5  
       eps1(5)=5.25  
        eps1(6)=7.5  
         eps1(7)=10.5  
          eps1(8)=13.5  
   EPSV(1)=0.2  
    EPSV(2)=0.22  
     EPSV(3)=0.3  
      EPSV(4)=0.25  
       EPSV(5)=0.06  
        EPSV(6)=-0.09  
         EPSV(7)=-0.45  
          EPSV(8)=-0.578  
   RATSIG(1)=2.25  
    RATSIG(2)=2.88  
     RATSIG(3)=3.03  
      RATSIG(4)=3.34  
       RATSIG(5)=3.43  
        RATSIG(6)=3.50  
         RATSIG(7)=3.57  
          RATSIG(8)=3.57  
SIG0=100.  
DO 10 I=1,8

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SIG1(I)=RATSIG(I)*SIG0
Q(I)=SIG1(I)-SIG0
PE(I)=(SIG1(I)+2SIG0)/3
EPS3(I)=(EPSV(I)-EPS(I))/2
GAMA(I)=EPS1(I)-EPS3(I)

A1(I)=Q(I)/PE(I)
A2(I)=GAMA(I)/A1(I)
A3(I)=EPS3(I)/EPS1(I)

10 WRITE(8,*)I,SIG1(I),Q(I),PE(I),GAMA(I)

WRITE(8,45)
45 FORMAT(//,3X,'EPS3(I)',10X,'Q/PE',8X,'GAMA/(Q/PE)',4X,
          & 'EPS3/EPS1',//)

DO 40 I=1,8
40 WRITE(8,*)EPS3(I),A1(I),A2(I),A3(I)

AM=.25
G=400
E=1000

WRITE(8,55)
55 FORMAT(//,2X,'DEPSVP(I)',9X,'DGAMAP(I)',7X,'(DQ/DPE)',4X,
          & 'DEPSVP/DGAMAP',//)

DO 50 I=1,8
IF(I.EQ.1)THEN
  DEPSV(I)=EPSV(1)
  DGAMA(I)=GAMA(1)
  DPE(I)=PE(1)-SIG0
  DQ(I)=Q(1)
ELSE
  DEPSV(I)=EPSV(I)-EPSV(I-1)
  DGAMA(I)=GAMA(I)-GAMA(I-1)
  DPE(I)=PE(I)-PE(I-1)
  DQ(I)=Q(I)-Q(I-1)
ENDIF
DEPSVE(I)=((1-2*AM)/E)*3*DPE(I)
DGAMAE(I)=DQ(I)/(2*G)
DEPSVP(I)=DEPSV(I)-DEPSVE(I)
DGAMAP(I)=DGAMA(I)-DGAMAE(I)
A4(I)=DEPSVP(I)/DGAMAP(I)
A5(I)=DQ(I)/DPE(I)
50    WRITE(8,*)DEPSVP(I),DGAMAP(I),A5(I),A4(I)
STOP
END
Flow Chart: Program 2

START

Input Model Parameters
\( G, \mu_1, \mu_2, M_{p, 0}, M_{p, 0}/C_{\text{ref}} \)
\( \gamma = 0.0 \)

\( \delta y = 0.005, \gamma = \gamma + \delta y \)

Calculate \( h(\gamma) \)

\( h(\gamma) < M_{p, 0} \)

Yes

Use \( \mu_1 \)

Use \( \mu_2 \)

Calculate \( q, p' \)

Compute \( d_z \) and \( e_z = e_z + d_z \)

\( \gamma \leq 0.15 \)

Yes

Output Parameters
\( q, p', \gamma, e_z \)

Do Results compare with experiments?

No

Yes

OUTPUT MODEL PARAMETERS

END

Assume Different \( G, \mu_1, \mu_2 \)

Figure B.11: Flow Chart of Program 2
Listing of Program 2

DIMENSION EPS1(100), SIG(100), EPSV(100), SIG1(100), Q(100) & PE(100), GAMA(100), EPS3(100), H(100), D(100), AA1(100), aa2(100), & DEPSV(IOO), DPE(100), DQ(100), DEPSVE(100), DGAMAE(100), & DEPSVP(100), DGAMAP(100), A1(100), DEPS1(110), DEPS3(100), & GGAMA(100), EEPSV(100), QQ(100), EEPS1(100)

REAL MU, MU1, MU2, MPHICV
OPEN(8, FILE='y180.dat', STATUS='NEW')
v = 0.3

C E=2.0*G*(1.0+v)
PHI=(38.0/180)*3.143
PHICV=(35.4/180)*3.143
CP=6.2
CR=7.50
MPHI=(6*SIN(PHI))/(3-SIN(PHI))
MPHICV=(6*SIN(PHICV))/(3-SIN(PHICV))

C
C MPHI=1.55
C
C MPHICV=1.4
MU1=1.5
MU2=0.54
P1=CP/TAN(PHI)
SIG0=100.0
G= 157.1 * Sig0
E=2*G*(1+v)
DGAMA=.005
RAT=1+SQRT(1-MPHICV/MPHI)
A=-(2*SIG0*(MPHI*RAT)**2/(MPHICV*G))
B=SIG0*MPHI*RAT/G
C=MPHICV
GAMA(1)=0
H(1)=0
Q(1)=0
QQ(1)=0
SIG1(1)=SIG0
PE(1)=SIG0
EPSV(1)=0
EPS1(1)=0
EPS3(1)=0
D(1)=0
A1(1)=0
DO 10 I=2,50

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GAMA(I)=GAMA(I-1)+DGAMA
H(I)=C*(GAMA(I)*((GAMA(I)-A)/((GAMA(I)+B)**2)
IF(H(I).LE.MPHICV)THEN
MU=MU1
C      P1=CP*MPHI/TAN(PHI)
ELSE
MU=MU2
C      P1=CR*MPHIC/TAN(PHICV)
ENDIF
Q(I)=3*SIG0*H(I)/(3-H(I))
PE(I)=3*SIG0/(3-H(I))
PE(I)=PE(I)-P1
QQ(I)=Q(I)/PE(I)
SIG1(I)=Q(I)+SIG0
DQ(I)=Q(I)-Q(I-1)
DPE(I)=PE(I)-PE(I-1)
DGAMA(I)=DQ(I)/(2*G)
DGAMAP(I)=DGAMA-DGAMA(I)
DEPSVE(I)=((1-2*v)/E)*3*DPE(I)
C      WRITE(*,*)'HI',DGAMAP(I),DGAMA(I),DPE(I),'E',E
DEPSVP(I)=DGAMAP(I)*(MPHICV-H(I))/MU
C      WRITE(*,*)'HI2',DEPSVE(I),DEPSVP(I)
DEPSV(I)=DEPSVE(I)+DEPSVP(I)
C      WRITE(*,*)DEPSV(I)
EPS1(I)=EPS1(I-1)+DEPS1(I)
DEPS3(I)=(2*DGAMA+DEPSV(I))/3
EPS3(I)=EPS3(I-1)+DEPS3(I)
D(I)=DEPSVP(I)/DGAMAP(I)
A1(I)=Q(I)/PE(I)
GGAMA(I)=GAMA(I)*100.0
EEPSV(I)=EPSV(I)*100.0
C      WRITE(8,*)GAMA(I),A1(I),EPSV(I)
10   CONTINUE
DO 20 K=1,50
20   WRITE(8,*)GGAMA(K),QQ(K),EEPSV(K)
C20   WRITE(8,*)GGAMA(K),A1(K),EEPSV(K)
C20   WRITE(8,*)H(K),MPHICV,MU,EEPSV(K)
STOP
END
Flow Chart: Program 3

START

Input Model Parameters
$G, \mu_1, \mu_2, M_{p,0}, M_{p,OV}$

$dy = 0.005, \gamma = \gamma + dy$

Calculate $h(\gamma)$

$h(\gamma) < M_{p,OV}$

Yes

Use $\mu_1$

No

Use $\mu_2$

Calculate $q, p, p'$

Using $\epsilon = 0$

Compute $\sigma_1, \sigma_3$

Pore Pressure $U = \sigma_3 - \sigma_0$

$\gamma \leq 0.15$

No

Yes

Output Parameters $q, p, \gamma, \epsilon$

Do Results Compare with Experiments?

No

Yes

OUTPUT MODEL PARAMETERS

END

Figure B.12: Flow Chart of Program 3
Listing of Program 3

C*******************************************************************************
C PROGRAM 3 UNDRAINED SIMULATION (Anand Puppala)
C*******************************************************************************
C UNDRAINED SIMULATION PROGRAM 3
DIMENSION EPS1(100),SIG1E(100),SIG3E(100),SIG1(100),Q(100),
   & PE(100),GAMA(100),EPS3(100),H(100),D(100),DH(100),T(100),
   & SE(100),S(100),DPE(100),DQ(100),DEPSVE(100),DGAMAE(100),P(100),
   & DEPSVP(100),DGAMAP(100),A1(100),DEPS1(110),DEPS3(100),U(100),
   & RGAMA(100),AA1(100),REPS1(100),RQ(100),RATO(100)
REAL MU,MU1,MU2,MPHI,MPHICV
OPEN(8,FILE='U185.DAT',STATUS='NEW')
SIG0=100.0
G=193.0*SIGO
v=0.3
E=2*G*(1+v)
MPHI=1.75
MPHICV=1.52
MU1=0.55
MU2=0.96
DGAMA=.005
RAT=1+SQRT(1-MPHICV/MPHI)
A=-(2*SIG0*(MPHI*RAT)**2/(MPHICV*G))
B=SIG0*MPHI**RAT/G
C=MPHICV
GAMA(1)=0
RGAMA(1)=0.0
REPS1(1)=0.0
H(1)=0.0
Q(1)=0.0
RQ(1)=0.0
RATO(1)=0.0
SIG1(1)=SIG0
PE(1)=SIG0
S(1)=SIG0
SE(1)=SIG0
U(1)=0.0
T(1)=0.0
EPS1(1)=0
EPS3(1)=0
D(1)=0
A1(1)=0
DO 40 I=2,90
   GAMA(I)=GAMA(I-1)+DGAMA
H(I) = C*GAMA(I)*(GAMA(I)-A)/((GAMA(I)+B)**2)
IF(H(I) .LE. MPHICV) THEN
  MU = MU1
ELSE
  MU = MU2
ENDIF
DH(I) = H(I) - H(I-1)
DPE(I) = E*(2*G*DGAMA-PE(I-1)*DH(I))*(MPHICV-H(I))/
  & (6*G*MU*(2*V-1)+E*H(I)*(MPHICV-H(I)))
IF(DPE(I) .LE. 0.0) GOTO 15
GOTO 25
15  DPE(I) = 0.0
C  WRITE(8,*) i, 'DP', DPE(i), 'DH', Dh(i)
25  DQ(I) = H(I)*DPE(I)+PE(I-1)*DH(I)
C  WRITE(8,*) i, DQ(I), 'DQ HERE'
PE(I) = PE(I-1) + DPE(I)
RATIO(I) = Q(I)/PE(I)
SIG1E(I) = PE(I)+(2*Q(I)/3)
SIG3E(I) = PE(I) - Q(I)/3
U(I) = SIG0 - SIG3E(I)
SIG1(I) = SIG1E(I) + U(I)
DGAMAE(I) = DQ(I)/(2*G)
DGAMAP(I) = DGAMA - DGAMAE(I)
DEPS1(I) = 2*DGAMA/3
DEPS3(I) = -DGAMA/3
EPS1(I) = EPS1(I-1) + DEPS1(I)
EPS3(I) = EPS3(I-1) + DEPS3(I)
REPS1(I) = EPS1(I)*100.0
S(I) = (SIG1(I)+SIG0)/2
SE(I) = (SIG1E(I)+SIG3E(I))/2
T(I) = (SIG1(I)-SIG0)/2
P(I) = PE(I) + U(I)
A1(I) = Q(I)/PE(I)
RGAMA(I) = GAMA(I)*100.0
RQ(I) = Q(I)*2.0
C  WRITE(8,*) RGAMA(I), A1(I), U(I)
40  CONTINUE
C  T(2) = 0.
D0 50 I = 1, 90
50  WRITE(8,*) RGAMA(I), A1(I), U(I)
C  WRITE(8, 45)
C45  FORMAT(//'EPS1', 'U', 'S', 'SE', 'T',/)
Appendix C

C.1 Cone Test Results

The MQSC test results conducted on both cemented and uncemented specimens are presented in this section.
Figure C.1: Cone Penetration Test Results on a Specimen ($D_r = 48.7\%$; $\epsilon_{max} = 0.85$; $\epsilon_{min} = 0.56$ and 100 kPa)
Figure C.2: Cone Penetration Test Results on a Specimen \( (D_r = 56.4\% ; \epsilon_{max} = 0.85; \epsilon_{min} = 0.56 \text{ and } 200 \text{ kPa}) \)
Figure C.3: Cone Penetration Test Results on a Specimen ($D_r = 54.8\%$; $e_{max} = 0.85$; $e_{min} = 0.56$ and 300 kPa)
Figure C.4: Cone Penetration Test Results on a Specimen ($D_r = 90.0\%$; $\epsilon_{\text{max}} = 0.85$; $\epsilon_{\text{min}} = 0.56$ and 100 kPa)
Figure C.5: Cone Penetration Test Results on a Specimen ($D_r = 90.0\%$; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$ and 200 kPa)
Figure C.6: Cone Penetration Test Results on a Specimen ($D_r = 88.0\%$; $\epsilon_{max} = 0.85$; $\epsilon_{min} = 0.56$ and 300 kPa)
Figure C.7: Cone Penetration Test Results on a Specimen ($D_r = 86.0 \%$; $\varepsilon_{max} = 0.85$; $\varepsilon_{min} = 0.56$ and 100 kPa)
Figure C.8: Cone Penetration Test Results on a Specimen ($D_r = 84.0\%$; $\epsilon_{max} = 0.85$; $\epsilon_{min} = 0.56$ and 100 kPa)
Figure C.9: Cone Penetration Test Results on a Specimen ($D_r = 86.0\%$; C.C. = 1\%; $\varepsilon_{max} = 0.85$; $\varepsilon_{min} = 0.56$ and 100 kPa)

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Figure C.10: Cone Penetration Test Results on a Specimen ($D_r = 84.6\%$; C.C. = 1\%; $\epsilon_{\text{max}} = 0.85$; $\epsilon_{\text{min}} = 0.56$ and 200 kPa)

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Figure C.11: Cone Penetration Test Results on a Specimen ($D_r = 89.2\%$; C.C. = 1 $\%$; $e_{\max} = 0.85$; $e_{\min} = 0.56$ and 300 kPa)
Figure C.12: Cone Penetration Test Results on a Specimen ($D_r = 48.8\%$; C.C. = 1\%; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$ and 100 kPa)
Figure C.13: Cone Penetration Test Results on a Specimen ($D_r = 46.6\%$; C.C. = 1\%; $\epsilon_{max} = 0.85$; $\epsilon_{min} = 0.56$ and 200 kPa)
Figure C.14: Cone Penetration Test Results on a Specimen ($D_r = 53.2\%$; C.C. = 1\%; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$ and 300 kPa)

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Figure C.15: Cone Penetration Test Results on a Specimen ($D_r = 88.2\%$; C.C. = 2\%; $\epsilon_{\text{max}} = 0.85$; $\epsilon_{\text{min}} = 0.56$ and 100 kPa)
Figure C.16: Cone Penetration Test Results on a Specimen ($D_r = 86.3\%$; C.C. = 2\%; $\epsilon_{\text{max}} = 0.85$; $\epsilon_{\text{min}} = 0.56$ and 200 kPa)

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Figure C.17: Cone Penetration Test Results on a Specimen ($D_r = 84.2\%$; C.C. = 2\%; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$ and 300 kPa)

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Figure C.18: Cone Penetration Test Results on a Specimen ($D_r = 47.2\%$; C.C. = 2\%; $\epsilon_{max} = 0.85$; $\epsilon_{min} = 0.56$ and 100 kPa)
Figure C.19: Cone Penetration Test Results on a Specimen ($D_r = 54.4\%$; C.C. = 2\%; $e_{\text{max}} = 0.85$; $e_{\text{min}} = 0.56$ and 200 kPa)
Figure C.20: Cone Penetration Test Results on a Specimen ($D_r = 52.0\%$; C.C. = 2%; $\varepsilon_{\text{max}} = 0.85$; $\varepsilon_{\text{min}} = 0.56$ and 300 kPa)
Appendix D

D.1 Program Listings for Cavity Expansion Models
The listing of the cavity expansion simulation program is presented in this appendix. This is based on the closed form solutions proposed by Carter et. al., 1986.

Program Listing

C Cavity Expansion Simulation (Anand Puppala)
C Closed Form Solutions Carter et al., 1986
C************************************************************
DIMENSION PL(600),ERR(600),RTERR(600),PL1(600),QC(600)
REAL K,M,N,LEFT1,NU,K0
OPEN(8,FILE='z',STATUS='NEW')
C Shear modulus from previous model
G=370000.0
C Soil properties
WRITE(*,*)'Cohesion ?'
READ(*,*)C
C C= 0.0
WRITE(*,*)'PHI(PEAK) ? '
READ(*,*)PHII
WRITE(*,*)'PHI(RES) ?
READ(*,*)PHICI
PHI=(PHII*3.1428/180.0)
PHIC=(PHICI*3.1448/180.0)
PC=(3.1428/4)+(PHIC/2)
PC1=(3.1428/2)-PHIC
C MPHI=38.9
C DIL=-33.0
C Spherical cavity
WRITE(*,*)'K = ? (1 FOR SPH OR 2 FOR CYL)'
READ(*,*)K
C K=1.0
C Coeff. of earth pressure at rest
WRITE(*,*)'K0 = ?'
READ(*,*)K0
C K0=0.43
C Initial confining pressure
WRITE(*,*)'SIGO = ?'
READ(*,*)SIGO
WRITE(*,*)'G/SIGO'
READ(*,*)XX
G=XX*SIGO
C SIGO=200.0
C Dilational angle
WRITE(*,*)'DIL ANGLE = ?'
READ(*,*)DILA
DILA=(DILA*3.14/180.0)
C Octahedral stresses and poisson ratio
P0=(((1+2*K0)/3)*SIG0)
NU=0.3
C Initial limiting pressure
PL(1)=20000.0
C Equations
M=(1+SIGN(DILA))/(1-SIGN(DILA))
N=(1+SIGN(PHI))/(1-SIGN(PHI))
ALPHA=K/M
BETA=1-K*((N-1)/N)
GAMA=((1+K)/(N+K))*N*P0
UP1=K*(1-NU)-K*NU*(M+N)+((K-2)*NU+1)*M+N
DOWN1=((K-1)*NU+1)*M+N
JAY=UP1/DOWN1
Z=(K+1)*K*JAY/(ALPHA+BETA)
T=(K+1)*(K*JAY/(ALPHA+BETA))
C WRITE(*,*)'A',ALPHA,BETA,'I',UP1,DOWN1,JAY
C WRITE(*,*)'Z,T','HERE',M,N
C Iterations
DO 10 I=2,590
PL(I)=PL(I-1)+50.0
P01=(P0+C/TAN(PHI))
SIGR1= (SIGR+C/TAN(PHI))
PL1(I)=PL(I)+(C/TAN(PHI))
C WRITE(*,*)'P01','I',SIGR1
C Calculating left and right side of the equations and
C comparing for various limiting pressures
LEFT1= 2*G*(N+K)/(P01*(N-1))
RIGHT1=(T*((PL1(I)/SIGR1)**GAMA))-Z*(PL1(I)/SIGR1)
ERR(I)=ABS(RIGHT1-LEFT1)
RTERR(I)=SQRT(ERR(I))
C IF (RTERR(I).LE.0.25) GOTO 40
C GOTO 40
C SHAPE FACTOR
S1=(TAN(PC)*TAN(PC))
S2=1/(1+SIGN(PHIC))
S3=PC1*TAN(PHIC)
S=S1*S2*EXP(S3)
QC(I)=S*PL(I)
WRITE(8,*')I,PL(I),LEFT1,RIGHT1,S
C RTERR(I), ERR(I)
10 CONTINUE
STOP
END
The following is the cavity expansion program based on the second procedure.

**Program Listing**

```plaintext
C******************************************************************************************************************************************
C Cavity Expansion Program - Anand J. Puppala
C Soil Model - Elasto-Plastic
C******************************************************************************************************************************************
DIMENSION EPS1(200),SIG(200),EPSV(200),SIG1(200),Q(200),
 & PE(200),GAMA(200),EPS3(200),H(200),D(200),AA1(200),AA2(200),
 & DEPSV(200),DPE(200),DQ(200),DEPSVE(200),DGAMAE(200),
 & DEPSVP(200),DGAMAP(200),A1(200),DEPS1(200),DEPS3(200),
 & GGAMA(200),DEPSV(200),QQ(200),EEPS1(200),
 & EPTT(200),DIL(200),RAT1(200),RAT2(200),
 & PSI(200),SIGC(200),DSIG(200)
REAL MU,MU1,MU2
OPEN(8,FILE='c.dat',STATUS='NEW')
v=0.3
WRITE(*,*)'SPHI ?'
READ(*,*) SPHI
WRITE(*,*)'SPHICV ?'
READ(*,*) SPHICV
WRITE(*,*)'MU1 ?'
READ(*,*) MU1
WRITE(*,*)'MU2 ?'
READ(*,*) MU2
WRITE(*,*)'SIGC ?'
READ(*,*) SIGC
WRITE(*,*)'G/SIGC ?'
READ(*,*) CC
WRITE(*,*)'C ?'
READ(*,*) C
G = CC * Sig0
E = 2 * G * (1 + v)
RAT = 1 + SQRT(1 - SPHICV/SPHI)
A = -(4 * SIG0 * ((SPHI*RAT)**2)/(SPHICV*G))
B = 2 * SIG0 * SPHI * RAT / G
C = SPHICV
GAMA(1) = 0
H(1) = 0
Q(1) = 0
QQ(1) = 0
SIG1(1) = SIG0
PE(1) = SIG0
```

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SIGC(1) = SIG0
X = 0.00318
R = 0.64
EPTT(1) = 0
EPSV(1) = 0
EPS1(1) = 0
EPS3(1) = 0
PSI(1) = 0
DIL(1) = 0
D(1) = 0
A1(1) = 0
WRITE(8,*) 'SIGMAC' 
DO 10 I = 2, 200
DEPTT = X/R
DGAMA = 1.732 * DEPTT
EPTT(I) = EPTT(I-1) + DEPTT
GAMA(I) = GAMA(I-1) + DGAMA
H(I) = C * GAMA(I) * (GAMA(I) - A) / ((GAMA(I) + B)**2)
IF (H(I) .LE. SPHICV) THEN
  MU = MU1
ELSE
  MU = MU2
ENDIF
C Dilation Angles
DIL(I) = ((SPHICV - H(I)) / MU)
RAT1(I) = 1 / (1 + GAMA(I) / DGAMA)
RAT2(I) = GAMA(I) / DGAMA
C The β Values
PSI(I) = (DIL(I) + RAT2(I) * PSI(I-1)) * RAT1(I)
XX1 = 1 - PSI(I)
XX2 = H(I) / (1 + H(I))
XX3 = (1 - EPTT(I)) / (EPTT(I) * (1 + EPTT(I)))
C Incremental Pressures
DSIG(I) = 1.33 * XX1 * XX2 * XX3 * DEPTT * SIGC(I-1) + (C * COS(SPHI) * XX1 * XX3)
SIGC(I) = SIGC(I-1) + DSIG(I)
10 CONTINUE
DO 20 K = 1, 200
  WRITE(8,*) 'EPTT', 'DSIG', 'SIGC'
  DO 10 I = 1, K
    WRITE(8,*) EPTT(I), DSIG(I), SIGC(I)
  10 CONTINUE
  WRITE(8,*) 'No. ', 'H(gamma)', 'Sphicv', 'Dil Angle'
  DO 30 L = 1, K
    WRITE(8,*) L, H(L), SPHICV, DIL(L)
  30 STOP
END
Figure D.1: Flowchart for Cavity Expansion Model (Procedure 2)
The ratios of tip resistance to limiting pressures are plotted in the following figures.

![Figure D.2: Ratios (q_e / P_L) Versus Effective Vertical Stresses (C.C. 0 %)](image-url)
Figure D.3: Ratios ($\frac{q_c}{p_L}$) Versus Effective Vertical Stresses (C.C. 1 %)
Figure D.4: Ratios \( \frac{q_c}{p_e} \) Versus Effective Vertical Stresses (C.C. 2%)
The figures used in Approach 1 of empirical methods are presented in this section.

Figure D.5: Approach 1 in Empirical Method ($q_f = 10 \text{ kPa}$)
Figure D.6: Approach 1 in Empirical Method ($q_f = 30 \text{ kPa}$)
Figure D.7: Approach 1 in Empirical Method ($q_f = 50$ kPa)
Vita

Anand Jagadeesh Puppala was born in Rajahmundry, India on May 8, 1964. He received his B.S degree in Civil Engineering from Andhra University, Vizag, India in May, 1985. Then, he received his M.S. degree in Civil Engineering from Indian Institute of Technology (IIT), Madras, India in February, 1987. Subsequent to receiving his MS degree, he joined IIT as a research assistant. In January, 1988, he was admitted to the Department of Civil Engineering at Louisiana State University as a graduate research assistant and a doctoral candidate. Mr. Puppala has successfully completed all requirements for the Degree of Doctor of Philosophy in Civil Engineering on November 12, 1992 and he will receive the Degree in Spring, 1993 Commencement.
DOCTORAL EXAMINATION AND DISSERTATION REPORT

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Major Field: Civil Engineering

Title of Dissertation: Effect of Cementation on Cone Resistance in Sands: A Calibration Chamber Study

Approved:

Major Professor and Chairman

Dean of the Graduate School

EXAMINING COMMITTEE:

Date of Examination:

Nov. 12, 1992