EVALUATION OF LIQUEFACTION POTENTIAL OF SILTY SAND BASED ON CONE PENETRATION TEST

by

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Liquefaction is a phenomenon where a saturated soil can temporarily lose its shear strength during an earthquake as a result of the development of excess pore pressures. For the past 25 years since liquefaction phenomenon was first explained, it was thought to be mainly a problem with clean sand, and most of the research has focused on these soils. However, as case history information has come to light, it has become apparent that silty sands are commonly involved, and in some cases even silts. This has generated a need for knowledge about the response of silty sands and silts under seismic loading. Related to this issue is the question of how best to determine the liquefaction resistance of these soils in a practical setting.

This research has the objectives of providing an understanding of the behavior of saturated silty sands under seismic loading, and developing a rational basis for the use of the Cone Penetration Test (CPT) to predict liquefaction resistance in these materials. The study is primarily experimental, relying on laboratory and field testing and the use of a unique, large scale calibration chamber. The calibration chamber allows the field environment to be duplicated in the laboratory where conditions can be closely controlled and accurately defined.

One of the first problems to be overcome in the research was to determine how to prepare specimens of silty sands that would reasonably duplicate field conditions in both the small
scale of the conventional laboratory tests, and the large scale of the calibration chamber. Out of four different methods explored, consolidation from a slurry proved to be best. Two silty sands were located which had the desired characteristics for the study. Field work, involving both the Standard Penetration Test (SPT) and CPT was done as part of this investigation. The behavior of the silty sands were determined in the laboratory from monotonic and cyclic loading tests.

The test results show that the effect of fines is to reduce the cone penetration resistance, but not to affect the liquefaction resistance. The steady state shear strength of the soils seems to be correlated to the cone tip resistance, however, this correlation shows a higher steady state shear strength than those back figured from case history data. The results were also used to define state parameters for both of the soils tested. The state parameter was found to be a reliable index to the liquefaction potential and further study in this area is recommended.
Acknowledgements

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The author owed the greatest debt to his family. Through it all, they have been at the author’s side with all their love, patience and continuous support. For them, this work is dedicated to my wife, , and my children, , and .
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INTRODUCTION

During the past twenty years, research in geotechnical engineering has amply demonstrated that seismic shaking can induce excess pore pressures in saturated sands. This pore pressure development leads to a reduction of the stiffness and strength of the sand, and in extreme, it can cause the soil to liquefy. Liquefaction is a condition where the soil can flow in the presence of shear stresses such as those induced by a slope, a building foundation, or an embankment. Where the subsurface conditions are optimal for liquefaction, this phenomenon can account for a significant percentage of the life and property loss that occurs in an earthquake.

To this date (1989), most investigations into liquefaction have focussed on clean sandy soils. In part, this was due to a natural tendency to work with the simplest materials to avoid complications in testing. This trend also reflected the fact that few field studies were available to document the kinds of soils involved in liquefaction events. With time, evidence has grown that liquefaction is often associated with silty sands, and in some cases silts (Andresen and Bjerrum, 1967; Dobry et al., 1967; Lee et al., 1975; Youd and Bennett, 1983; Zhou, 1981; Ishihara et al., 1984; Hsing and Seed, 1988). As a result more interest has developed in the response of saturated silty sand during seismic loading (Ishihara et al., 1978; Chang et al., 1982; Dezfulian, 1984; and Kuerbis et al., 1988). In the 1985 National Research Council workshop
on liquefaction, one of the priority research needs was identified as understanding the behavior of silty sands and silts (National Research Council, 1985).

In addition to the change in attitudes about silty soils, the past decade has seen a shift in views about the methods of testing that should be used to identify liquefaction resistance of soils. The general conclusion has been that while laboratory tests are useful in research studies, it is not possible to reproduce the vagaries of soil structure and stress history in the laboratory that exist in the field. Thus, there has been a move towards field testing as the preferred method to evaluate the resistance of soil to seismic loading. In this process, the early emphasis was placed on use of the Standard Penetration Test (SPT) to provide the data base to characterize the soil (Seed, 1979; Tokimatsu and Yoshimi, 1983). The SPT is a simple procedure, and is commonly done as a matter of course in most geotechnical investigations. Also, because the SPT has been around for a long time, data were often available for sites where behavior was documented under earthquake shaking (Ishihara, 1977; Seed and Idriss, 1981; Seed et al., 1983). Recently, it has been recognized that the cone penetration test (CPT) offers a number of advantages over the SPT for soil characterization and for help in quantifying liquefaction resistance (Zhou, 1980, 1981; Robertson and Campanella, 1985; Ishihara, 1985; Seed and de Alba, 1986; Shibata and Terapaksa, 1987). However, the data base supporting the CPT is limited, and few, if any, formal calibration studies have been conducted in silty sands.

The purpose of this investigation is to improve our knowledge about the undrained behavior of silty sands under cyclic loading, and to help formulate a technology for use of the cone penetrometer in quantifying the liquefaction resistance of silty sands. The basis of the study is experimental, relying on laboratory and field testing, and full-scale cone penetration tests in a unique large calibration chamber.

The first cone penetrometers were developed in the Netherlands to determine the soil parameters needed to define the resistance of piles in clays. Since then, this device has been
extended to a wide range of applications. A modern cone operates electrically (Figure 1.1), and records the resistance at the tip, and friction acting on the sleeve of the cone. According to ASTM standards, the cone is 35.7 mm in diameter, with a cone tip angle of 60°, a projected tip area of 10 cm², and a friction sleeve surface area of 150 cm². Recently smaller and larger cones than the standard have been proposed and are used in practice.

The use of the cone penetrometer in liquefaction studies has taken a number of different paths, most of which are based on empirical correlations related to past site performance in earthquakes (Zhou, 1980, 1981; Shibata and Terapaksa, 1987, 1988). In these methods, the cone information is often supplemented by SPT data converted to equivalent cone tip resistances using empirical factors. Such data are useful, but ultimately are questionable because of the scatter in the conversion relations. A recent development is the introduction of concept using a soil resistance known as the “undrained steady state shear strength,” and a “state parameter”. The undrained steady state shear strength is the resistance of the soil when it reaches what is known as the critical state, following failure and application of large strains (Poulos et al., 1985). The state parameter defines the degree by which the initial conditions of the soil deviates from the critical state, and it has been correlated with cone tip resistance (Been et al., 1987a; Jefferies, 1988). In this research, the state parameter is used to characterize the behavior of silty sands under seismic loading.

In addressing the issues related to the use of the cone penetrometer to identify liquefaction resistance, the objectives were to:

1.) determine a reliable and repeatable method to produce samples of silty sand for both small samples in laboratory tests and the large samples used in the Virginia Tech large scale calibration chamber that are as representative as possible of field conditions.

2.) perform static and cyclic laboratory tests to define parameters and behavior of the silty sands that relate to development of excess pore pressure in an earthquake.

3.) conduct cone penetration tests in silty sands in both the field and in the large scale calibration chamber to provide well documented data base for cone resistance in these conditions.
materials.

4.) Using the data from this investigation, improve the methods for predicting liquefaction resistance of silty sands both from a conceptual and practical standpoint.

Two silty sands were used for the test program, the first of which was obtained from the location of the old Pepper’s Ferry on the New River near Blacksburg. A field investigation using CPT and SPT procedures was performed at this site. The second soil came from the excavation for Yatesville Dam in Kentucky, and was provided by the U.S. Corps of Engineers. This material was used in the calibration chamber tests. In the course of this research, four procedures were examined to assess the sample formation issue. This work involved both small scale tests in the laboratory and full scale tests in the calibration chamber. To define the pore pressure and strength response of the silty sands, a total of 15 monotonic triaxial tests and 42 cyclic triaxial tests were conducted.

The calibration chamber tests involved excavating and replacing 6000 kg of the Yatesville soil for each test. To prepare the Yatesville sand for testing, it had to be processed to eliminate oversize particles and detritus. After processing, the soil was placed in the chamber in a slurry form, and consolidated for two to three weeks under stresses similar to those in the field conditions and also used in the small scale laboratory tests. A total of five calibration chamber specimens were created and 23 CPT’s were performed.

The results of the investigation are provided in the following chapters. Chapter 2 gives a background review of previous work on liquefaction evaluation and cone penetration testing related to this study. At the end of this chapter the justifications for this work are presented. The scope and general methods used in the investigation are given in Chapter 3, along with a description of the two field site and testing programs at the sites. Chapter 4 covers the studies performed concerning sample fabrication techniques. This effort turned out to be more difficult than originally thought in that well developed procedures for clean sands did not work for silty sands. It is believed that the results have implications important for a variety
CONNECTION ROD

ELECTRIC CABLE

CABLE CHANNEL

STRAIN GAGES FOR FRICTION SLEEVE

INCLINOMETER

STRAIN GAGES FOR CONE

"O" RINGS

CONNE

35.6 MM (1.4 INCH)

FIG 1.1. ELECTRICAL CONE PENETROMETER
of laboratory studies using silty sands. The findings obtained in the laboratory triaxial tests are presented in Chapter 5. Basic data for defining the state parameter for the two test sands is given, and compared to those for similar soils reported in the literature. Chapter 6 presents the results of the cone penetration testing in the calibration chamber. These data are unique in that the cone results can be interpreted in terms of well defined soil densities and stress conditions. The test results in the calibration chamber will be compared to the penetration characteristic in the field where the soil was derived. The field testing effort at the Pepper's Ferry site is covered in chapter 7, and the results are related to those obtained on the Peppers' Ferry soil in the other types of tests. Chapter 8 links the findings of the entire experimental program, and presents a new procedure for evaluating the liquefaction resistance of silty sands using a cone penetrometer. Finally Chapter 9 gives the summary and conclusions.
2.1. INTRODUCTION

Following the Niigata and Alaskan earthquakes in 1964, geotechnical engineers were motivated to investigate earthquake induced liquefaction and its cause. Since then, many field documented cases where liquefaction has and has not occurred have been used to construct correlations with earthquake activity. Parameters of importance were shown to include intensity and duration of the earthquake, distance from earthquake source, ground acceleration, depth of water table, overburden pressure, soil density, and soil type.

The purpose of this chapter is to review some of the procedures commonly used for evaluating the liquefaction potential of soils particularly those containing a significant amount of fines. Because of the different procedures involved in the methods, the corresponding proposed mechanisms will be briefly discussed. To afford better understanding of the different views, a case study is presented in section 2.3.7.
2.2. LIQUEFACTION MECHANISM AND THE CORRESPONDING EVALUATION METHODS

2.2.1. Concept of Pore Pressure Generation

The liquefaction of soils is considered to be a result of excess pore pressures generated during earthquake shaking. This occurs because when a loose saturated sand is subjected to vibrations, it tends to decrease in volume. If drainage is unable to occur, the tendency to decrease in volume results in an increase in pore water pressure. If the pore pressure develops to the point at which it is equal to the overburden pressure, the effective stresses become zero, the sand loses its strength completely, at which point it is considered to be liquified (Seed and Idriss, 1982).

2.2.1.1. Method based on Cyclic Stress Ratio

In the field, the increase of pore water pressure is generally understood as a result of the application of cyclic shear stresses induced by the ground motions. These stresses are due primarily to the upward propagation of shear waves in soil deposit. Thus, the soil elements can be considered to undergo a series of cyclic stress conditions. This stress series is somewhat random in pattern but nevertheless cyclic in nature. Idealisation of such a cyclic load in the field is shown in Fig. 2.1.(a). Due to the applied cyclic stresses, the structure of the cohesionless soil tends to become more compact, and as the occurrence is so rapid, time does not allow the compression, thus resulting in transfer of stress to the pore water and a reduction in effective stress between the soil grains.

Cyclic triaxial and simple shear tests were used by Seed and his colleagues to simulate the field behavior of soils subjected to cyclic loadings (Lee, 1965; Seed and Peacock, 1971). Large
IDEALIZED FIELD LOADING CONDITIONS

\[ \sigma_0' \]

\[ K_0 \sigma_0' \]

Initial stresses

\[ \sigma_0'' \]

\[ K_0 \sigma_0'' \]

Cyclic load sequence

CYCLIC SHEAR STRESS, \( \tau_c \)

ZONE OF LIQUEFACTION

CYCLIC STRESS CAUSING INITIAL LIQUEFACTION OR A GIVEN AMOUNT OF CYCLIC SHEAR STRAIN IN \( n \) CYCLES (FROM TESTING PROGRAM)

AVERAGE CYCLIC STRESS DEVELOPED FOR \( n \) CYCLES BY EARTHQUAKE MOTIONS

Fig 2.1.(a). IDEALIZATION OF THE EARTHQUAKE LOAD ON SOIL ELEMENT IN THE GROUND.
(b). COMPARISON OF CYCLIC SHEAR STRESS DEVELOPED BY EARTHQUAKE AND THOSE DETERMINED IN THE LABORATORY. (Seed, 1979)
deformations of soil samples were observed to occur after the generation of excess pore pressure \( \Delta u \) reduced the effective confining pressure \( \sigma_s' \) of the soil to low value. If the generation of excess pore pressure was sufficient to equal the effective confining stress, then the soil was said to have reached the condition of *initial liquefaction* (Seed, 1979b).

Seed found that a convenient parameter for expressing the cyclic liquefaction characteristics of sand under level ground conditions is the ratio of the average cyclic shear stresses, \( \tau \), developed from cyclic loading to the initial vertical effective stress, \( \sigma_s' \). This ratio was termed the cyclic stress ratio, \( \tau/\sigma_s' \), and it has the advantage of taking into account the depth of the soil layer involved, the depth of the water table, and the intensity of earthquake shaking. The liquefaction resistance of the soil is therefore determined as the ability of the soil to resist cyclic shear stresses. The basic principle used in applying this concept in the early approaches involved comparison of the normalized cyclic shear stress causing initial liquefaction as determined from laboratory tests, \( \tau/\sigma_s' \), to the normalized cyclic shear stress expected to be developed in the field by earthquake, \( \tau_d/\sigma_s' \). Any zone where \( \tau_d/\sigma_s' > \tau/\sigma_s' \) was designated as *zone of liquefaction* (Fig. 2.1.b).

Valera and Donovan (1977) stated that the methods of evaluation of liquefaction potential can be classified into three categories: (1) Methods where both the cyclic shear stresses induced within a soil deposit and the significant number of stress cycles and their distribution with time are computed using simplified procedures; (2) Methods where the cyclic stresses induced within a soil deposit are computed by means of ground response analysis and (3) Empirical methods based on field performance data.

Methods (1) and (2) require laboratory test data on the cyclic stresses required to develop liquefaction or significant cyclic strains on representative samples of the in situ soils. These data are usually in the form of the cyclic stress ratio versus the number of stress cycles required to develop liquefaction or significant cyclic strains. An example of such data is shown in Fig. 2.2.(a). A comparison of the cyclic stresses induced in the field with the cyclic stresses
required to develop liquefaction or significant cyclic strains in the laboratory permits an eval-
uation of the factor of safety against liquefaction.

A classic example of the method 1 was set out by Seed and Idriss (1971). It used laboratory
data from tests conducted using uniform cyclic loads to simulate the soil behavior under the
irregular cyclic loading of an earthquake. The test data were presumed to define the resist-
ance of the soil to liquefaction by virtue of its response to a given number of load cycles. The
driving shear stress to cause liquefaction involved the computation of the equivalent uniform
cyclic shear stress, $\tau_{av}$, induced at any point in a soil deposit using the relationship:

$$\tau_{av} = 0.65 \gamma h \frac{a_{\text{max}}}{g} r_d \quad [\text{eq. 2.1.}]$$

in which $a_{\text{max}} = \text{maximum horizontal ground surface acceleration}$, $\gamma = \text{total unit weight}$, $h = \text{depth below ground surface}$; and $r_d = \text{depth reduction factor}$. The 0.65 factor assumes that
the equivalent uniform shear stresses, $\tau_{av}$, is 65% of the absolute maximum earthquake in-
duced shear stress. The depth reduction factor, $r_d$, recognizes that the soil is deformable and
does not behave as a rigid body. A range of typical values of $r_d$ is from 1.0 at the ground sur-
face to 0.5 at about 100 ft below ground surface.

An example of the method 2 approach uses a ground response analysis to get $\tau_{av}$. This is often
done using one dimensional wave propagation theory (e.g. SHAKE, A Computer Program for
Earthquake Response Analysis of Horizontally Layered Sites, Schnabel et al., 1972) as the
means to estimate the induced dynamic shear stresses together with their distribution and
variation with time (Fig. 2.2.b). The basis for this type of procedure was first introduced by
Seed and Idriss (1967). Typically as first presented, this approach also relied upon laboratory
tests to define the response of the soil to cyclic loading.

Method (3) deviates from the other two method in using some in situ soil characteristic as a
means to define the response of the soil to cyclic loading by comparing the liquefaction po-
Fig. 2.2. (a). TYPICAL CYCLIC TRIAXIAL TEST RESULTS
(b). TYPICAL VARIATION OF SHEAR STRESS FROM GROUND RESPONSE ANALYSIS
tential of a proposed site with that of other sites where liquefaction is known to have occurred in previous earthquakes. In this method, the soil is commonly characterized by use of Standard Penetration Test (ASTM D-1586-67, 1974) or Cone Penetration Test (ASTM D-3411, 1979). Because of problems associated with obtaining undisturbed samples for use with methods 1 and 2, methods 3 has become the most commonly used approach for liquefaction evaluation. More detail on this approach will be given later in this chapter.

2.2.1.2. Method based on Cyclic Shear Strain

The approach based on cyclic shear strain assumes that pore water pressure generation during cyclic loading is controlled by the magnitude of the cyclic shear strain. This premise leads to the conclusion that shear modulus, G, is the main parameter controlling pore water pressure build up in the ground. An important practical consequence is that measurements of in situ modulus at small strains can be used for predicting pore pressure during cyclic events.

Dobry et al. (1982) suggested an equation to estimate the cyclic strains, \( \gamma_c \) induced by the earthquake.

\[
\gamma_c = 0.65 \frac{a_p \sigma_0 g}{G_{\text{max}} G (G/G_{\text{max}})} \quad [\text{eq.2.2.}]
\]

where \( a_p \) is the peak horizontal acceleration at the ground surface, \( g \) is acceleration of gravity, \( \sigma_0 \) is the total overburden pressure and \( r_d \) is the stress reduction factor as a function of depth which has been discussed previously. \( G_{\text{max}} \) is shear modulus of the soil at a very small shear strains \((\gamma_c \approx 10^{-4}\%)\) that can be estimated from shear wave velocity measurements using cross hole geophysical techniques (e.g. Stokoe and Hoar, 1978) or obtained from published correlations of \( G_{\text{max}} \) or \( V_s \) with Cone Penetration Tests (e.g. Robertson and Campanella, 1983) and with Standard Penetration Tests (e.g. Ohsaki and Iwasaki, 1973; Seed et al., 1986). \( (G/G_{\text{max}}) \) is
a modulus reduction factor of the soil depending on the cyclic shear strain, and for this particular expression, corresponding to the value of the induced cyclic strain, $\gamma_c$.

Dobry et al. (1982) have suggested the existence of threshold shear strain, $\gamma_t$. For shear strains less than this, volume change tendencies and pore pressure generation is negligible. As a consequence, if the calculated shear strain induced by earthquake, $\gamma_c$, is less than $\gamma_t$, neither pore pressure build up, nor liquefaction will occur. On the other hand, if $\gamma_c > \gamma_t$, the values of $\gamma_c$ should be used to estimate the magnitude of excess pore pressure at the end of the earthquake.

For saturated sand, one can estimate the increase of pore pressure in the deposit using the chart as presented by Dobry (1985) in Fig. 2.3. This value of $\Delta u/\sigma'_s$ estimated in this step is used to decide if the site will experience initial liquefaction ( $\Delta u/\sigma'_s = 1.0$ ). The recompression characteristics can be estimated from published data on recompression of the particular type of soil or it can be obtained from one dimensional or triaxial compression tests on undisturbed samples. Alternatively, a rough estimate may be obtained using data in Fig.2.4. (Castro, 1987).

2.2.2. Concept based on Undrained Steady State Shear Strength

The initial development of the steady state analysis method was motivated by the static liquefaction failure of Ft. Peck dam. The liquefaction failure took place under conditions of large statically induced strain (Casagrande, 1936, 1950). Early workers studying the mechanism of flow slides in fine sands recognised the importance of the in situ void ratio (Casagrande, 1938; Andresen and Bjerrum, 1967). They postulated that liquefaction and the associated large deformations could only occur above a certain (critical) void ratio. A for-
I.2

§ 1.0 _ _ _ ·

"Strain—Controlled Cyclic Triaxial Tests\n\n$n = 10$ cycles\n$\gamma_c = 1.5 \varepsilon_c$

![Graph of Pore Pressure Ratio vs. Shear Strain](image)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Sand</th>
<th>$\sigma^{*70}$ (psf)</th>
<th>$\text{Dr}$ (%)</th>
<th>Samples/Fabric</th>
<th>Measured $u$ Peak (P) or Residual (R)</th>
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<td>P</td>
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Fig. 2.3. PORE PRESSURE RATIO AS A FUNCTION OF CYCLIC SHEAR STRAIN FOR 7 DIFFERENT SANDS FROM RESULTS OF STRAIN CONTROLLED CYCLIC TRIAXIAL TESTS WITH DIFFERENT SPECIMEN PREPARATION TECHNIQUES, DENSITIES AND CONFINING PRESSURES (Dobry, 1985).
Fig. 2.4. VOLUME CHANGES OF SANDS AND SILTS DUE TO RECONSOLIDATION AFTER CYCLIC LOADING (Castro, 1987).
malized design procedure were then developed from this concept by Casagrande and his colleagues (Castro, 1969, 1975; Castro et al., 1982; Poulos, 1981; Casagrande, 1975).

Casagrande proposed that soils develop a *flow structure* at liquefaction where the relative positions of the grains are constantly changing in a manner that maintain a minimum resistance. This behavior was demonstrated in the laboratory by Castro (1969), and was termed steady state condition. Based on this concept, Poulos et al. (1985) proposed the steady state analysis method for the evaluation of liquefaction potential.

This method requires the presence of driving shear stresses in the soil, \( \tau_d \), from static loading existing prior to the earthquake. The term "driving" refers to those shear stresses that are required to maintain static equilibrium and therefore are available to drive the mass should the soil lose sufficient strength. Driving shear stresses correspond to those that one would calculate in a stability analysis. Driving shear stress exists, for example, in the soil supporting a heavy building, within an earth embankment and its foundation and beneath natural slopes. The concept based on steady state shear strength suggests that the soil can be liquefied under conditions of either static or dynamic loading. Liquefaction failure occurs at the time when the undrained shear strength of the soil is exceeded by the driving stress present in the soil mass during undrained loading. The development of excess pore pressure and cyclic shear strain in a soil due to earthquake shaking tends to push the soil into the steady state condition.

When loose saturated cohesionless materials are subjected to shear loading, the pore water pressure increases, the effective stresses in the soil become very small, and the strength of the soil is reduced. If the shear stresses in the mass are larger than the reduced shear strength of the soil, the mass will flow like a viscous liquid until equilibrium is reached and the soil mass is stable. Casagrande (1936) termed this behavior *spontaneous* liquefaction.

It is generally understood that the steady-state concept is a derivation of the Casagrande’s critical void ratio concept. Casagrande introduced the idea of a critical void ratio at which a
cohesionless soil can undergo any amount of deformation without volume change. During shear deformation in a drained condition, a loose sand would tend to compress until it reached the critical void ratio. In an undrained condition, the tendency for volume reduction would result in increased pore water pressure and reduced effective stress.

In 1975, Casagrande proposed that during undrained loading, the effective stress in loose sand is reduced until it reaches the steady state line as illustrated in Fig. 2.5. The soil is initially at an effective minor principal stress, $\sigma_{3e}'$, prior to liquefaction. At liquefaction, with no volume change, the increased pore pressure results in a reduced effective stress, $\sigma_{y}'$ but no change in void ratio. The critical void ratio and the effective stress at flow with constant volume and velocity are uniquely defined by the steady state line. Thus, it was proposed that liquefaction and flow are related to large reductions in effective stress, with the final value of effective stress depending on the in situ void ratio.

The Steady State Line (SSL) is defined as the locus of all points in void ratio - stress space at which a soil mass deforms under condition of constant effective stress, void ratio and velocity. When load is applied to soil, after sufficient straining, the soil will reach a point on the steady state line. This line is supposedly unique for a given soil and independent of the initial fabric, the loading path, or the strain rate (Poulos, 1981; Castro et al., 1982; and Poulos et al., 1985). The steady state of deformation was defined by Poulos (1981) as a condition in which a soil mass is continuously deforming at constant volume, constant effective stress, constant shear stress and constant velocity. The steady state of deformation is achieved only after all particle orientation has reached a steady state condition and after all particle breakage, if any, is complete so that the shear stress needed to continue deformation and the velocity of deformation remains constant.

According to the steady state concepts, the undrained steady state strength of a soil, $S_{us}$, is only a function of its void ratio, and thus the same value of $S_{us}$ applies whether the soil is loaded monotonically or cyclically. Test data supporting this have been presented by Castro.
CYCLIC MOBILITY (momentarily, zero $\sigma_3$ at zero shear stress results in large strains in laboratory tests)

EFFECTIVE MINOR PRINCIPAL STRESS

Fig. 2.5. UNDRAINED BEHAVIOR OF SATURATED COHESIONLESS SOIL
(Castro, 1977)
(1969, 1975) and Castro et al. (1982). Castro (1987) compared the stress strain behavior of soil under cyclic loading to the behavior under monotonic loading (Fig. 2.6.) for two cases. In the first case, the undrained steady state strength, $S_{us}$, is lower and in the other, $S_{us}$ is higher than the driving shear stress $\tau_d$. The stress strain behavior under cyclic loading for case (1) is very similar to that under monotonic loading, i.e., the peak strength is overcome by sufficient straining which is either caused by a single or by repeated loading. The strain at which the resistance of the soil starts decreasing towards $S_{us}$ is about the same in both cases and they end up at the same value of $S_{us}$ since the void ratio is the same. In case (2) cyclic loading causes an accumulation of strain often accompanied by an increase in pore pressure. The shape of the stress strain curve following cyclic loading may be different from the monotonic loading case, however they both reach the same value of $S_{us}$.

There are substantial differences in the field behavior that correspond to cases (1) and (2) and also in the method of analysis to predict the field behavior. In case (1), the consequence of cyclic loading is a massive failure such as a flow slide. In case (2) cyclic loading induces a limited deformation without changing the stable configuration of the soil mass.

Evaluation of liquefaction potential using the steady state concept involves the following steps:

1.) Determination of the steady state line from laboratory consolidated undrained triaxial tests conducted on compacted samples.

2.) Careful field fixed piston sampling for determination of in situ void ratio.

3.) Determination of the undrained steady state shear strength of the undisturbed sample using consolidated undrained triaxial tests.

4.) Construction of a field steady state line based on the undrained steady state shear strength of the undisturbed sample and the assumption that the field line is parallel to the line determined for the compacted samples.

5.) Determination of the in situ steady state shear strength corresponding to the in situ void ratio.

6.) Calculation of a factor of safety against liquefaction based on the in situ steady state shear strength.
Fig 2.6. STRESS STRAIN BEHAVIOR OF SATURATED SAND UNDER MONOTONIC AND CYCLIC LOADING (Castro, 1987).
strength and the in situ driving stresses.

A more rigorous treatment of the used of the steady state approach is given by Poulos et al. (1985). An example of the use of this method for predicting the steady state shear strength in situ is shown in Fig. 2.7.

2.2.3. State Parameter Approach

According to some researchers (Lee, 1965; Lade, 1972; Been et al., 1985), relative density or void ratio alone is not sufficient to characterize the behavior of cohesionless material. Been and Jefferies (1985) suggested that stress level must also be incorporated in an analysis. In a program to study the behavior of sand containing fines they found that under different combination of void ratio and mean effective stress, sands and silty sands behave similarly if test conditions assure an equal proximity to the steady state. The proximity to the steady state is identified as the state parameter which is defined as the void ratio difference between the initial sand state and the steady state conditions at the same mean effective stress, $\sigma_m'$ (Fig. 2.8.). The choice of $\sigma_m'$ is based on the assumption that the deviatoric component of stress will be reflected directly in the sand fabric parameter. Been and Jefferies (1985) used the symbol $\psi$ to represent the state parameter.

Been and Jefferies postulated that state parameter can be used to describe much of the behavior of granular materials over a wide range of stresses and densities. It was successfully applied to normalize large strain behaviors where the influence of initial fabric is small. The selection of steady state as reference line to the state parameter is based on the fact that at steady state conditions, sands and silty sands have a unique structure which is not influenced by the initial conditions. Some authors (Rowe, 1962; Schofield & Wroth, 1968) stated that sand has no structure at critical state (or steady state) while others (Poulos, 1981; Casagrande,
Fig. 2.7. STEADY STATE DIAGRAM SHOWING THE USE OF SSL FOR ESTIMATING THE STEADY STATE UNDRAINED SHEAR STRENGTH IN SITU (Castro, 1987)
Fig. 2.8. DEFINITION OF STATE PARAMETER
1975) stated that a "flow" structure exists. The argument proposed by Been and Jefferies however, does not depend on the nature of the sand structure at the steady state, but rather a unique and repeatable particle arrangement. This hypothesis is not supported by direct measurement of soil fabric in steady state condition.

Been and Jefferies (1985) have shown that state parameter can be correlated to the behavior of sand such as the drained angle of shearing resistance and the dilation rate (Fig. 2.9.). Stress strain behavior could also be characterised by state parameter independent of material type. This will be demonstrated in section 5.4.5. from results of tests performed in this study. Contractive material is separated from dilative material by the value of state parameter. Liquefaction potential can be evaluated based on the dependency of the soil characteristics on the values of the state parameter. It should be noted, however, that state parameter correlations were developed from a data base that only includes laboratory prepared sand samples.

2.3. LIQUEFACTION POTENTIAL EVALUATION WITH EMPHASIS ON SILTY SAND

2.3.1. Soil Conditions Susceptible to Liquefaction.

Field studies have shown that recent geologic deposits of sandy soils are the most prone to liquefaction. These studies have provided information concerning the soil types involved in liquefaction events, and have been instrumental in the development of design correlations to seismic behavior. On the other hand, laboratory studies have been conducted which allowed...
Fig. 2.9. RELATIONSHIPS OF
(a) DRAINED ANGLE OF SHEARING RESISTANCE WITH STATE PARAMETER
(b) DILATION RATE WITH STATE PARAMETER (Been, et al., 1987).
isolation of particular soil characteristics which control the generation of pore pressures during seismic events.

Based on previous work, it has been suggested by some researchers that saturated clean sands are more susceptible to cyclic pore pressure generation than silty or "dirty" sands. It has also been shown that greater the plasticity of the fines of a soil, the smaller the amount of pore pressure generation during seismic events. Other important factors related to liquefaction phenomena are the soil density, amplitude and direction of load, stress history of the deposit, the size, shape, fabric and gradation of the particles, confining pressure acting on the soil, and the age of the deposit.

The simplest criteria to use for identifying soil that is susceptible to liquefaction is by the grain size characteristics. Based on gradations of soils that did and did not liquefy during earthquakes, Tsuchida (1970) proposed the grain size distribution boundary curves as presented in Fig. 2.10, to identify susceptibility of the soil to liquefaction. The lower boundary on particle size reflects the influence of fines in decreasing the tendency of soils to densify during cyclic loading. Plastic fines make it more difficult for sand particles to come free of each other and seek denser arrangements. However nonplastic fines apparently do not have as much of this restraining effect. The upper boundaries are associated with the more permeable coarser soils and partial dissipation can occur even during earthquake.

Observations of data from various earthquakes by Tokimatsu and Yoshimi (1983) indicated that more than half of all the liquefied materials fell within a range of fines content less than five percent and that none of the soils containing more than 20% clays has suffered serious strength loss due to liquefaction. A study by Seed and Idriss (1981) agreed with this conclusion. Meanwhile, Finn (1982) suggested from a study in mainland China that plasticity index could be a promising parameter to estimate the liquefaction resistance of silty soil. From his study, it was recommended that a plasticity index of 10 seems to be the threshold for the possibility of liquefaction to be eliminated.
Fig. 2.10. GRAIN SIZES OF SOILS SUSCEPTIBLE TO LIQUEFACTION (Tsuchida, 1970).
In regard to in situ tests, Robertson et al. (1985) proposed a chart to identify the type of soil that could liquefy based on data obtained from the CPT. Soil that is susceptible to liquefaction falls within an area on the chart designated within the area of zone A (Fig. 2.11.). Loose, clean sand with a $D_{50} \geq 0.25\text{mm}$ tend to fall within the upper area of zone A with $q_c = 30 - 150\text{kg/cm}^2$ and friction ratio, $FR < 1.0$. Soil that falls within the lower area of zone A are the loose silty sands and silts since a decrease in mean grain size tends to cause a decrease in penetration resistance.

2.3.2. Laboratory studies

Case studies have revealed that both clean sand, silty sand and silt of low plasticity are susceptible to liquefaction (Dobry et al., 1967; Youd and Bennett, 1983; Lucia, 1981; and Ishihara, 1984). However, information concerning the quantifying behavior of the effect of fines on liquefaction resistance is scarce in the literature. Dezfulian (1984) suggested this was probably due to the difficulty in testing samples with the same relative density at different fines content.

2.3.2.1. The Effect of Fines on the Cyclic Strength of Soil

In early literature on liquefaction response, it was suggested that increasing the amount of fines would increase the cyclic shear strength of cohesionless soils (Seed and Iddriess, 1971). Later work has generally suggested that the main issue is the plasticity of the fines content, not the fines themselves. However, the evidence does not completely fall on one side or the other of the issue. Chang et al. (1982) made a systematic approach to evaluate the effect of fines on the cyclic strength of silty sands. In their study, coarse and medium Denver sand was selected for the investigation. Various amounts of silt were added to achieve a fines content of 10% to 60%, and the behavior of silty sand was compared with clean sand and pure...
Fig. 2.11. SOIL CLASSIFICATION CHART FOR ELECTRIC CONE SHOWING PROPOSED ZONE OF LIQUEFIABLE SOILS (Robertson and Campanella, 1985)
silt. Samples of sand and silty sand were prepared by the moist tamping technique and cyclic triaxial tests were performed. They found that specimens containing 10% to 30% fines are slightly less resistant to liquefaction than clean sands, however, samples with 60% fines contain was significantly more resistant than clean sands. They stated that samples with 60% fines had a similar cyclic strength as those of pure silts. It has to be noted that the pure silts was in a much denser state than the clean sand.

The findings of a number of laboratory studies of silty sands and sandy silts are compiled in Table 2.1. To simplify the observations, the cyclic strength of each material was represented by the values of the cyclic stress ratio at 15 cycles, corresponding to earthquake magnitude of 7.5 (Seed, 1983). The trend of the cyclic strengths of the soils represented in Table 2.1. were plotted in terms of fines content (Fig. 2.12) or the mean grain size (Fig. 2.13). Both figures suggest that there is no apparent increasing or decreasing trend of the cyclic strength of the material caused by fines content or the mean grain size $D_{50}$. Scatter in the plots is apparently the effects of varying densities or plasticity of the soil samples.

The conclusion from Figures 2.12. and 2.13. does not agree with the finding by Chang et al. (1982). One possible explanation can be derived from the finding by Ishihara et al. (1978) that overconsolidation increases the cyclic strength of cohesionless material. Notably, this effect is more pronounced as the fines content increases. Chang et al. (1982) prepared the samples using compaction, thus inducing an apparent overconsolidation effect by virtue of the soil structure and added lateral stresses. As the fines content increases, the overconsolidation induced by compaction would increase the cyclic strength of the soil (Ishihara, et al, 1978) and this would be greatest for finer grained soils in a similar manner to the outcome found by Chang et al. (1982). Thus, this explained why Chang et al. (1982) observed an increase in the cyclic strength of the samples with high percentage of fines.
<table>
<thead>
<tr>
<th>Boring or Location</th>
<th>Material</th>
<th>% fines</th>
<th>( D_w ) (mm)</th>
<th>PI of fines</th>
<th>Prepared</th>
<th>Range of Void Ratio</th>
<th>Remarks/Conclusion</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Koto A</td>
<td>sandy silt</td>
<td>58</td>
<td>0.061</td>
<td>20</td>
<td>slurry</td>
<td>0.83-0.85</td>
<td>OCR increases</td>
<td>Ishihara et al., 1978</td>
</tr>
<tr>
<td>Koto A</td>
<td>silt</td>
<td>100</td>
<td>0.023</td>
<td>20</td>
<td>slurry</td>
<td>0.89-1.02</td>
<td>cyclic strength</td>
<td></td>
</tr>
<tr>
<td>Koto B</td>
<td>silty sand</td>
<td>15</td>
<td>0.282</td>
<td>N/A</td>
<td>slurry</td>
<td>0.82-0.90</td>
<td>and the effect</td>
<td></td>
</tr>
<tr>
<td>Suzuki</td>
<td>silt</td>
<td>100</td>
<td>0.009</td>
<td>N/A</td>
<td>slurry</td>
<td>1.40-1.60</td>
<td>is more</td>
<td></td>
</tr>
<tr>
<td>Tksago</td>
<td>sand</td>
<td>0</td>
<td>0.180</td>
<td>N/A</td>
<td>slurry</td>
<td>0.82-0.91</td>
<td>pronounced for</td>
<td></td>
</tr>
<tr>
<td>TaitoA1</td>
<td>silty sand</td>
<td>29</td>
<td>0.182</td>
<td>N/A</td>
<td>slurry</td>
<td>0.54</td>
<td>soil with higher</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.52-0.62</td>
<td>lines content</td>
<td></td>
</tr>
<tr>
<td>TaitoA2</td>
<td>silty sand</td>
<td>20</td>
<td>0.351</td>
<td>N/A</td>
<td>slurry</td>
<td>0.46</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TaitoA3</td>
<td>silty sand</td>
<td>48</td>
<td>0.108</td>
<td>N/A</td>
<td>slurry</td>
<td>0.72</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.69-0.71</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Owls</td>
<td>silty sand</td>
<td>35</td>
<td>0.102</td>
<td>Low</td>
<td>undist.</td>
<td>0.75-0.85</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.255</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper-S Fernando</td>
<td>silty silt</td>
<td>20-68</td>
<td>N/A</td>
<td>N/A</td>
<td>undist.</td>
<td>Dr = 51-58%</td>
<td>No apparent distinction for results of fine silty sand and clean sand</td>
<td>Lee et al., 1975</td>
</tr>
<tr>
<td>Hyd Fill</td>
<td>sand</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>coarse sand</td>
<td>0-25</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower-S Fernando</td>
<td>silty sand</td>
<td>8-85</td>
<td>&lt;0.32</td>
<td>N/A</td>
<td>undist.</td>
<td>0.49-0.71</td>
<td>data from all samples follow one trend</td>
<td>Hsing &amp; Seed, 1988</td>
</tr>
<tr>
<td>Hyd Fill</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.256</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Fig. 2.12. CYCLIC STRENGTH OF SILTY SANDS VS. FINES CONTENT

NUMBER OF CYCLES = 15

FINES CONTENT (%)
Fig. 2.13. CYCLIC STRENGTH OF SILTY SANDS VS. MEAN GRAIN SIZE

NUMBER OF CYCLES = 15
In summary, the majority of laboratory test data show that the fines content by itself does not seem to increase the cyclic strength of the silty sand vis a vis a clean sand.

2.3.2.2. Effect of Fines on the Steady State Shear Strength

An investigation by Mohammad et al. (1985) on the behavior of sand containing 13%, 32% and 63% of fines under a series of monotonic and cyclic tests determined that (1) a unique steady state strength envelope was obtained for all three sands of different fines contents and (2) the pore pressure build up at low cyclic shear strain (< 0.05 %) is independent of fines content.

Currently, there is not much literature on this aspect of liquefaction flow failure behavior with respect to fines content. It is one of the goals of this research to study this phenomena.

2.3.3. Use of In situ Tests for Liquefaction Potential Assessment.

Variability in soil deposits, difficulties encountered during undisturbed sampling of sands below the ground water table, and to a lesser extent, problems related to laboratory equipment, have discouraged the use of laboratory determined dynamic soil properties. In these circumstances, methods correlating the observed performance of sand deposit during earthquakes to the SPT became an important tools for geotechnical engineers.

The SPT involves measuring penetration resistance of a split spoon sampler in number of blows per foot of penetration and retrieving a sample from a 46 cm (1.5 ft) interval. The penetration resistance measured during the SPT is the most common method for evaluating liquefaction susceptibility (Seed and Idriss, 1983; Iwasaki et al., 1981). The penetration resistance has been correlated to soil properties such as the drained friction angle and relative density. Also the disturbed samples retrieved from this test allow index property tests to be
conducted. SPT blow counts of cohesionless soils were found to correlate to their liquefaction resistance (Seed et al., 1979, 1983, 1986; Tokimatsu et al., 1981, 1983). Correlation charts based on N-SPT and cyclic stress ratio were used to distinguish liquefiable conditions from non-liquefiable conditions for soils with different fines content (Seed, 1984). The charts were constructed from case history studies.

The focus on the SPT was natural because of large data base from this test. However, several factors induce variability in this method including the different energy ratios supplied by the hammer, the nature of fluid in the drilling holes, the diameter of the drill hole, the type of bit used in the drilling operation, the configuration of the sampling spoon and the frequency of delivery of the hammer blows (Seed and de Alba, 1986).

Owing to the problems with the SPT, and the fact that the CPT is more economic, provides a continuous profile of data and can be easily automated, many professionals have expressed increased interest in using the CPT. The CPT measures tip resistance ($q_t$) and sleeve friction ($f_s$) as the penetrometer is pushed into the ground at a rate of 2 cm/sec (ASTM D1586-67). The tip resistance can be correlated to the strength whereas combination of the sleeve friction ratio and tip resistance can be used to indicate the soil type. Tip and friction sleeve resistance are continuously recorded during penetration.

In the last decade, researchers and practising engineers have focused their attention on the possibility of correlating the cyclic resistance of sand deposits with the cone penetration test (CPT) results (Zhou, 1981; Robertson and Campanella, 1985; Jamiolkowski et al., 1985; Seed et al, 1986; and Shibata et al., 1987, 1988).
2.3.4. Liquefaction Potential Evaluation based on SPT.

The concept of a critical N-value associated with liquefaction was first proposed by Koizumi (1966) and Kishida (1966) based on their work in the liquefaction areas associated with 1964 Niigata earthquake. An important fact in their work was that they obtained access to blowcount data measured before the earthquake when the soil was undisturbed by the corresponding induced liquefaction.

2.3.4.1. Method proposed by Seed

Seed (1971, 1979) and his co-workers developed the first general correlations between cyclic stress ratio and the SPT blowcount. The approach was similar to the procedure Seed used with laboratory undrained cyclic test results. The only difference was that the cyclic shear resistance of the soil was defined from a liquefaction chart which based on the field performance data and the blowcount. The shear stress from the earthquake was determined using eq. 2.1. For convenience, the shear stress was normalized by the effective overburden stress at the depth where liquefaction potential was to be evaluated:

\[
\frac{\tau_{av}}{\sigma'_o} = 0.65 \frac{a_{\text{max}}}{g} \frac{\sigma_o}{\sigma'_o} r_d \quad \text{[eq.2.3.]} \]

In which \(\sigma'_o\) = effective overburden stress at the depth of interests.

In the early development of the correlation of the cyclic stress ratio developed in the field and the SPT blow count, it was assumed that all the factors affecting the cyclic loading characteristics of sands were similarly represented in the number of blowcount. Thus, the SPT-N values were regarded more representative as an index of liquefaction resistance. Since the measured N values in the field reflect the influence of the soil properties as well as the effective overburden pressure, Seed used a normalized penetration resistance, \(N_r\), to eliminate
the influence of confining pressure. $N_t$ was defined as the equivalent SPT blowcount under an effective overburden pressure of 1 ton per sq.ft. (96 kPa). Seed suggested to use a correction factor for effective overburden pressure $C_N$ based on data by Marcuson and Biegansky (1977), and the value of $N_t$ was then determined as

$$N_t = C_N \cdot N_{(measured)} \quad \text{[eq. 2.4.]}$$

To adjust for the energy variability when using different types of hammers, an SPT blowcount corresponding to an energy ratio of 60%, $(N_t)_{60}$ was recommended as a standard. The 60% energy ratio is typical of that obtained with a safety hammer, the most commonly used hammer type.

Based on data for silty sands ($D_{50} < 0.15 \text{mm}$) from Tokimatsu and Yoshimi (1981) in the Miyagiken-Oki earthquake, Seed (1982) found that for a given blowcount, silty sands are less likely to liquefy than clean sands. He proposed a new boundary on his blowcount correlation chart to account for this. In other terms, it was noted that the normalized standard penetration resistance, $N_t$, for sand with $D_{50} > 0.25 \text{mm}$ was essentially equal to that of silty sands ($D_{50} < 0.15 \text{mm}$) plus 7.5 for purposes of liquefaction potential evaluation. It was concluded that the boundary previously established for sands could be used for silty sands, provided the $N_t$ value for silty sand is increased by 7.5 before entering the chart. This correction can have a very significant effect on liquefaction evaluations for silty sand deposits. Later Seed et al. (1984) refined the curves to take into account sands containing fines more than 5% (Fig. 2.14.).

2.3.4.2. Method proposed by Tokimatsu and Yoshimi, 1983.

Using data from earthquakes in Japan, China and the USA, Tokimatsu and Yoshimi (1983) also examined the use of SPT in the liquefaction evaluation of sands containing fines. In their study, they divided the liquefaction behavior of these soils into four major groups: extensive liquefaction, moderate liquefaction, marginal liquefaction and no liquefaction. Extensive liquefaction is defined as when a sand layer experiences 2% strain during an earthquake or
Fig. 2.14 RELATIONSHIPS BETWEEN STRESS RATIO CAUSING LIQUEFACTION AND NORMALISED N-SPT FOR SILTY SANDS FOR MAGNITUDE 7.5 EARTHQUAKE (Seed et al., 1984)
when a heavy structure settles more than 20 cm. For conditions when liquefaction is evident, but the resulting strains are less than the extensive case, the term moderate liquefaction is used. A site where there is no evidence of settlement or sand boils is called no liquefaction, and a marginal condition is used to classify the boundary between moderate liquefaction and no liquefaction. The results of their observation are shown in Fig. 2.15.

Fig. 2.16. shows the relationship between both fines contents and mean grain size and SPT \( N_i \) values for liquefied soil. It shows a fairly well-defined trend in which the SPT \( N_i \) values for the liquefied soils decrease with either an increase in fines content or decrease in mean grain size. The mean \( N_i \) values for liquefied soils in terms of fines content are presented in Table 2.2. There is a significant reduction of SPT \( N_i \) value for silty sands with more than 10 % fines compared to clean sands, suggesting that the presence of fines reduces SPT \( N \) values without significant changes in liquefaction resistance. Based on their data, Tokimatsu and Yoshimi proposed that \( D_{50} = 0.2 \) mm could be used as an index to separate clean sands from silty sands.

The conclusions drawn from their study were: (1) Sand containing more than 10 percent fines have much greater liquefaction resistance than clean sands having the same SPT \( N \)-values. (2) Extensive damage due to liquefaction would not occur for clean sand with SPT- \( N_i \) values greater than 25 and silty sands containing more than 10 % fines whose SPT- \( N_i \) values greater than 20.

To evaluate soil liquefaction susceptibility, a correlation based on laboratory and field data was derived. Relationships obtained in the laboratory between stress ratio and relative density were approximated by the following expression:

\[
\frac{\sigma_d}{2\sigma_C'} = a \left[ \frac{D_r}{100} + \left( \frac{D_r}{C} \right)^n \right] \quad \text{[eq.2.6.]} 
\]
Fig. 2.15 Correlation of field cyclic stress ratio and SPT-N1 for (a) clean sands and (b) sands containing more than 10% fines (Tokimatsu and Yoshimi, 1983).
Fig. 2.16 RELATIONSHIP FOR NORMALISED SPT-N VALUE FOR LIQUEFIED SOILS
(a) BASED ON MEAN GRAIN-SIZE  (b) BASED ON FINES CONTENT
(Tokimatsu and Yoshimi, 1983).
Table 2.2. MEAN SPT-$N_v$ VALUES FOR LIQUEFIED SOIL

(Tokimatsu and Yoshimi, 1983)

<table>
<thead>
<tr>
<th>Fines Content (%)</th>
<th>$N_v$</th>
<th>$N_v$ with more than 5% gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5</td>
<td>12</td>
<td>16</td>
</tr>
<tr>
<td>10 - 20</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>20 - 60</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>&gt; 60</td>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>
in which $a$ and $n$ are empirical constants and $C$ is an empirical parameter depending on strain amplitude. It should be noted that according to ASTM D-4254 (1983), relative density is not applicable for sand having more than 15% of fines. However, a relationship between relative density and $N$-values for clean sands given by Meyerhof (1957) was used:

$$D_r = 21 \sqrt{\frac{N}{\sigma_0' + 0.7}} \quad [eq.2.7.]$$

in which $\sigma_0'$ denotes the effective overburden stress in $kgf/cm^2$. Normalising the SPT-$N$ values given by:

$$N_1 = C_N \cdot N = \left( \frac{1.7}{\sigma_0' + 0.7} \right) \cdot N \quad [eq.2.8.]$$

the $D_r$ vs. $N$ relationship was reduced to:

$$D_r = 16 \sqrt{N_1} \quad [eq.2.9]$$

Taking into account the fines content, Tokimatsu and Yoshimi proposed a correction factor $\Delta N_r$ and incorporated into the expression as:

$$N_a = N_1 + \Delta N_r \quad [eq.2.10.]$$

 Adopting a correction factor suggested by de Alba et.al. (1976) for shear stress ratio in the field, $C_r$, the previous expression becomes:

$$\left( \frac{\tau_f}{\sigma_0'} \right)_{field} = C_r \left( \frac{\sigma_d}{2\sigma_c'} \right)_{triaxial} \quad [eq.2.11.]$$

Finally, substituting equations 2.6, 2.9, and 2.10 into equation 2.11, the liquefaction resistance can be expressed as:

$$\frac{\tau_f}{\sigma_0'} = a C_r \left[ \frac{16 \sqrt{N_a}}{100} + \left( \frac{16 \sqrt{N_a}}{C_s} \right)^n \right] \quad [eq.2.12]$$

REVIEW OF PREVIOUS STUDIES
in which \( a = 0.45, C_r = 0.57, n = 14, \) and \( C_s = 80 - 90 \) for separating liquefaction and no liquefaction conditions and \( C_s = 75 \) for extensive liquefaction.

The proposed method of liquefaction evaluation is applied like that given by Seed (1981), i.e. by comparing the stress ratio caused by earthquake to the stress ratio to liquefy as determined from a field SPT correlation. The following conditions apply for use of this theory:

1. If clay content is greater than 20 percent, a soil will not liquefy unless its plasticity index is low or the soil is sensitive.

2. The equivalent dynamic shear stresses are calculated as follows:

\[
\frac{\tau_d}{\sigma_o'} = 0.1(M-1) \frac{a_{\text{max}}}{g} \frac{\sigma_o}{\sigma_o'} (1 - 0.015z) \quad [\text{eq.2.13.}]
\]

in which \( M \) is the earthquake magnitude and the factor \( (1 - 0.015z) \) the same as the depth reduction factor, \( r_d \) accounting deformability of the soil mass as proposed by Iwasaki et al. (1978).

3. \( N \) values are corrected for hammer energy.

4. Compute adjusted \( N_a \) values by substituting eq. 2.8 into 2.10.:

\[
N_a = \frac{1.7}{\sigma_o' + 0.7} N + \Delta N_f \quad [\text{eq.2.14.}]
\]

where \( \Delta N_f = 0 \) for fines content < 5%, and \( \Delta N_f = 0.1 \text{ FC} + 4 \) for fines content \( \text{FC} > 10\% \).

5. Compute the soil liquefaction resistance from eq. 2.12.:

\[
\frac{\tau_l}{\sigma_o'} = aC_r\left[ \frac{16\sqrt{N_a}}{100} + \left( \frac{16\sqrt{N_a}}{C_s} \right)^n \right] \quad [\text{eq.2.12.}]
\]

6. Compute factor of safety against liquefaction:
Tokimatsu and Yoshimi noted that the method is limited to level ground conditions with no static load on the surface. For conditions where static shear stresses already exist in the ground, the method will likely overestimate the liquefaction potential.

2.3.5. Evaluation of Residual Shear Strength by SPT.

2.3.5.1. Correlation proposed by Seed (1987).

Recognising the importance of field data, Seed (1987) documented cases where major sliding has occurred due to liquefaction that allowed back analysis of the strength parameters of the liquefied soil. Seed developed an empirical relationship between the residual strength of liquefied sands and silty sands to the corrected SPT-N values of the soils (Fig. 2.17.). The residual strength calculated by Seed may be considered to be analogous to the steady state strength described earlier.

Residual strengths were calculated using limit equilibrium analysis. Two extreme cases were used in the analysis: (1) Examination of potential failure surfaces involving the original pre-earthquake section of the embankment. This case represents an upper bound to the gravity driving forces acting on the sliding mass. And (2) examination of the final configuration of the slopes after the earthquake. In this case, different potential failure surfaces were studied to determine a lower bound residual strength, i.e. that required to obtain a safety factor of unity immediately after the earthquake.
Fig. 2.17 RELATIONSHIP OF UNDRAINED RESIDUAL STRENGTH CALCULATED FROM FIELD DATA AND EQUIVALENT CLEAN SAND VALUES OF N-SPT (de Alba et al., 1987).
It has to be noted that in Seed’s analysis, both the effect of the gravity forces and the earthquake base motions were considered using a Newmark-type dynamic deformation analysis. Deformations were considered to be indicated by the simplified charts of Makdisi and Seed (1977). Thus, given the measured horizontal deformations and the free-field acceleration, it was possible to back calculate an approximate value for the yield acceleration, $k_y$, defined as the average acceleration (as a fraction of $g$) producing a horizontal inertia force on the potential sliding mass which induced a condition of incipient failure defined by a factor of unity. Accelerations above the yield acceleration will cause the mass to experience permanent displacements.

Seed’s correlations of residual strength and N-SPT also considered the amount of fines in the soil. It is recognised that, even for equal conditions of pore pressure generation or relative density, the penetration resistance of silty sands is lower than that for clean sands (Seed et al., 1983). Thus, Seed expressed the effective penetration resistance of silty sand in terms of equivalent clean sand value by use of the equation:

$$ (N_1)_{	ext{effective}} = (N_1)_{\text{measured}} + \Delta N_1 \quad [\text{eq. 2.16.}] $$

where $\Delta N_1$ depends on the fines content of the silty sand. Tentative values of $\Delta N_1$ are given in Table 2.3., but judgement is required in the use of these values since fines may differ in their liquefaction characteristics from one soil to another.

2.3.5.2. Correlation proposed by Poulos (1988).

Poulos (1988) used method similar to Seed (1987) to obtain correlation of the undrained steady state strength with the N-SPT. Based on the mechanics of a liquefaction failure, the geometry of the mass before and after failure, the weight of the soil involved in the failure, and the distances traveled by the failure mass; the average shear strength that was mobilised during
Table 2.3. APPROXIMATE VALUES OF $\Delta N_i$ \hspace{1cm} (Seed, 1987)

<table>
<thead>
<tr>
<th>Fines Content (%)</th>
<th>$\Delta N_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>25</td>
<td>2</td>
</tr>
<tr>
<td>50</td>
<td>4</td>
</tr>
<tr>
<td>75</td>
<td>5</td>
</tr>
</tbody>
</table>
failure could be estimated. The correlation was established with the corresponding SPT-N value.

A simplifying step in back analyzing the field value of the mobilised shear strength, $S_m$, was the assumption that the final slope angle after failure is zero. The liquefaction flow failure was considered as movement of the center of gravity of the mass from the original position to a new lower position. For this simplified approach, the mass accelerates down an original steep slope inclined at angle $\theta$, encounters the zero slope at the base, and decelerates to zero velocity. The equation from which the value of mobilised strength can be estimated is:

$$S_m = \delta_2 \frac{w \sin \theta}{(\delta_1^2 + \delta_2^2)}$$  \[eq.2.17.\]

where $w =$ weight per unit width of sliding mass, $\theta =$ angle of slope down which the mass accelerates, $\delta_1 =$ distance a failure mass moves down the original steep slope, $\theta$, and $\delta_2 =$ distance that failure mass moves along zero slope before it comes to rest. Poulos (1985) assumed that after very little deformation, the strength drops to its undrained steady state strength, $S_{us}$, and during the few second or minutes of the failure, little drainage takes place, so that the $S_{us}$ value remains essentially constant. Thus, the average mobilised shear strength, $S_m$, calculated by equation 2.17, is very close or the same as $S_{us}$.

The data used for the analysis consisted of silty sand with fines content of 30% or higher. Values of laboratory undrained steady state shear strength were measured for undisturbed samples taken from boreholes adjacent to the SPT holes (Fig. 2.18). The lower values of $S_{us}$ are in fair agreement with field data, but the higher values are well above the field trends. Notably, the $S_{us}$ values are also considerably larger than the residual strength, determined by Seed (Fig. 2.17). To date, no explanation of the discrepancy between the data in Fig. 2.17 and 2.18 has been advanced.
Fig. 2.18 RELATIONSHIPS OF UNDRAINED STEADY STATE SHEAR STRENGTH FROM FIELD AND LABORATORY DATA WITH SPT-N VALUES (Poulos, 1985).
2.3.6. Liquefaction Potential Evaluation based on CPT.


Zhou developed a method of evaluating liquefaction potential of sand by CPT based on field performance in the Tangshan (China) earthquake area. The sand layers in Tangshan are mostly clean sands or sands with minor amount of fines. A value of critical tip resistance based on statistical treatment of field data was proposed:

$$q_{cr} = q_{so} [1 - 0.065(H_w - 2)][1 - 0.05(H_0 - 2)]$$  [eq.2.18]

where $q_{cr}$ is the critical tip resistance at the boundary soil is likely to liquefy during earthquake and $q_{so}$ is the corresponding critical tip resistance for a depth of water table, $H_w = 2$ m and thickness of overburden pressure, $H_0 = 2$ m. The proposed value of $q_{so}$ depends on the intensity of the earthquake (Zhou, 1980).

This method agreed very well with the behavior in the the Haicheng earthquake. However, considerable deviations were found when the method was used to evaluate liquefaction phenomena over the Luthai area where the soil contained more fines. Zhou suggested that this deviation was due to the difference of the soil conditions between Luthai and the previously studied area, Tangshan. The average size of sand in Luthai is 0.064 - 0.078 mm while in Tangshan 0.076 - 0.61 mm, and the fines content in Luthai is higher (50 - 65 %).

To account for fines content in evaluating liquefaction susceptibility, Zhou recommended a correction to $q_c$, $\Delta q_c$. based on a laboratory study by Masamitsu (1977). Using the conversion factor of 4.5 for the ratio of $q_c$ over N, the suggested correction term is:

$$\Delta q_c = 584 (\sigma_v' + 0.7)(\Delta R)$$  [eq.2.19.]
where $\sigma'_v$ is effective overburden pressure in bars and $\Delta R_f$ is the correction for cyclic stress ratio for liquefaction due to the increase of fines content. The value of $\Delta R_f$ is between 0.0022 FC to 0.0075 FC where FC = fines content in percent. In any case $\Delta R_f$ does not exceed a value of 0.175. Zhou found that eq. 2.19. could be used to explain behavior in the Luthai area. It is interesting to note that correction for the tip resistance, $\Delta q_c$, corresponds to that of SPT correction for fines content suggested by Seed et al. (1982).

2.3.6.2. Method proposed by Robertson and Campanella (1985)

Based on the relationship of $q_c$ vs $D_r$ from Baldi et al. (1982) and $D_r$ vs $\tau/\sigma'_v$ from Christian and Swiger (1975), Robertson and Campanella (1985) constructed a chart of correlation between $q_c$ and $\tau/\sigma'_v$ for clean sand. Observations from case histories of field CPT data where liquefaction of silty sands had occurred lead to a distinction between clean sand and silty sand where the curve of clean sand was shifted to the left about 40 bars or moved up about 0.05. This chart was then reviewed using data obtained from liquefaction in Canada, USA and Japan. These data supported the correlation of CPT data to be used for identification of liquefaction-susceptible soils (Fig. 2.19).

Like the SPT method proposed by Seed, the cone tip resistance is first normalised to an overburden pressure of 1 kg/cm² using a correction factor $C_q$ which was established using data from Baldi et al. (1981) similar to those proposed by Seed (1983) for SPT. Soils that fall within the upper shaded area of zone A in Fig. 2.11. can be considered sands with $D_{50} \geq 0.25mm$ and soils that fall within the lower hatched area of zone A can be considered as silty sands or silts with $D_{50} \leq 0.15mm$. Fig. 2.19. is to be used in the same manner as Seed’s curves. The site chosen include Vancouver, Niigata and Tokyo Bay, and the USA. The data include field CPT and SPT profiles and laboratory cyclic triaxial test. The data from Canada and from Niigata for sands with $D_{50} < 0.15$ mm plot well above the proposed CPT correlation.
Fig. 2.19 CORRELATION OF CYCLIC STRESS RATIO FOR 5% DOUBLE STRAIN AMPLITUDE AND NORMALISED TIP RESISTANCE OF CPT (Robertson and Campanella, 1985).
It interesting to note that the data for silty sands with $D_{50} < 0.15$ mm plot well above the proposed CPT correlation.

2.3.6.3. Method proposed by Ishihara (1985)

Based on the assumption that the presence of fines had a similar effect on cone tip resistance as it did for the SPT blow counts, Ishihara (1985) introduced a correction factor into the correlation of cyclic stress ratio to the cone tip resistance previously established for clean sands. In the case of the standard penetration test, the effect of fines was taken care of by increasing the N-SPT blow counts by an amount equals to $6.5 \log FC$ where $FC =$ fines content in percent. To apply the correction factor for the tip resistance of CPT, Ishihara used an average of $q_c/N$ ratio of 4. Hence the corresponding correction factor is $4 \times 6.5 \log FC = 26 \log FC$. The correlation is shown in Fig. 2.20.

2.3.6.4. Method proposed by Seed and de Alba (1986).

Seed and de Alba (1986) published a modification of Seed’s earlier theory of predicting liquefaction susceptibility using the SPT which incorporates the use of cone penetration test results. Their method is based on the relationship between CPT tip resistance and SPT-N values, and on CPT results from liquefied deposits. As shown in Fig. 2.21., the correlations are presented in the same fashion as in SPT method, and the design procedure employed is much the same. However, unless the grain size characteristics of the soils involved are known with a good degree of reliability, use of this chart may lead to uncertainty owing to the sensitivity to $D_{50}$.

2.3.6.5. Method proposed by Shibata and Terapaksa (1987, 1988)

Shibata and Terapaksa (1987, 1988) proposed the latest liquefaction assessment method by CPT results for clean sand and silty sand based predominantly on the field performance that
$q_2 = q_1 + 26 \log FC$

Fig. 2.20 CPT-BASED METHOD FOR LIQUEFACTION POTENTIAL EVALUATION WITH CORRECTION OF FINES CONTENT (Ishihara, 1985).
Fig. 2.21 SPT-BASED CONVERSION TO CPT FOR LIQUEFACTION ASSESSMENT
(Seed and de Alba, 1986).
Fig. 2.22 PROPOSED CORRELATION OF CYCLIC STRESS RATIO FOR (a) CLEAN SANDS AND (b) SILTY SANDS BASED ON CPT (Shibata and Terapaksa, 1988)
has occurred during the past several earthquakes. Fig. 2.22 shows the proposed correlation in the form of the normalised cone resistance, $q_{c1}$, and cyclic stress ratio, $\tau/\sigma_o'$, that develop in the field during an earthquake. This applies for both clean sands with $D_{50} \geq 0.25 mm$ and silty sands with $D_{50} < 0.25 mm$. Their findings confirm that there is fairly well defined trend in which the upper bound of $q_c$ values for liquefied soils decreases with an increase in mean grain size for $D_{50} < 0.25 mm$. This suggests that given the same value of $q_c$, liquefaction resistance is greater for smaller grain sizes. On the other hand, for clean sands with values of $D_{50} \geq 0.25 mm$, the upper bound of $q_{c1}$ for liquefied soils seems to be independent of grain size. The correlation was applied to develop a procedure for liquefaction evaluation as follows:

1. The normalized cone resistance which is the corrected $q_c$ values for an effective overburden pressure, $\sigma_o'$, of 1 kg/cm$^2$ is approximated by:

$$q_{c1} = C_1 \cdot q_c = \left( \frac{1.7}{\sigma_o' + 0.7} \right) \cdot q_c \ (kg/cm^2) \ [eq.2.20]$$

where $C_1$ is a function of $\sigma_o'$ in kg/cm$^2$ at the depth where CPT was performed.

2. The cyclic stress ratio, $\tau/\sigma_o'$, that develops in the field during an earthquake was estimated from the following equation (Tokimatsu & Yoshimi, 1983):

$$\frac{\tau}{\sigma_o'} = 0.1(M - 1) \frac{a_{max}}{g} \frac{\sigma_o}{\sigma_o'} (1 - 0.015z) \ [eq.2.13.]$$

where $M$ is the earthquake magnitude, $a_{max}$ is the maximum acceleration at ground surface, $\sigma_o$ and $\sigma_o'$ are the total and effective overburden pressures and $z$ is the depth in meters.

3. The critical boundary that separates liquefaction and no-liquefaction condition was expressed in terms of normalised cone resistance, cyclic stress ratio and mean grain size of soils by the following equations:
\[ (q_{c1})_{cr} = C_2 \left[ 50 + 200 \left\{ \frac{(\tau/\sigma_0') - 0.1}{(\tau/\sigma_0') + 0.1} \right\} \right] \]  [eq.2.21]

or

\[ \left( \frac{\tau}{\sigma_0'} \right)_c = 0.1 + 0.2 \left\{ \frac{(q_{c1}/C_2) - 50}{250 - (q_{c1}/C_2)} \right\} \]  [eq.2.22]

where \( C_2 = 1.0 \) for clean sand with \( D_{50} \geq 0.25 \text{mm} \) and \( C_2 = D_{50}(\text{mm})/0.25(\text{mm}) \) for fine grained soils with \( D_{50} < 0.25 \text{mm} \). In the above equations, the critical corrected cone penetration values, \( q_{c1} \), can be estimated for any given values of cyclic stress ratio generated by the earthquake. In the same manner, for any given CPT \( q_c \)-value, the cyclic strength of soils can be evaluated also.

2.3.6.7 Method proposed by Been et al. (1987, 1988)

Been and Jefferies (1986) developed a relationship between the CPT tip resistance and state parameter for several sands (Fig. 2.23) based on calibration chamber results. The correlation suggested that the normalised tip resistance is a function of state parameter, \( \psi \), and the slope of the steady state line, \( \lambda_{ss} \). Although this relationship was not initially intended for liquefaction assessment, the strong dependency of \( q_c \) to the state parameter, \( \psi \) suggests that this relationship can be applied to measure the state of the soil in situ such as presented by Been et al. (1987).

Recently Jefferies (1988) proposed a generalization of the CPT-\( \psi \) correlation by incorporating the slope of the steady state line, \( \lambda_{ss} \). From the available data, the proposed single correlation is presented by the following expression:

\[ \psi = - \frac{1}{[8.1 - \ln(\lambda_{ss})]} \cdot \ln \left[ \frac{(q_c - \sigma_m)/\sigma_m'}{8 + 0.55/(\lambda_{ss} - 0.01)} \right] \]  [eq.2.23]
Fig. 2.23 RELATIONSHIP OF NORMALISED CPT DATA TO STATE PARAMETER
(Been and Jefferies, 1987).
where $\psi$, $\lambda_{ss}$, $\sigma_m$ and $\sigma_m'$ are previously defined. The use of mean effective stress as a measure of stress level in both the definition and the interpretation of the CPT place strong emphasis on the knowledge of horizontal stresses. Field measurement of horizontal stresses is recommended using either a self-boring pressumeter or dilatometer.

Based on this CPT - $\psi$ correlation, Jefferies proposed a method to determine in situ void ratio which involve steps as summarized in Fig. 2.24. First the SSL for the particular sand is determined by laboratory test. Step two involves estimation or measurement of $\sigma_{ns}$ and $\sigma_v'$. The SSL provides a value of $\lambda_{ss}$ which allows $\psi$ to be evaluated using equation 2.23. and the value of $q_c$ (step three). With $\psi$ and the SSL known, it is then possible to calculate $e_{ss}$, the void ratio corresponding to $\sigma_v'$, then the in situ void ratio ($e$) since by definition:

$$e = e_{ss} + \psi \quad [eq.2.24.]$$

2.3.7. Example of Analysis: Imperial Valley Earthquake, A Case History

Information collected from case histories is an important component in understanding soil behavior; particularly for studying the phenomena of soil liquefaction. Data from field explorations contribute to the knowledge of liquefaction and can be used to support method of liquefaction evaluation. The following case history is presented to better understand the methods of liquefaction potential evaluation.

Bennett et al. (1981) documented geotechnical properties of soils that did and did not liquefy during the 1979 Earthquake that struck the Imperial Valley near El Centro, California. Damage caused by liquefaction included differential settlements, fissures, fractured tile drains in fields, disrupted pavement on roads, and slumped banks along canals and drains (Youd and Wieczorek, 1982).
DETERMINE SSL BY INTERPOLATION OF TESTS OF RECONSTITUTED SAMPLES

CALCULATE $\psi$ FROM CPT USING $\lambda_{ss}$ AND $p'$

MEASURE $\sigma'_{nr}$ & HENCE CALCULATE $p'$ IN-SITU

LOG$_{10} (p')$

VOID RATIO

$\lambda_{ss}$

$\psi$

$\phi_{ss}$

$\phi = \phi_{ss} + \psi$

Fig. 2.24 SUMMARY OF METHODOLOGY TO DETERMINE VOID RATIO IN SITU (Jefferies, 1988).
A map of the area affected by the October 15, 1979 earthquake ($M_L = 6.6$) with pertinent geographic features is shown in Fig. 2.25. Two sites where liquefaction effects were particularly pronounced were selected for subsurface investigation: Heber Road site, where a lateral spread disrupted Heber Road, adjacent fields and a canal; and River Park site where hundreds of sand boils were found. Typical soil profile at Heber Road site is shown in Fig. 2.26.

2.3.7.1. Analysis based on Pore Pressure Generation.

At Heber Road site, Youd and Bennett (1983) designated the soils as $A_1$, $A_2$, and $A_3$. Of interest to this study is unit $A_2$, the central part of the section which is a *loose, very fine sand and silty sand*; a natural channel fill deposited by an ancient stream. Surficial effects of liquefaction were concentrated over unit $A_2$.

At River Park Site, the soil were also classified into 3 units A, B and C. Unit A, a layer of brown flood plain sediment 1.8 m to 2.5 m, thick lies immediately beneath ground surface. The texture varies from sandy silt to silt to clayey silt with cone resistances range from 10 - 40 kg/cm² and N values range from 1 to 7. Unit A overlies a 0.9 - 1.5 m thick layer of soft to medium-stiff clay and silty clay (unit B) having cone resistance range from 2-30 kg/cm². Unit C is a massive bed of gray point bar deposit. The loose zone at the top of unit C was likely produced by upward migration of pore water during the 1979 and past earthquakes as a consequence of densification of sand in the underlying layer.

For the analysis, Youd and Bennett used the simplified method of Seed et al. (1979). The N-SPT values were corrected for overburden pressure and incremented by 7.5 to correct for a mean grain size less than 0.15 mm. Evaluation of the cyclic stress ratios were estimated using eq. 2.3. based on $a_{max} = 0.8$ g. The cyclic stress ratios for the Heber Road Site and the River Park Site were found to be 0.75 and 0.25 respectively. At the Heber Road Site, unit $A_2$ and $A_1$ agree with the field observation, while unit $A_3$, although plotted on liquefied side of Seed's curve, surficial liquefaction were not detected (Fig. 2.27.a.) Both units A and C of the...
Fig. 2.25 MAP OF IMPERIAL VALLEY (Youd and Bennett, 1979).
Fig. 2.26 SOIL PROFILE AT HEBER ROAD SITE (Castro, 1987).
Fig. 2.27. ANALYSIS BASED ON CYCLIC STRESS RATIO

(A) YOUD AND BENNETT (1983)

(B) SEED AND DE ALBA (1986)
River Park Site were prone to liquefaction based on this analysis and examination of textures of sand boil deposits confirmed this.

A similar study to that of Youd and Bennett was done by Seed et al. (1986) by comparing data from unit $A_2$ and $A_3$ with the boundary curve they proposed for silty sand with $D_{50} = 0.1$ mm. A ground surface acceleration of 0.55 g was used to calculate the stress ratio in the field using eq. 2.3. No explanation was provided as to why the surface acceleration was different from that used previously by Youd and Bennett (1983) other than a reference to a personal communication with Youd. The results of Seed et al. analysis (Fig. 2.27.b.) shows that the proposed limiting curve appropriately separates the unit which liquefied from the one which did not.

2.3.7.2. Analysis Based on The Steady State Shear Strength

Unlike the analyses based on pore pressure generation as described previously, Castro presented the case at Heber Road Site as a problem of lateral spreading. The driving shear stress is very small since the overall ground surface inclination is very gentle. By stability analysis, Castro (1987) estimated a value of 40 psf as the driving shear stress. Castro (1987) viewed the phenomena at Heber Road site as lateral spreading mechanism where deformation of a stable mass accumulate during earthquake.

A Newmark-type of analysis was used to calculate slope movement which was assumed to be initiated when the earthquake acceleration exceeded the yield acceleration of the soil mass. The average acceleration of the mass was computed using the program SHAKE developed at the University of California, Berkeley (Schnabel et al., 1972) with soil moduli obtained from cross hole seismic data by Sykora and Stokoe (1982). An earthquake record with peak acceleration of 0.8 g was applied to the base of the loose sand, i.e., at the top of the denser underlying soil. The resulting acceleration of the mass that moved has a peak value equal to 0.47g.
Fig. 2.28 LIQUEFACTION ANALYSIS BASED ON FLOW FEATURE MECHANISM (Castro, 1987).
The yield acceleration, \( k_y \), was obtained from a pseudostatic stability analysis for a sliding surface along the base of the mass that moved. The sliding surface was selected to be consistent with the location of observed crack. The yield acceleration, \( k_y \), was computed as a function of the yield strength, \( S_y \), in the loose sand layer and is presented in Fig. 2.28. The computed displacements for various assumed yield accelerations are shown in the same figure. In the case of Heber Road Site, the center of gravity moved about 5.5 ft which is the average of the measured displacements at the road and at the canal. A displacement of 5.5 ft corresponds to a yield acceleration of 0.05 g and yield strength of 100 psf. Castro noted that this very low value of yield strength is consistent with the very loose condition of the soil as indicated by the measured blowcount of SPT of 0 to 1 and the cone penetration of 5 - 15 kg/cm² (Fig. 2.26).

The Newmark analysis assumes that the soil has a rigid plastic type of behavior as shown by the dashed line in Fig. 2.28. Castro suggested that the actual stress strain behavior is approximately as given by the solid line in the same figure, and that the yield strength is approximately equal to the undrained steady state strength (\( S_u \)) soon after the start of the earthquake. Thus it was concluded that \( S_u \) in the silty sand at Heber Road is about equal to 100 psf. The low value of \( S_u \) is consistent with the low tip resistance of CPT (about 5 - 15 bars) and SPT blow counts (0 - 3).

Castro noted that the driving shear stress of 40 psf is smaller than the back figured strength of 100 psf, which is consistent with the assumed mechanism of lateral spreading, i.e., the case of limited (although damaging) deformations and not one of the instability and flow (liquefaction) slide.
2.4. SUMMARY.

The preceding sections shows that many different theories have been provided for the assessment of the liquefaction potential of soils. Each of these theories differs in the assumptions involved in the analysis, their use of laboratory and in situ data, and in the procedure of evaluation.

Some effects of liquefaction in the field, such as the occurrence of sand boils and the differential settlement of structures due to uneven post earthquake densification of the foundation soil, can be explained by the presence of excess pore pressures and associated hydraulic gradients. Other manifestations of seismically-induced liquefaction, which are associated with large or unlimited shear straining of the soil, can be explained by the decrease in shear strength associated with excess pore water pressure. Both pore water pressure generation and the decrease in shear strength are studied in this research and correlated to the CPT.

While Seed and his colleagues explained the phenomena from the point of view of the generation of high pore pressure and large accumulated cyclic deformations due to shear stress developed during earthquake, Casagrande and his followers have expressed the importance of the critical void ratio on the behavior of the soil during monotonic and cyclic loading. The approach by Seed et al. is based on the cyclic strength ratio that can be obtained by cyclic triaxial test in laboratory or by empirical correlation based on field evidence. The approach by Casagrande and his colleges requires the existence of driving shear stresses for liquefaction failure to occur, i.e. if the driving shear stress is lower than the undrained steady state strength, then a liquefaction failure will not occur. In such a case, cumulated strain will produce lateral spreading. Also, definition of liquefaction by Castro involves large shear deformations which occur at relatively rapid rate so that it appears to be flowing. The steady state approach is based on laboratory stress-strain behavior and the undrained residual strength.
In an effort to clarify the mechanism of earthquake induced phenomena, Castro (1987) classified the phenomena into three different categories, i.e., (1) sand blows and settlement of level ground, (2) flow slide liquefaction and (3) limited deformation or lateral spreading. The main characteristics of the three induced phenomena are presented in Table 2.4, where the controlling soil properties of mechanism (1) were shear modulus, permeability and compressibility, while the controlling soil parameter for mechanism (2) and (3) was the steady state shear strength. However, Seed and his colleagues prefer relative density as the key index characterizing the cyclic strength of the soil.

Any of the approaches for liquefaction evaluations require the knowledge of void ratio in situ, but accurate measurement of the in situ properties of soil is difficult. Under sampling, the structure of cohesionless material may collapse at small strains, and any form of undisturbed sampling in sands is difficult and expensive. For these reasons, considerable effort is underway to develop empirical correlations between the cyclic strength of the soil and in situ test results. It is the objective of this research to develop such correlation with particular interest in the use of the cone penetration test and emphasis on silty sand. It is of interest that the data shown in Table 2.1 and Fig. 2.12 and 2.13 led to the conclusion that the liquefaction resistance of sand and silty sand is similar. It appears that the primary differences in these materials are their respective penetration resistance.

Presently, the liquefaction of a given soil deposit is often estimated on the basis of the standard penetration test (SPT) and cone penetration test (CPT). It is generally recognized that the blow count of the SPT, and the cone penetration resistance, \( q_c \), are influenced by the amount of fines present in the soils. It is also known that both N-SPT and \( q_c \) value are dependent on the density of the sand as well. A decrease in N-SPT or \( q_c \) can be caused by either an increase of fine contents or a decrease of density of the sand. Therefore, if the blowcount value is low for a given soil, it could mean either one of the following soil conditions: (1) When the soil is clean sand, the possibility is that it is of low density and hence susceptible to
Table 2.4. SUMMARY OF CHARACTERISTICS OF EARTHQUAKE INDUCED PHENOMENA
(Castro, 1987)

<table>
<thead>
<tr>
<th></th>
<th>Sand Blows Settlemens of Level Ground</th>
<th>Flow Slides- Liquefaction</th>
<th>Limited Deformation- Lateral Spreading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driving Shear Stresses, $\tau_d$</td>
<td>No</td>
<td>Yes, $\tau_d &gt; S_{u_s}$</td>
<td>Yes, $\tau_d &lt; S_{u_s}$</td>
</tr>
<tr>
<td>Undrained Steady State Strength, $S_{u_s}$</td>
<td>Not a Factor</td>
<td>Triggers failure</td>
<td>Accurates Deformation if $\tau_e + \tau_d &gt; S_{u_s}$</td>
</tr>
<tr>
<td>Role of Earthquake Stresses, $\tau_e$</td>
<td>Increase pore pressure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress Strain Behavior</td>
<td>Not relevant</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Controlling Soil Properties</td>
<td>Shear modulus permeability, compressibility</td>
<td>Undrained Steady State Strength</td>
<td>Undrained Steady State Strength</td>
</tr>
</tbody>
</table>
liquefaction; or (2) if some fines are present in the soil, it may also be low in density but less vulnerable to liquefaction. Some researchers proposed methods for correction of the effects of fines on the evaluation of liquefaction potential using CPT. Table 2.5. gives summary of the methods.

Investigation of the correlation of the CPT and the cyclic and monotonic behavior of cohesionless soil is difficult due to the disturbance effect during sampling. On the other hand the study of loose silty sand on the performance of CPT and on its response to cyclic or monotonic loading can be made possible by the use of calibration chamber. If the same fabric can be similarly produced in both laboratory specimens and calibration chamber specimens, the mechanical behavior should be same in both cases.
Table 2.5. SUMMARY OF LIQUEFACTION POTENTIAL EVALUATION OF SILTY SAND USING CPT

<table>
<thead>
<tr>
<th>PROPOSER</th>
<th>METHOD</th>
<th>CORRECTION FOR FINES CONTENT</th>
</tr>
</thead>
</table>
| ZHOU (1981) | - field performance data  
  - statistical analysis  
  \[ \Delta q_c = 584 (\sigma_0' + 0.7)(\Delta R)^2 \]  
  \[ \Delta R_s = 0.00225 - 0.00755(\leq 0.175) \]  
| ROBERTSON & CAMPANELLA (1985) | - develop chart of \( \frac{q'_c}{\sigma_0'} \) vs. corrected tip resistance based on data of CPT vs. \( D_r \) and \( (\eta/\rho_o') \) vs. \( D_r \)  
  - SPT-CPT correlation  
  - chart verified by field data  
  - Shift curve for sand about 40 bars to the left or 0.05 up  
| SEEDE et al. (1986) | - use large body of SPT data with direct conversion of SPT to CPT  
  - chart verified by field data  
  - the conversion use SPT-CPT correlation that include the mean grain size, \( D_{50} \)  
| SHIBATA & TERAPAKSA (1988) | - predominantly field performance data of CPT  
  - use a factor, \( C_2 \), to adjust for the mean grain size  
  \[ (q_c)_0 = C_2 \left[ 50 + 200 \left( \frac{\eta/\sigma_o'}{\rho_o'} - 0.1 \right) \right] \]  
  \[ C_2 = 1.0 \text{ for } D_m \geq 0.25 \text{mm} \]  
  \[ C_2 = D_m(\text{mm})/0.25(\text{mm}) \text{ for } D_m < 0.25 \text{mm} \]  
| BEEN et al. (1986, 1987) | - use state parameter concept based on steady state line  
  - use tip resistance to measure \( Y \)  
  - the method applies for all cohesionless soil  
|
3.1. INTRODUCTION

The experimental program in this investigation consisted three principal phases:
1.) Laboratory tests to determine sample fabrication techniques, index property evaluation, and cyclic and static soil properties.
2.) Cone penetrations tests conducted in the Virginia Tech calibration chamber.
3.) Field cone penetration tests

Two different silty sands formed the material used in the investigation.

3.2. LABORATORY TESTING PROGRAM

The laboratory testing program consisted of two parts. First, techniques of sample fabrication were investigated with emphasis placed on forming samples of silty sand for both calibration chamber testing and triaxial testing. This presented a challenge in that the fabricated silty
sand specimens should have the same fabric, void ratio, and stress history for small 1.4”x3.0” triaxial specimens and very large 60”x60” calibration chamber specimens. Further, the fabric of the soil specimens should be consistent with that found in field conditions. Second, property evaluation tests were conducted to collect the various parameters necessary for investigation of the different theories for liquefaction evaluation.

### 3.2.1. Sample Fabrication Techniques

Four techniques were studied to arrive at a sample fabrication method:

1. Pluviation through air
2. Pluviation through vacuum
3. Moist tamping
4. Consolidation from a slurry

Pluviation through air involves raining the material through a nest of sieves. This method has been used successfully for deposition of clean sands (Villet; 1975, Sweeney, 1987). One possible problem with pluviating a silty sand through air is possible segregation of the material, thus creating a stratified specimen. As an attempt to minimize this, pluviation through an evacuated chamber was also investigated.

Moist tamping has been widely used for fabrication of soil samples for triaxial testing. This technique allows a good control of soil density, thus it was included in this study.

The fourth alternative, consolidation of the soil from a slurry, is based on a similar process of the natural deposition of alluvial soils. Alluvial soils are perhaps the most prone to liquefaction, therefore this sample fabrication techniques was studied.
3.2.2. Laboratory Tests for Property Evaluation

Laboratory tests were conducted to obtain index, compression, and cyclic and monotonic strength properties. Sieve, hydrometer, and specific gravity analysis were performed. Atterberg limit tests were attempted, but due to the lack of plasticity of the fines, were not possible. Also, one dimensional consolidation tests were conducted to provide information concerning the behavior of the soil for the slurry consolidation sample fabrication technique.

Monotonic and cyclic triaxial tests were conducted to provide the information necessary for investigating the different theories for liquefaction evaluation. This included:

- the steady state line (e versus $S_u$)
- the slope of the steady state line, $\lambda_{ss}$.
- undrained steady state shear strength, $S_u$
- peak and residual effective stress friction angle
- number of cycles to liquefaction for a given cyclic stress ratio.

In all, 15 isotropically consolidated undrained triaxial tests (ICU) and 42 cyclic isotropically consolidated triaxial tests (CICU) were performed.

3.3. Cone Penetration Tests in the Calibration Chamber

The Virginia Tech calibration chamber, one of the largest in existence, affords a unique opportunity for controlled cone penetration testing. A soil specimen, approximately 5 feet in diameter by 5 feet tall, can be subjected to horizontal and vertical stresses up to 700 kPa. The chamber will allow up to seven cone penetration tests to be conducted in one test specimen. For this study, a 10 cm$^2$ standard Fugro electric cone and a 4.2 cm$^2$ miniature cone developed
at Virginia Tech were used. Five calibration chamber specimens were fabricated, and 5 standard cone tests and 18 miniature cone tests were conducted.

3.4. CONE PENETRATION TESTS IN THE FIELD

In addition to the calibration chamber tests, 8 cone penetration tests were conducted in situ. All tests were conducted next to the downstream left abutment of the Pepper’s Ferry Bridge between Radford and Blacksburg. At this site adjacent to the New River, 4.2cm$^2$, 10cm$^2$, and 15cm$^2$ were used. Standard penetration tests were also conducted and both disturbed and undisturbed samples were taken for laboratory testing.

3.5. SOILS

Two different soils were selected for this study, Yatesville silty sand and Pepper’s Ferry silty sand. These soils were selected based on their grain size distribution, evidence of liquefaction susceptibility in situ and availability.

3.5.1. Yatesville Silty Sand

Approximately 90 tons of Yatesville silty sand were acquired for this research. This alluvial soil from Lawrence County, Kentucky, is from the site of the Yatesville Lake Dam on Blaine Creek. Yatesville Dam was first designed to be founded on this soil, but subsequent
liquefaction analysis conducted by the U.S. Army Corps of Engineers deemed this soil to be prone to liquefaction. The soil for this investigation was excavated from the site and delivered to Virginia Tech and a sufficient amount was processed to eliminate organics and gravel particles. It was used both for laboratory tests and for calibration chamber tests. Yatesville silty sand contains approximately 40% non-plastic fines, and has a specific gravity of 2.67. A gradation curve is shown in Fig. 3.1.

3.5.2. Pepper's Ferry Silty Sand

The Pepper’s Ferry field test site has been used for cone penetration testing in several projects at Virginia Tech. This site contains alluvial deposits of silty sand very similar to Yatesville silty sand, and was chosen because of the assessibility of the site for field testing, and the wealth of data previously collected at the site. This material has approximately 25% non-plastic fines and specific gravity of 2.67. A gradation curve is shown in Fig. 3.2.
Fig. 3.1. GRAIN-SIZE DISTRIBUTION OF YATESVILLE SILTY SAND
Fig. 3.2. GRAIN-SIZE DISTRIBUTION OF PEPPER'S FERRY SILTY SAND
4.1. INTRODUCTION

There is evidence that preparation method strongly influences the measured static and cyclic strength and deformation characteristics of cohesionless soils (Ladd, 1974; Miura and Toki, 1982; Mori et al., 1978; Nemat Naser et al., 1984; and Miura et al., 1984). This has been found to especially be true for sands with elongated, angular particles (Oda, 1972). Careful attention is required to obtain accurate and reproducible data for reconstituted specimens of cohesionless soils. The specimen must have a uniform density throughout and no segregation of coarse and fine particles. Also, the specimen preparation technique should model as closely as possible the in situ soil fabric.

The objective of this phase of the study was to examine methods of sample fabrication for silty sand. Not only should the samples have a fabric similar to silty sand in situ, but the fabrication method should be applicable for both small scale samples (triaxial test specimens) and large scale samples (calibration chamber specimens). It was expected from the outset of this work
that conventional methods of sample preparation would not work for silty sand since the silt fraction of the soil is so fine that gravity forces do not fully control the fall of the material through air or water. Also, in the presence of the humidity of the air, the fine particles tend to adhere, forming clumps in the soil and sticking to parts of equipment.

4.2. PLUVIATION TECHNIQUES

Deposition by pluviation usually involves the free fall of soil through a media (air or water) so that the particles come to rest in a certain repeatable configuration. The rate of deposition is controlled by the aperture through which the soil flows. The materials can be poured either through an opening or rained from a surface which may cover the full cross sectional area.

The pluviation technique has advantages over other techniques because of its simple operation and is considered to approximate a natural deposition process. Both the nature of soil anisotropy and fabric obtained by pluviation methods can satisfactorily duplicate those observed in a natural alluvial environment (Oda et al., 1978). Hence this technique of sample preparation allows a convenient study of mechanical response of natural sandy soil.

4.2.1. Previous Studies

For clean sands, there are established sample preparation procedures involving most of the previous studies of pluviation in air or water. ASTM D2049-69 provides a method to determine the maximum void ratio of uniform cohesionless soil based on pluviation. The method uses a funnel with an inner spout diameter of 0.5 in. The soil is deposited into a mold of 0.1 cu.ft. while maintaining a constant drop height of 1 in. by raising the funnel as the mold is filled.
The funnel is rotated in a spiral motion from center to the edge of the mold to form a soil layer of uniform thickness.

Castro (1969) used similar procedure as ASTM D2049-69 except that a dispersion plate was attached below the top of the funnel opening and the mold was smaller. The ASTM method was modified by Mulilis et al. (1975) to achieve a loose soil structure within 2.8 in diameter triaxial mold. They varied the fall height from 6 to 20 in. Descriptions of the ASTM, Castro and Mulilis methods are presented in Fig. 4.1.

Studies of pluviation are reported by Kolbuszewski (1948), Tatsuoka (1982), Miura and Toki (1982), Vaid and Negussey (1984), Jacobsen (1976), Eid (1987) and Rad et al. (1986). Of these, Jacobsen (1976) and Eid (1987) focussed on pluviation in large calibration chamber. Key findings from these studies are:

1.) There are potentially many parameters that influence the results of pluviation - drop height, intensity of deposition, whether or not sieves are used to spread the sand, sizes of openings in the sieves, distances between the sieves, and manner in which the pluviation process is begun. Of these it is best to control the process so that intensity of deposition is the key parameter.

2.) For a given set of parameters, different results are obtained depending on the grain size and grain size distribution of the sand.

3.) Dropping sand through a set of sieves produces a uniform rain of sand which yields a uniform density in a large calibration chamber.

4.) Dropping sand from a height beyond that where the terminal velocities of the particles are reached does not increase density.

5.) Pluviation with clean sand using a sieve system to form a sand rain can produce a wide range of densities from very loose to denser than that obtained using the ASTM procedure.
Fig. 4.1. ALTERNATIVE PLUVIATION TECHNIQUES FOR SAMPLE FABRICATION
4.2.2. Pluviation of Silty Sand through Air

The pluviation through air trials focused on forming 3 in. diameter specimens and used four alternative approaches:

1. Dropping the soil through a small pluviator which incorporated a set of diffuser sieves to distribute the soil in the form of rain. The upper portion of the pluviator was a hopper from which the soil flowed through a set of holes drilled in a plate.
2. Dropping the soil from an inverted flask into only the sieve system of the pluviator (this meant that the hopper and its perforated plate were bypassed).
3. Dropping the soil directly from the inverted flask, eliminating use of the pluviator altogether. This process duplicates that of Mulilis et al. (1975) as shown in Fig. 4.1.
4. Same as (3) with addition of a small amount of water to the soil in an attempt to aggregate the silt and sand particles to minimize segregation.

The pluviator was designed by Eid (1987), and it had been used effectively in forming samples of clean sands. It was of particular interest because it simulated the large scale process used for clean sands in the calibration chamber. The pluviator was a plexiglass cylinder with the upper portion set aside as a hopper to hold the soil. Beneath the hopper was a perforated plate to allow the soil to fall in thin jets onto a set of diffusser sieves, which converted the sand into a uniform rain that fell into the sample chamber. In the flask process, the flask was capped by a rubber stopper which has a hole drilled in it. A machined brass insert was placed in the hole. The insert also had a hole in it and the size of the hole could be changed by having multiple sets of inserts.

The soil used in the pluviator trial was oven dried Yatesville silty sand which was processed so that only the portion passing the No. 10 sieve was present. It was apparent from the early trials that the Eid pluviator would not work for silty sands because the soil simply would not
pour in a regular fashion through the diffusser sieves. The fines would come to rest on the
sieves and clog up the holes, causing the remaining soil to pile up on the sieves. This process
was made more difficult as the silty sand did not pour in a steady continuous fashion through
the perforated hopper plate.

By removing the hopper system and feeding the soil to the diffusser sieves using the inverted
flask, it was hoped that a steadier flow of soil onto the the sieves could be obtained. Four
different stopper inserts were used with diameter of 4, 6, 8 and 11 mm. Deposition rates of the
silty sand using the different inserts are given in Table 4.1. As expected, much higher rates
are obtained with increases in size of aperture. Unfortunately, problems still arose with soil
going caught on the diffusser sieves and this disrupted the raining process leading to non
uniform sample formation.

Finally, the pluviator was eliminated entirely and only the flask was used to place the soil.
When the flask was inverted, and sand was being poured, the flask was moved in a circular
fashion over the top of the sample to distribute the soil. This technique was similar to that of
Mulllis et al. (1975). Drop height and aperture were varied.

With flask pluviation, it was possible to create a sample. However, during deposition a small
cloud of fines would form in the air indicating segregation of the particles in the soil. Fig. 4.2.
shows the variation of density due to drop height, and Fig. 4.3. the effect of deposition rate
on the density obtained. As opposed to clean sands, the sample density did not change with
the change in drop height. The range of densities for various drop heights was less than 5
pcf. (0.785 kN/m³) and the void ratios of the samples obtained were very high. The range of
densities induced with varying deposition rate is more significant than those observed for drop
height. The highest dry densities were obtained at the smallest deposition rates. Unfortu-
nately, even the highest dry densities (85-90 pcf.) are still too low to be representative of most
silty sand in situ (Fig. 4.3). Fig. 4.4. shows range of void ratios for pluviated Yatesville silty
Table 4.1. RECORD OF DEPOSITION RATE (g/sec) vs. DIAMETER OF APERTURE

<table>
<thead>
<tr>
<th>Test #</th>
<th>Diameter of opening</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4mm</td>
</tr>
<tr>
<td>1</td>
<td>1.48</td>
</tr>
<tr>
<td>2</td>
<td>1.62</td>
</tr>
<tr>
<td>3</td>
<td>1.66</td>
</tr>
<tr>
<td>4</td>
<td>1.65</td>
</tr>
<tr>
<td>5</td>
<td>1.31</td>
</tr>
<tr>
<td>average</td>
<td>1.54</td>
</tr>
</tbody>
</table>
Fig. 4.2. EFFECT OF DROP HEIGHT ON DENSITIES OF PLUVIATED SAMPLES
Fig. 4.3. EFFECT OF DEPOSITION RATE ON DENSITIES OF PLUVIATED SAMPLES
sand versus those of two standard sands. This figure also shows that the range of void ratios of the pluviated samples were much higher compared than those obtained by compaction and consolidation of silty sand from a slurry as discussed subsequently.

The final series of tests were conducted with the soil mixed to 1.5% water content to help control segregation. It was found that the deposition rate of the moist sand could not be controlled by the size of the flask opening. No cloud of fines were seen since the fine particles aggregated together, but the material frequently obstructed the flask and consequently no control of the deposition rate was possible. The densities obtained were slightly lower than that of the dry samples.

4.2.3. Pluviation of Silty Sand through Vacuum

To minimize segregation and to increase velocity and energy at impact, pluviation through vacuum was attempted. This was investigated at small scale with expectation to extend it to large scale if it succeeded. The primary interest in this attempt was that in the presence of a vacuum, the silt fraction would have the same falling velocity as the larger sand particles, thus preventing segregation.

4.2.3.1. Theoretical Consideration

Under a vacuum, there is no air resistance to a falling object, hence theoretically particles will fall freely without deceleration. Compared to pluviation in air, there is a substantial gain of velocity and energy at impact for particles dropped under vacuum. There is no terminal velocity and height of fall should becomes an important factor.
Fig. 4.4. COMPARISON OF EFFECT OF DROP HEIGHT ON DENSITIES OF SILTY SAND AND CLEAN SAND
Vaid and Negussey (1984) presented theory on calculating the terminal velocity of spherical particles falling freely in air. Fig. 4.5.a. shows the influence of particle size on impact velocity as a function of drop height using the Vaid and Negussey theory. They stated that the magnitude of terminal velocity attained increases almost linearly with an increase in particles size, but the drop height needed to attain this also increases. Their study is extended for particles falling through vacuum. Fig. 4.5.b. shows the increase in ratios of energy between particles drop under vacuum and through air. For example, with 50 cm height of fall, the ratio of velocity under vacuum to air is 5.6 and 1.3 for particles of diameter 0.1 mm and 0.4 mm respectively. For the same drop height and diameters, the ratio of energies are 31.8 and 1.69. This shows that the smaller particles gain much more impact energy than if pluviated in air.

Assuming that the silty sand material can be represented by its mean grain size, \( D_{50} \), then Yatesville silty sand particles with \( D_{50} \) equals to 0.1 mm pluviated under vacuum at the above drop height will increase velocity 460% and energy at impact 3080% compared to that if pluviated through air. Therefore this technique showed promise in increasing density and decreasing segregation as opposed to pluviation through air.

4.2.3.2. Design of Vacuum Pluviator

The vacuum pluviator was designed for detailed study of two parameters: intensity of deposition and height of fall. Intensity of deposition was varied by the size of funnel openings, and height of fall was varied by using extension tubes. Two funnel openings of 8 mm and 11 mm were used and extension tubes were constructed in increments of 5 in. high. To allow visual observation of the pluviation process, a plexiglass tube was used. To prevent air leak during vacuum process, the system was made air tight using rubber O-ring between joints. The pluviation mold was 5" high and 3" in diameter giving a volume of 662.3 cm\(^3\). For this pluviator, the position of the funnel is fixed, hence it is understood that material deposited at the bottom of the collector will fall with a higher impact energy than material deposited in the upper portion. A schematic of the vacuum pluviator is shown in Fig. 4.6.
Fig. 4.5. (a) THEORETICAL IMPACT VELOCITY OF FALLING PARTICLES
(b) INCREASE OF ENERGY RATIO IN PLUVIATION THROUGH VACUUM
Fig. 4.6. THE VACUUM PLUVIATOR
4.2.3.3. Test Procedure and Results

Soil was placed in the pluviator with the pluviator in an upside down orientation. A vacuum was applied through an air water interface. In this way, any dust sucked by the vacuum system will be collected in the bottle and vacuum condition in the pluviator can be confirmed when there is no more air bubbles in the water. When vacuum was achieved, the pluviator was turned over to start pluviation process. For these tests, two diffusser sieves were initially used but then removed due to problem that soil was trapped on top of the sieves. At the end of the pluviation, the supply mold and the body were removed carefully and the surface of the sample was leveled so that the volume and weight of the material could be measured.

Observation during the vacuum pluviation showed that there was much less dust in the tube compared to pluviation in air. The dust stayed close to the surface of the deposited material. However the rain of the soil achieved was not uniform; the diffusser did not help the spreading of the material and a cone shape surface was formed in the collector because the funnel was not rotated. The fixed position of the funnel caused the material to be concentrated in the center, therefore the packing energy of the surrounding material was less than the impact energy since particles slid downward to its rest involving some friction.

Densities obtained by pluviation through vacuum are shown in Fig. 4.2., Fig. 4.3., and Fig. 4.4. There was a small increase in density of samples pluviated through vacuum compared to those pluviated in air, but the increase in density was not proportional to the increase in impact of energy, which for an average 20 in. of drop heights results in about 30 times larger energy. Similar to pluviation in air, the increase in drop height through vacuum did not vary the density much. This fact agreed with conclusion drawn by Vaid et al. (1984), i.e. there is certain maximum impact energy, beyond which the sand do not compact any further.
4.3. COMPACTION METHOD

Investigation of sample fabrication by moist compaction was not intended to be applied in the calibration chamber due to the amount of time and labor required. This method was primarily investigated for triaxial specimen fabrication. The main purpose was to give information on the possible range of densities which could easily be obtained to be used for developing the steady state line for undrained monotonic and cyclic response of silty sand.

Compaction can be considered as an alternative in laboratory testing, specially if a wide range of density is desired. Variation of density can be obtained by varying the water content, the number of layers, number of tamps per layers and the tamping force. In this study, the Harvard compaction apparatus has been chosen due to its popularity in geotechnical engineering laboratory practice. The procedure and description of the equipment are discussed in detail by Wilson (1963), but is briefly reviewed in this report.

The apparatus consists of a cylindrical mold of 2.8 in. high and 1.31 in. diameter with a detachable collar. A metal tamper consisting of 0.5 in. diameter cylinder enclosed in handle with compressed spring preset to 20 lbs. A collar remover and sample ejector are provided to remove the sample from the compaction mold.

The soil was prepared by mixing to the desired water content. In these tests, 7 samples of different moisture content of approximately 2, 4, 6, 8, 10, 12 and 14 % were prepared. A value of dry density was assumed to estimate the required amount of soil to form five equal layers. The number of tamps was chosen to be 25 tamps per layer.

After compaction of the last lift, the collar was removed and the upper part of the sample was trimmed and the sample was extruded. The sample was then weighed and put in oven to
determine its water content. Plots of relationship of the dry density or void ratio and water content could then be established.

Results of compaction tests are presented in Figures 4.7. and 4.8, where for the chosen compactive effort, the optimum water content was found about 12.5% with the range of dry density from 105-120 pcf. This range of density is representative of those normally found in nature for this type of material and acceptable for laboratory investigations. To obtain samples with densities beyond these limits, one could apply a lower or higher compactive effort either by varying number of tamps per layer or changing the spring force. Table 4.2 gives comparison of densities obtained by different methods of fabrication where it is shown that the density range of the compacted material is more reasonable than those of pluviated samples. Fig. 4.4. shows the range of void ratios of pluviated samples of Yatesville silty sand and two other standard clean sands. The results of compaction tests are shown in the lower portion of the figure beyond the void ratios of the pluviated samples.

4.4 CONSOLIDATION OF SILTY SAND FROM A SLURRY

In nature, alluvial soils are deposited and consolidated over a period of years. This natural process can be simulated in the laboratory by consolidating the silty sand from a slurry. Such a process of consolidation is suitable for liquefaction studies since many reports have presented evidence that the liquefiable soils are recently deposited alluvial materials. Sample can be fabricated very close to full saturation with this method. Disadvantages of the process derive from the fact that it can not be automated like pluviator, and time is required for the consolidation process.
Fig. 4.7. RESULTS OF COMPACTION TESTS ON YATESVILLE SILTY SAND
FIG. 4.8. VOID RATIO OF THE COMPACTED MATERIALS

WATER CONTENT (%)
Nacci et al. (1976) successfully applied this method to fabricate layered silt specimens by artificially sedimenting the silt slurry and consolidating it for use in laboratory tests. The resulting specimens, according to their study, had similar structure, properties and similar water content as the original in situ deposit.

A batch consolidometer was used for this study. The slurry was prepared by mixing the material with water and stirred until uniform. Three random samples were taken from the silty sand slurry to determine the initial water content and the data showed that the slurry was sufficiently uniform. The slurry was then spooned into the mold. Pneumatic pressure was applied from the bottom of the mold with a piston with the desired pressure selected using a pressure regulator. The displacement was measured over time to determine when the sample achieved 100% consolidation.

In this study, two samples with different initial moisture contents were consolidated under 25 psi and 40 psi. The slurry had water content of 21.7% and 23.8% respectively. The specimen condition and the consolidation data are shown in Fig. 4.9. and Fig. 4.10. These figures show that the change in void ratio, $\Delta e$, was higher for the higher consolidation pressure, however the final void ratio were slightly lower in the 25 psi consolidation pressure. This was probably due to different initial water content of slurry. The void ratios obtained were 0.438 for the 25 psi consolidation pressure and 0.446 for the 40 psi consolidation pressure. This range of void ratios falls within those obtained by compaction, and it is acceptable for laboratory tests as well as tests in the calibration chamber. No further effort was extended for this method.

The consolidation data was very useful since the method was eventually selected for sample fabrication in the calibration chamber. The time required to complete consolidation was slightly faster for the 40 psi consolidation pressure (Fig. 4.9. and Fig. 4.10.). Typical range of time for consolidation in the batch consolidometer was 40 - 60 minutes in the range of the applied pressure (40 - 25 psi). Based on this data, estimated time to complete consolidation
Fig. 4.9. RESULTS OF CONSOLIDATION TEST ON SLURRY

Specimen condition

- Slurry w = 23.8 %
- Water w = 16.7 %
- Initial height H_slurry = 4.735 in.
- Final height H_f = 4.170 in.
- Initial compaction force F_slurry = 0.635 in
- Final compaction force F_f = 0.4459
- Settlement Δe = 0.1896

Consolidation data

σ_c = 40 psi

- t_100 = 38 min.
- t_50 = 7.4 min.
- C_v = 0.1319 in^2/min
Fig. 4.10. RESULTS OF CONSOLIDATION TEST ON SLURRY

Consolidation data

\[ \sigma_c = 25 \text{ psi} \]
\[ t_{100} = 61 \text{ min.} \]
\[ t_{50} = 13 \text{ min} \]
\[ C_v = 0.0739 \text{ in}^2/\text{min} \]

Specimen condition

\[ w_{\text{slurry}} = 21.7\% \]
\[ w_f = 16.4\% \]
\[ H_{\text{slurry}} = 4.650 \text{ in.} \]
\[ H_f = 4.185 \text{ in.} \]
\[ e_{\text{slurry}} = 0.5794 \]
\[ e_f = 0.4379 \]
\[ \Delta e = 0.1415 \]
of slurry in 5' high by 5' diameter sample of the same material will require about one to two weeks.

4.5. SUMMARY

1.) Pluviation through a diffusser sieve system to form a sand rain was not possible because the silt particles clogged the sieves causing uneven soil distribution.

2.) Pluviation by dropping silty sand from a simple inverted flask proved workable from the standpoint of literally getting the soil into a mold. However, the sample densities obtained were very low, particle segregation occurred and the process is not adaptable for the calibration chamber.

3.) In flask pluviation, there is a deposition rate associated with the maximum void ratio obtained, beyond with, a looser sample is not possible. For the Yatesville silty sand, this limit is 5 g/sec.

4.) Pluviation through vacuum helps to prevent segregation and increases the energy of the falling particles. However, the densities obtained were again very low and did not vary by changes in the height of fall. This shows that there is a certain limit of impact energy beyond which the material can not be more compact. It is concluded that even if the impact energy is increased substantially by increasing the drop height and thus the impact velocity, the silty sand will not densify any further.

5.) Due to the low sample density, segregation and the potential to volume change during saturation, pluviation of silty sand is not recommended as technique of sample fabrication.
6.) The compaction technique which is relatively easy to be carried out in small scale, proved a good procedure to create a test specimen. However, the method is not feasible for sample fabrication for the calibration chamber, and it would also be difficult to saturate the sample.

7.) A summary of the range of void ratios of the samples obtained by different methods of fabrication is shown in Tables 4.2. and Table 4.3. list the advantages and disadvantages of the sample fabrication. Consolidation from a slurry was found to be the best method to form a sample of silty sand. This method approximates the natural formation process and lead to a sample with a high degree of saturation. Densities of the soil can be varied over a wide ranges and sample structure is similar to that of the silty sand in the field. The only demerits with this method is that consolidating a large calibration chamber sample take a considerable amount of time, and the process can not be automated.
Table 4.2. SUMMARY OF RANGE OF VOID RATIO OF SILTY SAND SAMPLES OBTAINED BY DIFFERENT METHODS OF FABRICATION

<table>
<thead>
<tr>
<th>METHOD</th>
<th>VOID RATIO</th>
<th>REMARK</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLUVIATION</td>
<td>1.2 - 0.8</td>
<td>- effect of drop height or velocity at impact not significant</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- density variation is more dependent to deposition rate</td>
</tr>
<tr>
<td>COMPACATION</td>
<td>0.58 - 0.35</td>
<td>- use Harvard Compaction Equipment</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- only water content are varied</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- sample is devided in 5 layers with 25 tamps per layer</td>
</tr>
<tr>
<td>CONSOLIDATION</td>
<td>0.45 - 0.44</td>
<td>- only two tests, each with 40 psi and 25 psi consolidation pressure</td>
</tr>
</tbody>
</table>
Table 4.3. SUMMARY OF ADVANTAGES AND DISADVANTAGES OF DIFFERENT METHODS OF SAMPLE FABRICATION OF SILTY SAND

<table>
<thead>
<tr>
<th>METHOD</th>
<th>ADVANTAGES</th>
<th>DISADVANTAGES</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLUVIATION</td>
<td>- simple operation</td>
<td>- segregation</td>
</tr>
<tr>
<td></td>
<td>- fast</td>
<td>- very low densities</td>
</tr>
<tr>
<td></td>
<td>- applicable to both small and large samples</td>
<td>- difficult to saturate due to volume collapse</td>
</tr>
<tr>
<td>COMPACTION</td>
<td>- can obtain large range of densities</td>
<td>- difficult operation for calibration chamber</td>
</tr>
<tr>
<td></td>
<td>- relatively fast (for lab specimen)</td>
<td>- saturation problem for calibration chamber</td>
</tr>
<tr>
<td>CONSOLIDATION</td>
<td>- similar to natural alluvial formation process</td>
<td>- time required for consolidation</td>
</tr>
<tr>
<td></td>
<td>- applicable to both small and large samples</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- can obtain reasonable densities</td>
<td></td>
</tr>
</tbody>
</table>
5.1. INTRODUCTION

This chapter presents the results of laboratory tests investigating the undrained behavior of silty sand under monotonic and cyclic loading. For a better understanding of the tests and analyses to follow, a brief review of some aspects of the behavior is presented.

As discussed in chapter 2, the behavior of silty sand under monotonic loading for very large strain can be explained in terms of the steady state concept. Casagrande and his co-workers have used the SSL to distinguish between dilative soils (located on the left side of SSL) from contractive soil (plotted on the right side of SSL). However, no quantification was made on the proximity of the state of the soil to the SSL. Been and Jefferies (1985) were probably the first to quantify this proximity with the state parameter, \( \psi \).

The state parameter serves to relate density, stress level and volumetric behavior. For silty sands, there is not a standard method for determining the relative density, and the state pa-
parameter offers a useful means to characterize the behavior in the laboratory as well as in the field through a correlation to cone penetration resistance.

5.2. GENERAL STRESS-STRAIN BEHAVIOR OF COHESIONLESS SOIL


The stress strain behavior of cohesionless soils under monotonic loading has been a subject of discussion in textbooks, conference proceedings, and journals of geotechnical interest. This review does not attempt to go into detail of all these aspects but only the behavior related to the steady state theories.

In undrained loading, saturated granular materials ultimately reach a steady state characterised by deformation at constant volume, effective stresses, and velocity (Poulos et al., 1981; Castro et al., 1982). The friction angle mobilised at steady state has been shown to be constant for a given sand independent of initial void ratio. The stress path will terminate at the steady state envelope, thus the envelope in p'-q space is unique and characteristic of the sand.

Analogous with drained shearing of loose granular materials where volume decreases all the way to steady state, undrained shearing of loose saturated material results in pore pressure increase to steady state, where the effective stress remains constant thereafter. Thus, a constant volume condition at critical state in drained shear has for its counterpart a constant effective stress condition at steady state in undrained shear. Several investigators (Castro, 1969; Dobry et al., 1984; Sladen et al., 1985; Negussey et al., 1988 and others) have shown this aspect of undrained behavior at steady state.
A description of these characteristics is presented in Fig. 5.1. The state of soil is said to be in contractive condition if the plot in void ratio-mean effective stress space falls to the right side of the steady state line (SSL). Thus, paths 1 and 3 are in a contractive state. At the initial state 3, the soil is said to be contractive because it would decrease in volume as it is sheared and then reach a constant volume upon approaching steady state. On the other hand, if it is sheared undrained (path 1), the tendency for volume decrease (hence called contractive) would take place and the pore pressure will increase all the way until steady state. This increase in pore pressure is generally accompanied with drop in shear strength. For a dilative material (path no. 2), undrained shearing causes initially a pore pressure increase, but the approach to steady state thereafter occurs through the dilation tendency and pore pressure decrease.

In all cases (paths 1, 2 and 3) after sufficient straining, the state of the soil reaches a point on the SSL regardless of the stress path or initial condition. The shape of the stress strain curve prior to reaching steady state is a function of the initial structure of the soil; however, its steady state position is not, since at steady state the soil is thoroughly remolded and has lost all memory of its initial structure (Castro, 1987). The undrained steady state strength, designated as \( S_u \), is only a function of void ratio of the sand and not of its initial state of stress, the type of undrained loading (monotonic or cyclic), or its initial fabric (Castro, 1969).

**5.2.2. Behavior of Cohesionless Soil under Cyclic Loading**

The cases discussed for Fig. 5.1. show undrained loading that consists of a monotonic increase in shear stress. However, in the case of dynamic soil behavior, the sample is loaded cyclically from an initial static effective stress. Since the undrained steady state strength of the sand, \( S_u \), is only a function of its void ratio, the same value of \( S_u \) applies whether the soil is loaded monotonically or cyclically (Castro, 1975; Castro et al., 1982; and Castro, 1987).
Fig. 5.1. BEHAVIOR OF SANDS UNDER MONOTONIC LOADING

Initial State
Steady State

Undrained Contractive
Undrained Dilative

Phase Transformation
Steady State

Mean Effective Stress, \( p \)
Void Ratio, \( e \)
Shear Stress, \( \tau \)
Strain

Pore Pressure
This phenomena was investigated by Vaid and Chern (1985) as illustrated in Fig. 5.2. A distinction was made for two different types of response of saturated sand subjected to cyclic loading. Fig. 5.2.a. shows the case of loose or contractive soils. At some stage in cyclic loading, large deformations result from strain softening behavior associated with a loss of shear resistance. In the extreme case, the sand may lose a large portion of its shear resistance and deform continuously in a state of constant stress called steady state. In the second type of response (Fig. 5.2.b.), the deformation is the result of progressive stiffness degradation of the sand due to a build up pore pressure with cycles of loading. Sufficient cycles of such shear stresses can momentarily reduce the effective stresses to zero. Such phenomena is termed cyclic mobility (Casagrande, 1975; Castro, 1977) and unlike liquefaction failure, cyclic mobility causes accumulation of deformations which are limited in magnitude.

5.3. CONSOLIDATED UNDRAINED TESTS ON SILTY SAND

For monotonic loading, the samples of Yatesville silty sand and Pepper’s Ferry silty sand were isotropically consolidated before shearing. Tests were carried out to obtain the steady state line of each material. Eight isotropically consolidated triaxial tests were performed for Yatesville silty sand and seven tests were conducted on Pepper’s Ferry silty sand.

To simplify discussion, the numbering of the tests was chosen as follows; (1) The material is identified by the first letter where Y stand for Yatesville and P stand for Pepper’s Ferry; (2) Since the samples were prepared by compaction and by consolidation from slurry, COM or SL were appended to the test number after the material name and (3) the magnitude of the initial effective stress under which the samples were consolidated before shearing is assigned at the end. Thus, test on a sample of Yatesville silty sand prepared by compaction and consolidated to 30 psi before shearing is called test No. YCOM-30.
Fig. 5.2. BEHAVIOR OF SANDS UNDER CYCLIC LOADING (Vaid & Chern, 1985).
Initial studies of steady state behavior of sand were performed using stress controlled consolidated undrained triaxial tests (Castro, 1969; Castro et al., 1982). Some strain-controlled tests were also performed and shown to produce good agreement with the stress controlled tests. This has been further confirmed to limited extent by other investigators (Hungr and Morgenstern, 1984; Dobry et al., 1984; and Sladen et al., 1985). Use of strain controlled triaxial testing is now suggested (Castro, 1987; GEI, 1988), and accordingly, the triaxial tests conducted in this study were sheared at a constant rate of strain.

The triaxial tests were conducted on samples consolidated from slurry and compacted using the Harvard apparatus to investigate the influence of initial fabric on the test results. Previous research has shown that the position of the SSL for a certain soil is independent of the sample fabrication technique (Castro et al., 1982; GEI, 1988). However, the initial fabric of triaxial specimens has been shown to have a great influence on the sand behavior in undrained triaxial tests (Ladd, 1974; Townsend, 1978).

As an initial step in the triaxial testing procedure, the pore pressure board, pore pressure lines, triaxial cell, and porous stones were flushed with CO₂ under a low pressure. Since CO₂ is much more soluble in water than air, this technique has shown to reduce the back pressure and time required for saturation of a triaxial sample. After the specimen was placed in the triaxial cell, CO₂ was percolated through the sample prior to percolation with deaired distilled water. After saturation and consolidation, the samples were sheared at a slow enough rate such that the time to failure was greater than 7x t₅₀. In all tests, a B, Δu/Δσ_{cell}, value of 0.98 or higher was achieved prior to shearing.

At the end of the consolidation, the void ratio, eₐ, was less than the initial void ratio eₒ, and adjustment was therefore made for this to use in the analysis. Since the sample was consolidated isotropically, adjustment of the final dimension and the void ratio can be done using the expression as follows:
where \( H_c = \) height of sample after consolidation, \( H_o = \) initial height of the sample and \( \Delta V \) is the volume change of the sample. For practical purposes, the volume change of the sample was taken equal to the volume of water displaced during consolidation. Thereafter, strain was calculated referring to this height, \( H_c \) and similarly the void ratio after consolidation can be calculated:

\[
e_c = e_o - (1 + e_o) \frac{\Delta V}{V_o} \quad \text{[eq.5.2.]}\]

\( e_o \) and \( e_c \) were the corresponding void ratio initially and at the end of consolidation.

At this stage, it is possible to measure the compressibility of the soil. To distinguish the compressibility as obtained in oedometer test, the volumetric compressibility of the isotropically consolidated soil was designated with symbol \( m_{vi} \), that can be expressed as follows:

\[
m_{vi} = - \frac{\Delta V/V}{\Delta \sigma'} \quad \text{or} \quad m_{vi} = \frac{\Delta e}{\Delta \sigma'} \frac{1}{1 + e_o} \quad \text{[eq.5.3]}\]

where \( \Delta \sigma' \) is the change in effective stress and \( \Delta e \) is the change in void ratio calculated previously.

Table A.2.1 and Table A.2.2 show the values of \( m_{vi} \) and \( t_s9 \) for Yatesville and Pepper’s Ferry Silty Sand. The volumetric compressibility for both silty sands was very close, on the order of 4 to 10 \( \times \) 10^{-4} (psi^{-1}), typical values for these types of material. The time required for consolidation was relatively short due to the high permeability of the silty sand and the small size of the sample. It is also interesting to note that in Yatesville silty sand, the time for consolidation is substantially faster for the compacted sample compared to the sample consolidated.
from slurry. The rate is as high as ten times as for the compacted samples. This tendency is also true for Pepper’s Ferry silty sand except that the difference is not as significant.

As a triaxial sample shears, the cross-sectional area increases while the sample compresses. In conventional triaxial testing, an area correction is performed assuming that the sample compresses as a right circular cylinder. The following expression is generally applied for area correction:

\[ A = \frac{A_c}{1 - \epsilon} \]  
[eq.5.4.]

where \( A_c \) is the initial cross sectional area (after consolidation) and \( \epsilon \) is the corresponding axial strain. While this correction may be appropriate for calculating peak normal stresses associated with small strains, it may not be applicable to samples at the large strains used for steady state strength determination.

In some of the tests, the samples were sheared as far as 30% strain or more. The triaxial sample condition at large strains is shown in Fig. 5.3. At large strain, the barelling effect correction results in higher corrected area, however when samples experience slipping along a plane, this correction is inappropriate since the cross sectional area of the sample is actually decreasing. Description of this phenomena and the appropriate correction factor can be found in Head (1985).

Fig. 5.4. shows an example of the significance of applying the barelling and the slipping correction for Test No. YSL-20. Up to 10% of strain, only the barelling correction is applied before slippage was observed. Curve 1 shows the ratio of the corrected area to the initial area of the sample when only barrelling occurs. Curve 3 is the correction factor if only slippage is assumed without further barelling and curve 2 is the corresponding factor for slipping and barelling to occur simultaneously after the point of slippage. From this figure it is shown that the difference of the correction factor of curve 1 to curve 2 is almost 20% and from curve 1
Test No. YSL-20

Fig. 5.4. AREA CORRECTION
to curve 3 is almost 30% at the end of the test. The difference would even be more significant if the strain at the end of the test is higher (30 to 40 %). From observations made during the tests, it appears that after slippage occurs, barelling is still occurring and curve 2 is the more appropriate correction factor to be used in the data reduction for test YSL 20. Correspondingly for the other tests, the failure modes were observed and the correction factor was applied similar to those of Fig. 5.4. using either curve 1 or 2.

5.4. DISCUSSION AND INTERPRETATION OF TEST RESULTS

Test results for Yatesville silty sand and Pepper's Ferry silty sand are shown in Table 5.1. and Table 5.2. The data are characterized in terms of state parameters and the peak friction angle. Similar analysis of silty sands from the Lower San Fernando Dam and two clean sands, Banding #6 and Sacramento River sands are included in Tables A.3 - A.6 in the appendix. The silty sands from the Lower San Fernando Dam were selected because of their fines contents. Batch Mix No. 3 has 8% fines content and Batch Mix No. 7 has 48% fines content, and because the soils were tested thoroughly as a result of their involvement in the liquefaction failure of the Lower San Fernando Dam in the San Fernando Earthquake of 1971. The two clean sands were chosen as a basis for comparison to the silty sands, and because enough data were available to get steady state parameters. Data for the Lower San Fernando Dam soils and the Sacramento River sand were reported by Hsing and Seed (1988) and Banding #6 sand by Castro et al. (1982). The grain size characteristics and the steady state parameter for each sand are given in Table 5.3. along with those for Yatesville and Pepper's Ferry soils.
### TABLE 5.1. RESULTS OF ICU TRIAXIAL TESTS OF YATESVILLE SILTY SAND

<table>
<thead>
<tr>
<th>Test</th>
<th>$e_c$</th>
<th>$\sigma_m^*$ (psi)</th>
<th>$\sigma_3^*$ (psi)</th>
<th>$S_{us}$ (psi)</th>
<th>$p_{ss}$ (psi)</th>
<th>$q_{ss}$ (psi)</th>
<th>$\phi_s$ (deg)</th>
<th>$\phi_{peak}$ (deg)</th>
<th>state $\psi$</th>
<th>Behavior</th>
</tr>
</thead>
<tbody>
<tr>
<td>YCOM-10</td>
<td>0.404</td>
<td>10.0</td>
<td>18.8</td>
<td>18.5</td>
<td>40.8</td>
<td>22.0</td>
<td>32.6</td>
<td>38.5</td>
<td>-0.087</td>
<td>D</td>
</tr>
<tr>
<td>YCOM-15</td>
<td>0.540</td>
<td>15.0</td>
<td>2.8</td>
<td>2.9</td>
<td>6.3</td>
<td>3.5</td>
<td>33.6</td>
<td>33.7</td>
<td>0.079</td>
<td>A</td>
</tr>
<tr>
<td>YCOM-30</td>
<td>0.429</td>
<td>30.0</td>
<td>14.3</td>
<td>14.2</td>
<td>30.9</td>
<td>16.9</td>
<td>33.2</td>
<td>35.7</td>
<td>0.018</td>
<td>A</td>
</tr>
<tr>
<td>YCOM-30A</td>
<td>0.449</td>
<td>30.0</td>
<td>9.6</td>
<td>9.3</td>
<td>20.6</td>
<td>11.0</td>
<td>32.2</td>
<td>33.7</td>
<td>0.038</td>
<td>A</td>
</tr>
<tr>
<td>YSL-10</td>
<td>0.460</td>
<td>10.0</td>
<td>7.6</td>
<td>7.6</td>
<td>16.7</td>
<td>8.9</td>
<td>32.3</td>
<td>35.1</td>
<td>-0.031</td>
<td>C</td>
</tr>
<tr>
<td>YSL-20</td>
<td>0.419</td>
<td>20.0</td>
<td>14.3</td>
<td>15.5</td>
<td>33.1</td>
<td>18.8</td>
<td>34.6</td>
<td>36.0</td>
<td>-0.021</td>
<td>D</td>
</tr>
<tr>
<td>YSL-30</td>
<td>0.435</td>
<td>30.0</td>
<td>12.4</td>
<td>11.9</td>
<td>26.6</td>
<td>14.0</td>
<td>31.8</td>
<td>34.4</td>
<td>0.024</td>
<td>D</td>
</tr>
<tr>
<td>YSL-40</td>
<td>0.387</td>
<td>40.0</td>
<td>23.5</td>
<td>23.4</td>
<td>50.4</td>
<td>28.2</td>
<td>34.0</td>
<td>35.1</td>
<td>-0.003</td>
<td>D</td>
</tr>
</tbody>
</table>

**LEGEND:**
- A: Contractive behavior with a peak shear stress prior to steady state
- B: Contractive behavior with no peak shear stress prior to steady state
- C: Dilative behavior with a peak shear stress prior to steady state
- D: Dilative behavior with no peak shear stress prior to steady state
### TABLE 5.2. RESULTS OF ICU TRIAXIAL TESTS OF PEPPER'S FERRY SILTY SAND

<table>
<thead>
<tr>
<th>Test #</th>
<th>$\phi_c$ (deg)</th>
<th>$\sigma_{m'}$ (psi)</th>
<th>$\sigma_{3r'}$ (psi)</th>
<th>$S_{us}$ (psi)</th>
<th>$P_{ss}$ (psi)</th>
<th>$q_{ss}$ (psi)</th>
<th>$\phi_s$ (deg)</th>
<th>$\phi_{post}$ (deg)</th>
<th>state $\psi$</th>
<th>Behavior</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCOM-40</td>
<td>0.577</td>
<td>40.0</td>
<td>39.6</td>
<td>39.2</td>
<td>86.2</td>
<td>46.6</td>
<td>32.7</td>
<td>36.0</td>
<td>-0.033</td>
<td>D</td>
</tr>
<tr>
<td>PCOM-10</td>
<td>0.629</td>
<td>10.0</td>
<td>13.7</td>
<td>13.1</td>
<td>29.3</td>
<td>15.5</td>
<td>32.0</td>
<td>34.0</td>
<td>-0.062</td>
<td>D</td>
</tr>
<tr>
<td>PCOM-20</td>
<td>0.639</td>
<td>20.0</td>
<td>13.9</td>
<td>13.2</td>
<td>29.5</td>
<td>15.6</td>
<td>21.9</td>
<td>21.1</td>
<td>-0.011</td>
<td>D</td>
</tr>
<tr>
<td>PCOM-35</td>
<td>0.580</td>
<td>35.0</td>
<td>36.8</td>
<td>36.7</td>
<td>80.4</td>
<td>43.6</td>
<td>32.8</td>
<td>36.1</td>
<td>-0.037</td>
<td>C</td>
</tr>
<tr>
<td>PCOM-30</td>
<td>0.585</td>
<td>30.0</td>
<td>34.3</td>
<td>32.9</td>
<td>73.1</td>
<td>38.8</td>
<td>32.0</td>
<td>36.2</td>
<td>-0.041</td>
<td>C</td>
</tr>
<tr>
<td>PSL-30</td>
<td>0.584</td>
<td>30.0</td>
<td>31.6</td>
<td>34.8</td>
<td>74.1</td>
<td>42.5</td>
<td>35.0</td>
<td>35.2</td>
<td>-0.042</td>
<td>D</td>
</tr>
<tr>
<td>PSL-10</td>
<td>0.574</td>
<td>10.0</td>
<td>38.5</td>
<td>36.9</td>
<td>82.1</td>
<td>43.5</td>
<td>32.1</td>
<td>35.6</td>
<td>-0.116</td>
<td>D</td>
</tr>
</tbody>
</table>

#### LEGEND:
- **A**: Contractive behavior with a peak shear stress prior to steady state
- **B**: Contractive behavior with no peak shear stress prior to steady state
- **C**: Dilative behavior with a peak shear stress prior to steady state
- **D**: Dilative behavior with no peak shear stress prior to steady state
5.4.1. The Steady State Line (SSL)

The steady state line is usually a straight line or has a slight downward concave shape when plotted with the void ratio on an arithmetic scale (Y-axis) and the effective normal stress in a logarithmic scale (x-axis). The slope of the line is defined as the change in void ratio for a tenfold change in effective stress or steady state shear strength (one log cycle). The slopes are generally expressed as:

\[ C_{ss} = -\frac{\Delta e}{\Delta (\log \sigma_{3r}')} \]  \[ eq. 5.5.a \]

or when plotted in terms of Steady State Shear Strength,

\[ C_{ss} = -\frac{\Delta e}{\Delta (\log S_{us})} \]  \[ eq. 5.5.b \]

and alternatively when they are plotted in terms of the mean normal effective stress, \( \sigma_{m}' \)

\[ \lambda_{ss} = -\frac{\Delta e}{\Delta (\log \sigma_{m}')} \]  \[ eq. 5.5.c \]

Fig. 5.5. gives the SSL for Yatesville silty sand in terms of void ratio plotted against the minor principal effective stress developed at the steady state. The diagram also shows the initial condition and the path followed by the sample to the SSL. The data show a consistent pattern with samples initially loose of SSL undergoing a decrease in effective confining stress during shear and those initially dense of the SSL show the opposite behavior. The final conditions of each test defines a straight line with almost no scatter in the data. Notably samples prepared by either compaction or consolidation from a slurry give the same SSL. This follows the conclusion of Castro et al. (1982) that initial fabric has no influence on SSL. Figures 5.6. and 5.7. give the SSL for Yatesville silty sand in terms of \( S_{us} \) and effective mean normal stress,
Fig. 5.5. SSL FOR YATESVILLE SILTY SAND IN TERMS OF MINOR PRINCIPAL EFFECTIVE STRESS
C_{st} = -0.168

YATESVILLE

- COMPACTED
- SLURRY

Fig. 5.6. SSL FOR YATESVILLE SILTY SAND IN TERMS OF $S_{us}$
Fig. 5.7. SSL FOR YATESVILLE SILTY SAND IN TERMS OF MEAN EFFECTIVE STRESS

\[ \lambda_{se} = -0.168 \]
Figures 5.8., 5.9., and 5.10. give the SSL for Pepper’s Ferry silty sand in terms of residual effective stress, $S_{m'}$, and effective mean normal stress, $\sigma_m'$ respectively. As with the Yatesville results, the data define the SSL with little scatter, and the $C_{ss}$ and $\lambda_{ss}$ values are similar at a value of -0.134. Again there is no influence of initial sample fabric.

In Figure 5.11. the SSL for the silty sands of this investigation are compared to SSL for the Banding #6 and Sacramento River clean sands and the silty sands from the Lower San Fernando Dam. All of the soils but the Sacramento River Sands show straight SSL. The SSL of Sacramento River sand gives a relatively flat slope (-0.02) to a mean normal effective stress of about 20 psi and thereafter assumes a steeper slope (-0.17).

In general, finer grained soils tend to be more compressible than coarse grained soils, and hence SSL for a fine grained soil would typically be steeper than that of a coarse grained soil. This type of behavior has been reported in experimental findings by Been and Jeffries (1985) and Castro et al. (1982). However, evidence to the contrary was published by Mohammad and Dobry (1984). In Fig. 5.11. the soils with the largest degree of fines (Yatesville, Pepper’s Ferry and Lower San Fernando Dam Batch Mix No. 7) exhibit steeper SSL’s than the other sands with the exception of the steeply sloping second part of the Sacramento River sand SSL.

5.4.2. The Envelope and the Mobilized Friction Angle at Steady State

Steady state envelopes for both Yatesville Silty Sand and Pepper’s Ferry silty sand confirm that these envelopes are unique regardless of the initial density (Fig. 5.12 and Fig. 5.13). Data for other silty sands from San Fernando Dam (Fig. A.21 and Fig. A.22.) and clean sands (Fig.
Fig. 5.8. SSL FOR PEPPER'S FERRY SILTY SAND IN TERMS OF MINOR PRINCIPAL EFFECTIVE STRESS
Fig. 5.9. SSL for Pepper's Ferry Silty Sand in terms of $S_{us}$. 

$C_{er} = -0.134$
FIG. 5.10. SSL FOR PEPPER'S FERRY SILTY SAND IN TERMS OF MEAN EFFECTIVE STRESS

- PEPPERS FERRY
- COMPACTED SLURRY

![Graph showing the relationship between void ratio (e) and mean effective stress (σm).](image-url)
Fig. 5.11. SSL FOR SIX SANDS
Fig. 5.12. STEADY STATE ENVELOPE FOR YATESVILLE SILTY SAND
A.23 and A.24) also support this. The slopes are related directly to the mobilized friction angle at steady state, $\phi_s$, in the following expression:

$$\sin \phi_s = \tan \alpha_{ss} \quad [eq. 5.6.]$$

Thus, it can be inferred that for a given sand, the mobilized friction angle at steady state is also constant and not a function of the initial void ratio, nor the initial or final effective stress. This friction angle is inherent and specific for the particular sand tested. Table 5.3. shows the angle $\phi_s$ for all clean sands and silty sands investigated. All of the $\phi_s$ values are between 32° and 35.5° except for Banding #6 sand which gave a $\phi_s$ of 26.4°.

The undrained steady state shear strength can be related to the residual effective minor principal stress as follows (Castro and Poulos, 1985):

$$S_{us} = \sigma_3' \cdot \frac{\sin \phi_s \cdot \cos \phi_s}{(1 - \sin \phi_s)} \quad [eq. 5.7.]$$

in which $\phi_s$ is friction angle at steady state.

Equation 5.7. suggests that the undrained steady state shear strength is related to the effective minor principal stress by the value of $\phi_s$ which is essentially constant over a range of stresses. Fig. 5.14. shows a plot of $S_{us}$ vs $\sigma_3'$ for silty sands with lines showing the corresponding values of $\phi_s$. It can be seen that the data follow straight lines as suggested by equation 5.7. The values of $\phi_s$ of Yatesville, Pepper’s Ferry and Lower San Fernando Dam Batch Mix no. 7 fall in the range of 31 to 35 degrees with an average of 33 degrees. It is interesting to note that when $\phi_s = 33$ degrees, equation 5.7. predicts a value of $S_{us}$ equals to $\sigma_3'$. Except for Batchmix No. 3. of San Fernando Dam, other silty sands have $S_{us}$ equal to the residual minor effective stress $\sigma_3'$. A similar plot for clean sands (Fig. 5.15) shows that Banding #6 sand has a $\phi_s$ lower than silty sand. However, Sacramento River sand has behavior similar to the silty sands.

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Fig. 5.14. CORRELATIONS OF $\sigma_v$ and $S_u$ FOR SILTY SANDS
Fig. 5.15. CORRELATIONS OF $\sigma_3$, and $S_{us}$ FOR CLEAN SANDS
In Figure 5.16, the tan $\alpha_{ss}$ value is plotted against void ratio of the soil for all six sands described to this point. All data points tend to indicate that tan $\alpha_{ss}$ is not dependent on void ratio. The data suggest that silty sands have average values of tan $\alpha_{ss}$ equal to 0.55. The data for Banding sand is more scattered and the values of tan $\alpha_{ss}$ are lower than those of silty sands and the Sacramento River Sand.

### 5.4.3. Mobilized Friction Angle at Phase of Transformation

The point of maximum pore pressure is said to correspond to the *phase of transformation state* (Ishihara, 1975). Vaid and Chern (1985) have shown that the friction angles mobilized at phase transformation and steady state of contractive sand are identical and are unique property of a given sand. Plot of Yatesville and Peppers Ferry Silty Sand steady state envelopes also show this agreement (Fig. 5.12. and 5.13.). In previous studies, similar behavior was also reported by Castro et al. (1982) but no suggestion as to the equality of $\phi_{pt}$ and $\phi_{ss}$ was made.

Luong (1980) has suggested that the friction angle mobilised at the point of maximum contraction in drained shear, $\phi_{mc}$, is identical to the friction angle mobilised at phase transformation in undrained shear. A close correspondence between mobilised friction angles at undrained steady state and drained critical state was suggested by Castro (1969). Test results by Negussey et al. (1988) indicated that friction angle at constant volume, $\phi_{cv}$, is a lower bound on shear resistance and is independent of particle size, confining pressure, and density. Also, Negussey et al. (1988) suggested that there is a close agreement between, $\phi_{cv}$ and mobilized friction angles at phase transformation and steady state. Thus the above findings suggest that

$$\phi_{mc} = \phi_{cv} = \phi_{pt} = \phi_s \quad [eq.5.8.]$$
Fig. 5.16. $\tan \alpha_{ss}$ vs. Void Ratio for Clean Sands and Silty Sands
5.4.4. Observation of the stress-strain behavior

For monotonic loading, Castro et al. (1982) suggested four distinct types of stress-strain curves observed from undrained triaxial tests. They reported that the type of stress-strain curves for particular test was found to be a function of the soil type and the position of the initial state relative to the steady state. It is therefore interesting to make a comparison of the difference of these curves with regard to sand and silty sand. The soils tested by Castro et al. (1982) are predominantly clean sands or sands with an insignificant amount of fines, except for a mine tailings material that has a fines content of approximately 6.5%.

The four different stress-strain behaviors of the soils were classified as follows: (A) contractive behavior with a peak shear stress prior to steady state deformation, (B) contractive behavior with no peak shear stress prior to steady state deformation, (C) dilative behavior with a peak shear stress prior to steady state deformation and (D) dilative behavior with no peak shear stress prior to steady state deformation.

The stress strain behaviors of Yatesville silty sand and Peppers Ferry silty sand are presented in Appendix A.1 - A.15. Each figure also shows the stress path corresponding to the state parameter. Test results on soil which initially has a contractive state (e.g. YCOM-15) shows that pore pressures rises immediately upon shearing to maximum values then it stays constant without any tendency to decrease. Steady state was achieved at moderately lower strains. On the other hand, test results on soil with an initial dilative state show that steady state conditions are achieved only at very large strains. Two samples of Pepper's Ferry silty sand at initially strong dilative state (PCOM-10 and PSL-10) did not reach steady state.

Stress-strain type (A) were observed in tests No. YCOM-15, YCOM-30 and YCOM-30A for Yatesville silty sand. In these tests, the pore pressure and shear stress both increased gradually as axial load was applied. The peak shear stress was typically reached at 1% to
2% of strain and then dropped slowly to reach the steady state strength at 8% to 10% strain. Even so, tests were continued until axial strains of 20% to 30%. At strains greater than 10%, the shear stress and the effective stress are essentially constant. Stress-strain type (B) was not observed in these test results except possibly test No. YCOM-15. The difference in this type of stress strain curve of silty sand as opposed to clean sand (such as those reported by Castro, 1969) is the magnitude of the drop in shear strength. In silty sand, the strain softening behavior after the peak is gentle, not as severe as a clean sand. This difference of behavior of clean sand and silty sand will be discussed in section 5.4.5.6.

The results of tests no. YSL-10, YSL-20, YSL-30, YSL-40 and YCOM-10 and almost all tests on Peppers Ferry silty sand (except PCOM-30 and PCOM-40) fall into category type (D) stress strain curve. In these tests, pore pressure rose only temporarily and then decreased while the shear stress kept increasing. The maximum pore pressure (or the phase of transformation) was reached at small strains, depending on the initial state of the samples, but never higher than 2%. Steady state deformation was achieved at much higher strains unlike type (A) stress strain behavior. Most of the tests conducted on Yatesville silty sand reached steady state about 15% strain or higher, but the Peppers Ferry silty sand did not reach steady state until 30% strain or more. This is due to the strongly dilative state of the samples before the tests. Only tests no. PCOM-30 and PCOM-40 showed typical type (C) stress strain behavior. As will be shown subsequently, the state parameter serves as a vehicle to quantify the relative responses of the different soil conditions.

5.4.5. Behavior of Silty Sand as a function of State Parameter

Although it is recognized that the anisotropic fabric of sand is not accounted for in the current application of state parameter, the state parameter has been successfully used to normalize large strain behavior where the influence of initial fabric is small (Been and Jefferies, 1985).
Based on this premise, the state parameter was applied to the behavior of the silty sands tested in this study and other soils including silty sands and clean sands.

To incorporate the state parameter, the SSL's in terms of mean effective stress, $\sigma_m'$, were reported for the four silty sands and the two clean sands being investigated in Fig. 5.11. For simplicity, the steady state lines of the soils were assumed straight lines. The lines can be represented as:

$$e_{ss} = \lambda_{ss} \log \sigma_m' + e_1 \quad [eq.5.9.]$$

where $e_{ss}$ indicates the void ratio at steady state corresponding to the mean effective stress $\sigma_m'$. $\lambda_{ss}$ is the slope of the line and $e_1$ is the void ratio at steady state corresponding to the mean effective stress equal to unity (depending on the units used). Table 5.3. gives the numerical values of $\lambda_{ss}$ and $e_1$ for all soils considered. Note that $e_1$ is not a common parameter and it serves only as a reference void ratio. At very low stresses, $e_1$ perhaps corresponds to the maximum void ratio.

5.4.5.1. Changes of State Parameter upon Shearing

Been and Jefferies (1985) have correlated certain aspect of soil behavior to state parameter. It should be noted however that the state of soils is continuously changing as it is sheared. The previous discussion of the state parameter refered only to the initial state. As the soil is sheared undrained, the change in effective stresses change the state parameter and at large strain it eventually contacts the SSL indicating that the soil has reach steady state condition. By definition, the state parameter at steady state is zero. In subsequent discussion, the term state parameter always refers to the initial state and the state parameter at various other stages will be attributed by an appropriate label. $\psi_{ss}$ is used for the state parameter at steady state condition and $\psi_{st}$ is used for the state parameter at phase of transformation, and thus it follows that $\psi_{ss} = 0$. 

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<table>
<thead>
<tr>
<th>SOIL</th>
<th>Gs</th>
<th>Fines (%)</th>
<th>D50 (mm)</th>
<th>λss</th>
<th>e₁</th>
<th>ϕₜₜ (deg)</th>
<th>References</th>
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<td>40</td>
<td>0.10</td>
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<td>0.659</td>
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<td>0.824</td>
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<td>8</td>
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<td>0.794</td>
<td>35.2</td>
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<tr>
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<td>0.737</td>
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<td>-0.019</td>
<td>0.864</td>
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</tr>
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<td>BANDING #6</td>
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<td>0</td>
<td>0.16</td>
<td>-0.039</td>
<td>0.814</td>
<td>26.4</td>
<td>Castro et al. (1982)</td>
</tr>
</tbody>
</table>
The changes of the state parameter during shear for all test data on Yatesville silty sand are shown in Fig. 5.17, where the state at every stage is plotted versus the strain. In most cases, samples that initially had a positive state \( (\psi > 0) \) stayed at a positive state until it reached steady state. The same tendency was also true for sample that had negative initial state parameter. An exception occurred in sample no. YSL-30. Although it initially had positive state \( (\psi = 0.024) \), pore pressure was generated upon shearing and the state moved to left of the SSL, thus, the state parameter at that stage was negative. Fig. 5.18 is a plot of the state at phase of transformation, \( \lambda_{st} \), versus the initial state, \( \psi \). The trend of the data suggests that a soil sample at an initially small positive state, roughly \( \psi = 0.03 \), can have a negative state at phase of transformation.

5.4.5.2. State Parameter and Stress Strain Behavior

The individual stress strain behavior of all CIU-tests on Yatesville and Pepper's Ferry silty sands can be observed in the appendix (Fig. A.1 to A.15). All of the samples of Peppers Ferry silty sand were in an initially negative state and dilatant stress strain behavior was pronounced. Tests on Yatesville silty sand have both dilative behavior and contractive behavior thus the influence of the state parameter on the stress strain behavior will be investigated from test results of this soil.

Stress paths were plotted normalising both axis, \( p' \) and \( q \), by a factor \( p_{st} \), the mean effective stress at steady state condition. Since all samples were sheared until they reached steady state or close to steady state, all of the stress path would end up at the mean effective stress equal to \( p_{st} \), consequently all stress path would end at a value of \( p'/p_{st} \) equal to unity. Similarly they would also end up at an ordinate \( q/p_{st} \) equal to \( \tan \alpha_{st} \). By plotting all the stress paths in this way, effect of the state parameter on the stress strain behavior could reasonably be observed.
Fig. 5.17. CHANGES OF STATE PARAMETER UPON SHEARING
Fig. 5.18. STATE PARAMETER AT PHASE OF TRANSFORMATION VS INITIAL STATE PARAMETER
As seen in Fig. 5.19, the shape of the normalized stress path is strongly influenced by the value of the state parameter, \( \psi \). As the values of the state parameter are more positive, the curves of the stress paths consistently move to the right. An exception to this behavior was observed for Test No. YSL-30, and as discussed previously, this sample had negative state at phase of transformation. Similar phenomena were also observed by Been and Jefferies (1985) but no quantification of the value of the state (such as Fig. 5.18.) was given. Test No. YCOM-30 with initial state parameter of + 0.018 (less positive than test No. YSL-30) still has a positive state at phase of transformation. Consistently, contractive behavior was reflected in its stress path. On the other hand, test No. YCOM-30A, initially having a state parameter of + 0.038, showed contractive behavior, consistent with its initial state, even though it had a negative state at the phase of transformation.

It is clear that both the initial state parameter, \( \psi \), and the state at phase of transformation, \( \psi_{pt} \), characterize the volume change tendencies of the soil. This conclusion allows one to construct a boundary in the SSL diagram where behavior of the soil could be dilative or contractive (Fig. 5.20). This boundary is a line parallel to the SSL offset distance of 0.03. This conclusion is also supported by the observed stress strain behavior of Batch Mix No.3 and Batch Mix No. 7 of the San Fernando Dam. Although the states at phase of transformation were not shown, the state parameters of the samples were calculated and presented in Table A.3. and Table A.4. in the appendix. The stress strain behavior is indicated by the letters A, B, C or D which were explained in section 5.4.4. It is shown on these tables that all samples with initially negative state parameter had a dilative stress strain behavior and samples with state parameters 0.03 or higher exhibited contractive behavior. However, samples with state parameter between 0.0 to 0.03 had either contractive or dilative behavior. Similar conclusions were also suggested by Poulos (1988), but the boundary was not numerically determined. A liquefaction analysis performed by Sladen (1987) indicated that a value of \( \psi = + 0.024 \) or higher was needed to show the strain softening behavior.
Fig. 5.19. NORMALISED STRESS PATH OF SILTY SAND WITH VARIOUS INITIAL STATE
Fig. 5.20. PROPOSED BOUNDARY OF DILATIVE-CONTRACTIVE BEHAVIOR
5.4.5.3. State Parameter and The Peak Friction Angles

According to Castro et al. (1982), the soil condition at steady state is not affected by its initial fabric or effective stress, because at large strains the soil has lost “its memory” of its original conditions. However, when soil sample is sheared, the condition at small strains is presumably influenced by its initial state which is best represented in state parameter, $\psi$. To quantify this, the peak friction angle, $\phi_{\text{peak}}$, and the friction angle at steady state, $\phi_{ss}$ of the Yatesville and Pepper’s Ferry silty sands are plotted against state parameter (Fig. 5.21.). The peak friction angle was defined corresponding to the maximum value of the ratio of the major effective principal stress to the minor effective principal stress, $(\sigma'_1/\sigma'_2)_{\max}$. Although scattered, the data suggest that the peak friction angle is influenced by the initial condition (void ratio, stress and fabric) while the steady state friction angle is not. At negative state, the peak friction angle is higher due to the dilation tendency. The peak friction angles of the compacted samples seem to be slightly higher than those of samples consolidated from a slurry. A line can be drawn to show the trend of the peak friction angle against the state parameter.

5.4.5.4. State Parameter and Normalized Steady State Strength

The values of $S_u$, for four silty sands tested in this study and data from published reports were normalized by the mean effective stress, $\sigma'_m$ and plotted versus the state parameter, $\psi$ in Fig. 5.22. This plot suggests that all four silty sands follow the same trend. A decreasing ratio of $S_u/\sigma'_m$ is consistent with the more positive $\psi$ and approaches zero at $\psi = 0.12$. On the other hand, if the value of $\psi$ is negative, this ratio is significantly higher and there is a sudden increase at $\psi = -0.08$. Theoretically, this ratio is a function of the state parameter and the slope of the SSL. Such a plot is useful and can be used to estimate the in situ steady state shear strength should one could measure the values of the state parameter in situ.

Observations of a similar plot for clean sands (Fig. 5.23) show that Sacramento River Sand essentially follows those of silty sands, although the trend line is slightly higher. However,
Fig. 5.21. PEAK AND STEADY STATE FRICTION ANGLE VS. STATE PARAMETER
Fig. 5.22. Normalized steady state undrained shear strength vs. state parameter.
Fig. 5.23. NORMALIZED STEADY STATE UNDRAINED SHEAR STRENGTH VS STATE PARAMETER FOR CLEAN SANDS
data of Banding #6 sand shows a ratio of $S_{us}/\sigma_m'$ much lower than the silty sands, as low as 0.1 was indicated for this ratio for a range of state parameter from -0.02 to 0.02, and the trend line of the data is dissimilar from the other soils. The reason for this is not clear, except that it was found that $\phi_s$ for Banding #6 sand was lower than that of the other sands.

Similar trends were also observed when the ratio of the peak deviator stress, $q_{max}$ over the mean effective stress, $\sigma_m'$ is plotted versus the state parameter, $\psi$ (Fig. 5.22 and Fig. 5.23). Unlike the steady state shear strength, $S_{us}$, the peak deviator stress, $q_{max}$ was reached at a small strain, and therefore effects of initial fabric (method of sample preparation) was expected to influence the results. However, the data did not reflect this.

5.4.5.5. Pore Pressure during Shear as A Function of State Parameter

At very low strains, even a dilative material will yield some value of positive pore pressure upon shearing before the dilation tendency starts to decrease the excess pore pressure. In this study, attempts were made to correlate the maximum positive pore pressure during the test and the residual pore pressure at steady state to the state parameter.

The pore pressures were normalized by the initial mean effective stress, $\sigma_m'$ and these values are shown in Fig. 5.24. For positive state parameters, the pore pressure at steady state is closer to the maximum pore pressure. At state parameters equal to 0.08 or higher, these values are essentially the same. At state parameter less than -0.05, the residual pore pressure is negative. The trend of the maximum pore pressure and the trend of the residual pore pressure are indicated by lines on the figure. Cone penetration of silty sand may be fully undrained or partially drained due to the intermediate permeability of this material compared to clean sands and clays. If the penetration is undrained, it is expected that the pore pressure registered during the cone penetration testing will fall in the region between these two lines. However, if the penetration of silty sand is partially drained, then the pore pressure ratio is
Fig. 5.24 NORMALIZED PEAK DEVIATOR STRESS VS. STATE PARAMETER
Fig. 5.25. PEAK DEVIATORIC STRESS VS. $\psi$ FOR CLEAN SAND
smaller than the residual pore pressure ratio. Further discussion will be presented in chapter 6.

5.4.5.6. Shear Strength Reduction

The typical effective stress behavior of a loose fine grained cohesionless soil is shown in Fig. 5.27. The soil reaches a peak strength at a very small strain, usually on the order of 1%, and then experiences a marked reduction in shear strength. After substantial strain, the sample reaches a residual strength, $q_r$, indicating that under this conditions, the flow structure of the soil has developed.

Bishop (1973) proposed that the potential for post failure flow should be related to the peak and residual strengths by use of soil parameter termed the brittleness index ($l_b$) which is defined as:

$$l_b = \frac{\tau_r - \tau_f}{\tau_f} \quad \text{[eq.5.10.]}$$

where $\tau_r$ is the peak shear stress and $\tau_f$ is the residual shear stress (see Fig. 5.27.). In terms of parameters previously defined in this study, the expression can be written as follows:

$$l_b = \frac{q_{\text{max}} - q_{ss}}{q_{\text{max}}} \quad \text{[eq.5.11.]}$$

in which $\tau_r = q_{\text{max}}$ and $\tau_f = q_{ss}$. Bishop suggested that soils with high values of $l_b$ are more susceptible to flow slides, however this method does not provide a basis for predicting the extent of flow.

One of the purpose of this research was to develop a simple quantitative approach to predicting the post-failure behavior of liquefied soils based on examination of the stress-strain behavior of sand containing fines. From a liquefaction viewpoint, there is concern in reduction
Fig. 5.26. PORE PRESSURE GENERATED DURING UNDRAINED SHEAR VS. $\psi$

\[ \frac{\Delta u}{\sigma_{mn}'} \]

STATE PARAMETER, $\psi$

\[ \Delta u_{\text{max}} \]

\[ \Delta u_{\text{residual}} \]
Fig. 5.27  DEFINITION OF BRITTLENESS INDEX

\[
I_b = \frac{\tau_r - \tau_f}{\tau_f}
\]

\[
I_b = \frac{q_{\text{max}} - q_{ss}}{q_{\text{max}}}
\]

\[\tau_r = q_{\text{max}}\]

\[\tau_r = q_{ss}\]

\[\tau_f - \tau_f\]
in strength and the increase in pore pressure which occur thereafter. Hird and Hassona (1986) used the ratio $q_{ss}/q_{\text{max}}$ to describe the severity of liquefaction and plotted this ratio against state parameter, $\psi$. This quantification is a step ahead of the Bishop’s approach. To simplify the discussion, an index suggested for this ratio, termed Ductility Index, $I_q$ which is defined as:

$$I_q = \frac{q_{ss}}{q_{\text{max}}} \quad \text{[eq.5.12.]}$$

$I_q = 1.0$ if the soil after failure is completely ductile and equals 0.0 if the soil loses all its strength in a completely brittle failure. In many respects, this expression is related to the Brittleness Index proposed by Bishop and could be stated as follows:

$$I_q = 1 - I_b \quad \text{[eq.5.13.]}$$

Fig. 5.28 gives the $I_q$ values plotted vs state parameter for the test data for the silty sand of this investigation as well as those for the silty sand from the Lower San Fernando Dam and clean sands as reported by Castro (1969) and Hird and Hassona (1986). The sands containing mica are the most ductile.

Alternatively, the severity of liquefaction can be thought in terms of a reduction in mean normal effective stress $\sigma_{m}'$ such as suggested by Hird and Hassona (1986):

$$I_p = \frac{(\sigma_{m}')_s}{(\sigma_{m}')_c} \quad \text{[eq.5.14.]}$$

where $(\sigma_{m}')_s$ and $(\sigma_{m}')_c$ refer to mean effective stress at steady state and at initial state consecutively. In this case, then:

$$I_p = 10^{\psi/\lambda_{ss}} \quad \text{or} \quad I_p = \exp(\psi/\lambda_{ss}) \quad \text{[eq.5.15.]}$$

depending on the root of the logarithm used for the axis of $\sigma_{m}'$. The physical representations of eq. 5.15. is presented in Fig. 5.29. where it can be seen that at positive state, the potential
Fig 5.28 LIQUEFACTION SEVERITY VS. STATE PARAMETER
Fig. 5.29 SEVERITY OF LIQUEFACTION POTENTIAL IN TERMS OF MEAN EFFECTIVE STRESS

\[ \frac{\epsilon}{\sigma_{\text{eff}}} = \nu \]

\[ \lambda_{ss} = 0.05 \]
\[ \lambda_{ss} = 0.10 \]
\[ \lambda_{ss} = 0.15 \]

\[ \nu = 1.0 \]
Fig. 5.30. SEVERITY INDEX FOR SILTY SAND
Fig. 5.31. SEVERITY INDEX OF BANDING #6 SAND AND SACRAMENTO RIVER SAND
of liquefaction is higher for sands with flatter SSL slopes. Figures 5.30. and 5.31. show the $l_p$ values from the soils considered in this investigation plotted against $\psi$. It is interesting to note that the trend of $l_p$ and $S_{u}/\sigma_{m}^*$ against $\psi$ is similar. If both curves were plotted in the same graph, two parallel lines will be shown.

5.5. CYCLIC TRIAXIAL TESTS ON SILTY SANDS

5.5.1. Test Procedures

Cyclic triaxial tests were performed on reconstituted samples of Yatesville silty sand and Pepper’s Ferry silty sand. These samples were prepared in the same manner as in the static triaxial tests, with the samples being consolidated to $K_0$ conditions from a slurry in the batch consolidometer, or compaction with a Harvard miniature apparatus. The void ratios resulting in these fabrication techniques ranged from 0.36 to 0.45.

Samples were tested at isotropic confining pressure of 70, 100, and 140 kPa. Ishihara et al. (1978) reported that the cyclic resistance of silty sand was a function of the overconsolidation ratio, OCR, therefore samples were tested for values of OCR of 1 and 2. In all, 37 tests were conducted on Yatesville silty sand and 9 tests were conducted on Pepper’s Ferry silty sand. The tests on Yatesville silty sand included samples consolidated from slurry and those fabricated by compaction. However, the tests on Pepper’s Ferry silty sand covered only samples consolidated from a slurry. Criteria for failure were the occurrence of full pore pressure ($r_u = 100\%$) or the double strain amplitude equals to 2.5%.

The cyclic tests were conducted on two different apparatus. A CKC Electro/Pneumatic device was used for the first series of tests on the Yatesville silty sand. For the remainder of the tests
on Yatesville and Pepper’s Ferry silty sand, an MTS cyclic test apparatus was employed. All
tests were conducted by stress control and a 0.5 hz sine wave loading function was used.

5.5.2. Test Results and Discussions

Fig. 5.32 and fig. 5.33. show the results from the cyclic triaxial tests on Yatesville silty sand
and Pepper’s Ferry silty sand in terms of cyclic stress ratio at failure vs. number of load cy-
cles. The cyclic stress ratio is defined as the ratio of the cyclic deviator stress, \( \sigma_{dc} \), to twice
the effective confining pressure, \( 2\sigma_\varepsilon' \), at the time of consolidation. Although considerable
scatter exists in the data, the following trends may be inferred:

1.) The effective confining pressure has no significant influence on the cyclic resistance. This
   would be expected since the cyclic stress ratio is normalized by the effective confining
   pressure.

2.) Samples with an overconsolidation ratio of 2 show a slight increase in cyclic resistance
   vs. those at OCR = 1.0

3.) Samples prepared by compaction, show that compacted samples exhibit higher cyclic
   resistance than those fabricated from slurry. Similar behavior was shown by Mullilis,
   Chan and Seed (1975) for clean sands where samples prepared by moist tamping showed
   a more resistance to liquefaction than samples prepared by pluviation due to different soil
   fabric.

4.) Pepper’s Ferry silty sand is slightly less resistant to liquefaction compared to Yatesville
   silty sand.

To compare the behavior of silty sands to clean sands under cyclic loading, other test results
on silty and sandy soils (performed by Hsing and Seed, 1988) and test results on clean
Monterey #0/30 sand (Sweeney, 1987) are considered.
Fig. 5.32. CYCLIC TRIAXIAL TEST RESULTS FOR YATESVILLE SILTY SAND
Fig. 5.33. CYCLIC TRIAXIAL TEST RESULTS FOR PEPPER'S FERRY SILTY SAND
Data from Hsing and Seed (1988) consisted of undisturbed samples from the Lower San Fernando Dam with fines content ranging from 5% to 85%. Out of twenty five samples tested, 5 samples were sandy silts of low plasticity and the rest were silty sand or sands. Most of the samples were isotropically consolidated with an initial effective confining pressure of 2 ksc. Some of the samples were anisotropically consolidated at $K_c = 1.75$ by applying an additional axial consolidation stress.

As shown on Fig. 5.34., the liquefaction resistance curves of the Lower San Fernando Dam soils follow conventional trend. As expected, the anisotropically consolidated samples show higher cyclic resistance in terms of cyclic stress ratio (CSR). At 15 cycles, the CSR for isotropically consolidated samples was 0.221 while for the samples anisotropically consolidated at $K_c = 1.75$ was 0.38; an increase of over 70%.

The cyclic triaxial tests on clean sand were conducted by Milstone (1985) and Muzzy (1983) on Monterey No. 0/30 sand. This sand was studied by Sweeney (1987) to develop correlation of the cyclic strength of the sand to the cone penetration resistance. The samples were prepared by air pluviation with average relative density of 40% and 60% by Milstone and Muzzy respectively. Fig. 5.35. shows the results of these tests.

A comparative observation is made by combining cyclic triaxial test results on Yatesville silty sand, Pepper's Ferry silty sand, undisturbed samples of the Lower San Fernando Dam and clean Monterey No. 0/30 sand (Fig. 5.36.). To simplify the observation, the curves relating to the type of soils were numbered correspondingly from 1 to 8. A direct comparison is not simple since the samples were prepared in different manners. However, it is shown that samples of silty sand exhibit a similar cyclic resistance to those of clean sands. In fact, samples from San Fernando Dam with varying fines contents plot in one trend line (Hsing and Seed, 1988) and fall slightly below the cyclic strength data of Yatesville silty sand prepared from a slurry with OCR = 1 (curve 1). Curve 1 of Yatesville silty sand is close to the pluviated samples of Monterey #0/30 at 40% relative density (curve 7).
Fig. 5.34 CYCLIC TRIAXIAL TEST RESULTS OF UNDISTURBED SAMPLES FROM LOWER SAN FERNANDO DAM (Hsing and Seed, 1988)
Fig. 5.35 CYCLIC TRIAXIAL TEST RESULTS FOR MONTEREY #0/30 SAND (Sweeney, 1987)
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<th>PREPARED</th>
<th>OCR</th>
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**Fig. 5.36** COMPARABLE CYCLIC TRIAXIAL TEST RESULTS OF SILTY SANDS AND CLEAN SAND
6.1. INTRODUCTION

The cone penetration test is a popular in situ test method for the characterization of soils. However, it must be accepted that the cone penetrometer is an empirical instrument. Soil properties are not directly measured with this device, but must be derived from existing empirical correlations or back-calculated from theories of penetration. The use of calibration chambers has proven to be a valuable tool in the development of the correlations. These chambers have the advantage over field test conditions in that the boundary conditions during penetration (stress or strain) may be controlled and accurately measured. Also, accurate determination of soil density removes an important element of uncertainty in developing the correlations with soil parameters.

While the calibration chamber offers advantages, it should be realized that the calibration chamber sample size and the imposed boundary conditions are important and can bias test data. Boundary conditions in calibration chamber are usually either constant stress or zero
lateral strain. In the ground, the boundary conditions can be expected to be between the limits of constant stress and zero deformation for both the vertical and horizontal directions. Holden (1977) suggested the following four limiting conditions which were designated as B1 through B4, and these have been adapted by most researchers.

\[
\begin{align*}
B1 : & \quad \sigma_v, \sigma_n \quad \text{constant} \\
B2 : & \quad \varepsilon_v = \varepsilon_n = 0 \\
B3 : & \quad \sigma_v \quad \text{constant, } \varepsilon_n = 0 \\
B4 : & \quad \sigma_n \quad \text{constant, } \varepsilon_v = 0
\end{align*}
\]

Of these possibilities, the most commonly applied boundaries are B1 and B3 (Parkin, 1988).

### 6.2. DEVELOPMENT OF CALIBRATION CHAMBERS

According to Parkin (1988), the original concept for calibration chamber can be credited to J.C. Holden at the Road Construction Authority (RCA) of Victoria, Australia. This chamber, housing a sample of 0.75 m dia. and 0.90 m high was constructed in 1969. The base piston was inflated by water and sample deformation was observed from changes in the water level. The sample was fabricated using travelling sand spreader. This calibration chamber could impose all four types of boundary conditions described previously. A detailed description of this chamber can be found in Veismanis (1974).

Following the Holden concept, calibration chambers were constructed over the past 20 years at the University of Florida, Norwegian Geotechnical Institute (NGI), Monash University, and the Italian Electricity Board (ENEL). The University of Florida chamber allowed for samples of 1.20 m dia. and 1.20 m high and had a design essentially similar to RCA chamber. A bigger chamber, 1.20 m dia. and 1.80 m high, was constructed at Monash University, Australia.
(Chapman, 1974). The NGI chamber had dimensions of a 1.20 m by 1.50 m. Two chambers, the same size as the NGI chamber, were developed in Italy. At the Italian Electricity Board (ENEL), Milan, significant developments (Bellotti et al., 1982) were made in selection of a precision servo-controlled mechanical drive for the penetrometer (replacing the hydraulic ram), a highly sensitive device for volume change measurements, and advanced methods for saturating samples.

A calibration chamber with dimension of 0.76m dia. and 0.8 m high was developed at the University of California, Berkeley (Villet, 1981; Tringale, 1983; Huntsman, 1985). Confining pressure was applied by pressurizing water surrounding the sample and the vertical stress was applied pneumatically by jacking apart the top load transfer plate and the chamber lid. The vertical force was measured using 8 load cells located on top of the load transfer plate. This arrangement allowed independent horizontal and vertical stresses control during penetration test, and the sample remained free to strain ($B_4$, boundary conditions).

The most recent chambers were constructed at Calgary, Canada (Been et al., 1987) and at Virginia Polytechnic Institute and State University (Sweeney, 1987; Eid, 1987). Except for the sample size, there are many similarities in the features of both chambers. The Calgary chamber was constructed by Golder Associates and has a sample size of 1.4 m dia. and 1.0 m high. The Virginia Tech calibration chamber, built in 1986, houses 1.5 m dia. by 1.5 m high sample. Both calibration chambers enclose the sample with a flexible membrane and use air pressure for confinement. Movable rigid plates are used at the bottom of the sample to apply vertical stresses independently of the confining stresses. The Virginia Tech calibration chamber applies a $B_4$ boundary condition.

Most data collected employing calibration chambers have only been for clean sands. Sweeney (1987) and Eid (1987) successfully fabricated Monterey #0/30 samples in the Virginia Tech calibration chamber by static pluviation using a multiple sieve diffusser. Tests in Calgary chamber, however, have been conducted using silty sand with fines contents ranging from 3
6.3. FEATURES OF VIRGINIA TECH CALIBRATION CHAMBER

The Virginia Tech calibration chamber was designed to accommodate various in situ probes such as the cone penetrometer, the dilatometer and the pressuremeter. A schematic of the chamber system assembled for cone penetration testing is shown in Fig. 6.1. A detailed description of the chamber has been presented by Sweeney (1987).

The overall size of the chamber is 1.85 m high and 1.8 m dia. and the chamber is capable of housing a 1.5 m dia. by 1.5 m high soil sample. The main part of the chamber is a 2.9 cm thick cylindrical shell bolted to a circular bottom plate 7.6 cm thick. The sample is contained in a membrane and lateral stress is applied to the sample by air pressure. Three pressure bags were mounted over the bottom plate and fitted to the base sample plate having the same diameter as the sample. These pressure bags can be inflated to apply the vertical pressure on the sample. Thus, the radial pressure and the vertical pressure can be independently applied to the sample.

Lateral movement of the sample base plate is prevented during inflation of the air bags by three guide rods located between the sample base plate and the chamber bottom plate. The guide rods are equally spaced in plan view and slide inside a 3.8 cm dia. pipe. This arrangement also prevents crushing of the air bag by the weight of the sample before the application of the vertical pressure.

The vertical compression of the sample can be measured using a linear variable displacement transformer (LVDT) mounted at the center of the base plate. When testing dry sand, the verti-
1. HYDRAULIC PISTON
2. TROLLEY
3. REACTION FRAME
4. POSITION TRANSDUCER
5. LOAD CELL
6. CONE PENETROMETER
7. PIN VALVE
8. PRESSURE GAGE
9. O-RING
10. TOP CHAMBER PLATE
11. TOP SAMPLE PLATE
12. SAMPLE BOTTOM PLATE
13. PRESSURE BAG

Fig. 6.1. VIRGINIA TECH CALIBRATION CHAMBER
cal displacement data permit the measurement of the constraint modulus corresponding to the applied vertical stress. For samples deposited in the chamber as a slurry, the LVDT allows the consolidation curve of the material to be determined.

The initial cylindrical shape of the sample is maintained by the sample former. This former is made of a perforated steel sheet 1.5 m dia. by 1.5 m high. A PVC liner with the same diameter as the former is used to house the sample inside the sample former. The liner is 30 mil thick and is about 5 in. higher than the former to provide a seal between the top sample plate and the top plate of the chamber.

The sample top plate and the top plate of the chamber both have seven holes concentrically aligned to provide access for the cone penetrometer. A plan view of the test holes is shown in Fig. 6.2. During consolidation of the slurry, these holes served as outlet for the water that was displaced due to the application of the radial and vertical stresses to the slurry.

The cone penetrometer was inserted into the sample by an 8 ton (7200 kg) hydraulic piston with a travel of 163 cm. The speed can be regulated by a pressure compensated flow control valve located on the hydraulic line. A steel frame is mounted on the top plate to provide the required reaction during cone insertion. The hydraulic piston is mounted on a trolley which allows lateral mobility of the piston along the reaction frame beam. The frame itself can be rotated around into three positions corresponding to the alignment of the test holes. This arrangement allows seven cone penetration tests to be conducted in one soil sample. Notably, the type of sand being tested and size of cone penetrometer will dictate whether the multiple penetration approach is valid since boundary conditions can affect the test results.
Fig. 6.2. PLAN VIEW OF THE TEST HOLES IN THE CALIBRATION CHAMBER
6.4. THE CONE PENETROMETERS

Both standard cones and miniature cones were used in this investigation. The standard cone has 10 cm$^2$ projected tip area with a 60° apex angle and a 150 cm$^2$ friction sleeve. This configuration has been specified as the standard type in both Europe and the USA. Two standard cones were used in this investigation. The first one has a pore pressure element mounted immediately above the cone tip which allows the measurement of the pore pressure generated during penetration. This cone is a subtraction cone and is manufactured by Hogentogler Co. In this design, the end bearing is sensed by compression in the cone load cell (C) and the combined forces of the cone bearing and the friction is sensed by the rear load cell (C + S). The friction is then obtained by subtracting the cone bearing from the combined force. The subtraction is done electronically inside the cone. The second cone penetrometer has circumferential load cells and is manufactured by Fugro. In this type of cone penetrometer, the end bearing causes compression in the cone load cell (C) while the sleeve friction puts the sleeve load cell (S) in tension. A schematic description of both types of cone penetrometers is shown in Fig. 6.3. Both cone penetrometers have a 5 ton capacity.

The minicone is basically a scaled down version of the Fugro electric cone. The minicone has a 4.2 cm$^2$ projected tip area and a 63 cm$^2$ sleeve area. It has a capacity of one ton, corresponding to a tip resistance of 220 bars, which is sufficient for liquefaction studies involving the penetration of shallow, loose to medium dense sand. Field case studies using the minicone have been presented by Sweeney and Clough (1987) and Dickenson et al. (1988).

Prior to advancing the piezocone into the calibration chamber specimen, the porous element behind the tip was saturated by immersing the cone tip into a water-filled vessel. Vacuum was applied to the vessel to deair the element. Normally, one hour was required for saturation. This technique has been successfully used by Huntsman (1985). Fig. 6.4 shows the vacuum vessel used for deairing the piezocone. The penetrometer was pushed hydraulically...
a) CIRCUMFERENTIAL LOADCELLS

b) LOADCELLS IN SERIES (SUBTRACTION TYPE)

Fig. 6.3. SCHEMATIC CROSS SECTIONS OF THE CONE PENETROMETERS
(Schaap and Zuidberg, 1982)
into the chamber at a constant speed of 20 mm/sec. During the penetration, the tip resistance and the frictional resistance (and pore pressures if applicable) were recorded. The output during the cone penetration was transmitted via cables from the tip of the cone through the push rod and then collected by a personal computer based data acquisition system.

6.5. SAMPLE FABRICATION AND TESTING PROCEDURES

Based on results of the small scale investigations of sample fabrication of Yatesville silty sand in the laboratory, consolidation from a slurry was the technique selected to use in the calibration chamber. The first step of the sample fabrication procedure was placement of the sample former which was held concentrically at the bottom by the sample bottom plate and at the top by the membrane clamping ring. The membrane was placed in the chamber, flattened out against the former jacket, and folded at the top into a lip in the membrane clamping ring. The membrane clamping ring was used only temporarily to stiffened the former and to hold the membrane as the slurry was poured.

Eight to ten layers of burlap cloth enclosed in a cotton fabric was used for both top and bottom drainage. Six plastic lines were attached to the bottom drainage layer to allow water to be removed through the top drainage layer. Except for the first test (test no. CC-1) that had radial drainage as well as top and bottom drainage, the other samples were consolidated by top and bottom drainage only.

The slurry was made as thick as possible to prevent any segregation of the soil particles and yet thin enough to be workable. This was satisfied using an average water content of about 24-28%. However, due to the large volume of the sample and to speed up the preparation, only visual judgement was relied upon to maintain the uniformity of the water content.
Fig. 8.4. THE VACUUM VESSEL TO DE-AIR THE TIP OF PIEZOCONE
thoughout the slurry. The slurry was prepared by hand mixing or by using a concrete mixer. Typically, when using the concrete mixer with two laborers, the fabrication of 92 cu.ft (2.6 cu.m) of slurry could be completed in about eight hours. Before it was consolidated, the slurry was allowed to compress for 24 to 48 hours under its own weight. Observation showed that the settlement of the slurry was about 0.5 inches from its original deposited surface.

The next step was to level the surface of the slurry and to close the sample with the sample top plate. At this stage, the clamping ring was removed. One O-ring on the sample top plate and one O-ring on the flange of the chamber shell were installed before covering the sample with the chamber top plate. The seven holes in the sample top plate were aligned with the holes in the chamber top plate. The chamber top plate was then bolted to the chamber with 42-in dia. bolts using a pneumatic wrench. The slurry was then consolidated. A photograph of the calibration chamber assembled with consolidation of a slurry in progress is shown in Fig. 6.5.

To prevent the vertical displacement from exceeding the maximum allowable displacement of the bottom pressure bags, the consolidation pressure was applied step by step. Typically radial pressure was applied first in increments of 2 psi every 24 hours until 6 psi was achieved. At this pressure, the contribution of the radial pressure to vertical force was still lower than the weight of the slurry, hence it was assumed that there was no vertical displacement.

After 24 hours of consolidation under 6 psi, the sample was ready to be fully consolidated under the desired final pressures. In the calibration chamber test program, the samples were consolidated under 70 kPa (10.15 psi), 100 kPa (14.5 psi) and 140 kPa (20.3 psi) with $K_o = 1.0$ (isotropically consolidated). At this point, the vertical movement of the sample was monitored over time and the consolidation curve was obtained. A typical consolidation curve of specimen No. CC-5 is shown in Fig. 6.6.
Fig. 6.6: TYPICAL CONSOLIDATION DATA FROM SPECIMEN NO. CC-5

SPECIMEN No. CC-5

$\sigma_c = 100$ kPa

TIME (hours)

COMPRESSION (mm)
When sample was ready for testing, the reaction frame was mounted on the top plate and the cone penetrometer was installed. Multiple minicone penetration tests in the exterior holes and one standard cone penetration test in the center could be carried out per calibration chamber specimen. In this research, the six minicone penetration tests were performed first and the standard cone penetration test was done at least two days later to allow dissipation of pore pressure generated during the minicone penetration tests. Fig. 6.7 shows a photograph of the chamber ready for testing.

Upon completion of each penetration test, the sample was excavated carefully. One shelby tube sample and several small block samples were taken to measure the density profile of the calibration chamber specimen. This excavated material was then re-used to form the following calibration chamber specimens.

In all, five calibration chamber specimens were formed during the course of this research. The first two calibration chamber specimens were penetrated only once using a standard cone. The rest of the specimens were for multiple penetration tests with one standard cone used in the middle hole, and the minicone for the peripheral hole. A total of five standard cone penetration tests and 18 minicone penetration tests were conducted.

6.6. THE CALIBRATION CHAMBER SPECIMEN

6.6.1. Consolidation Data

As discussed previously, the specimens were consolidated in stages. During the radial consolidation, no vertical compression occurred since the sample was constrained by rigid top and bottom plate. However, vertical compression of the sample was measured for the final
Fig. 6.7. SET UP OF CONE PENETRATION TESTING IN THE CALIBRATION CHAMBER
isotropic consolidation stage. The consolidation data of the five calibration chamber specimens are shown in appendix B.1 - B.5. A typical time for consolidation with functioning drainage at the top and bottom of the sample takes about 250 - 400 hours (10 - 16 days). Specimens CC-1, CC-2 and CC-5 are examples of this. However, when the bottom drainage failed, the consolidation took considerably longer (sample CC-3 and CC-4).

Due to an air leak in the chamber, specimens no CC-1 and CC-2 had to be reconsolidated. Specimen CC-1 had been consolidating for 4 days (about 100 hours) when the pressure was released and the slurry was reconsolidated. Fig. B.1 shows both the initial consolidation and the continuation of the consolidation progress. Specimen CC-2 was reconsolidated only after one day from the initial application of pressure. In both cases, continuation of the consolidation did not cause a sudden compression.

In specimens CC-3 and CC-4, the bottom drainage failed. The plastic lines carrying the water to the surface were plugged with the slurry or were not in the proper position. Apparently, the bottom part of the sample was not fully consolidated. Results of the cone penetration tests for these two specimens showed that the lower parts of the soil were softer than observed in other tests. Measurement of the void ratios also indicated consistently lower densities.

Because of this experience, bigger plastic lines were used for specimen no. CC-5. The consolidation data (Fig. B.5) shows that the sample consolidated in much less time and results of the cone penetration tests and void ratio measurements show a more homogeneous specimen.
6.6.2. Specimen compressibility

The vertical displacements for the final consolidation stages ranged from 20 mm to 45 mm. However, there was no direct relationship between the amount of compression and the applied vertical pressure in all the specimens. The volume change of each specimen can be calculated using eq. 5.1. by substituting the values of the initial specimen height, $H_i$, and the final specimen height, $H_e$. The initial height is 4'11" (149.8 cm) and the final height of the specimen was calculated by subtracting the measured vertical compression from the initial height. Using eq. 5.3., the volumetric compressibility, $m_v$, of the specimens be computed and the results are shown in Table B.1. The range of values of $m_v$ is 0.002 - 0.005 /psi. These values are higher than those observed in the laboratory (Table A.1), which is probably due to the differences in their initial conditions. In the laboratory, the sample was previously consolidated in the batch consolidometer or compacted in the Harvard apparatus before consolidated in the triaxial test, while in the calibration chamber, the samples were consolidated directly from a slurry.

6.6.3. Void Ratio, Density and Water Content Profiles

After cone penetration tests were conducted in the calibration chamber, the entire specimen was carefully excavated. Block samples and thin-walled tube samples were obtained during the excavation at 1 ft. depth intervals for determination of void ratio and water content. It should be noted that the density tests could only be conducted after the stress was released from the specimen, so the void ratio of the sample during penetration testing would be lower than that measured. However, the slope of the rebound curve determined from consolidation tests, $C_r$, was very small, so the change in void ratio was considered to be negligible.
Density and water content profiles for calibration chamber test specimens are shown in Tables B.2 to B.6. Except for the first foot of soil, the degrees of saturation achieved using the slurry consolidation technique averaged 95%. There was some variation of void ratio with depth in the test specimens, with the largest spread of 0.41 - 0.54. This variation of void ratio was most evident in tests CC-3 and CC-4 due to the failure of the bottom drainage during consolidation. In all tests, however, there was not a significant lateral variation of void ratio.

6.7. CONE PENETRATION TEST RESULTS

6.7.1. Cone Tip Resistances

A total of 23 cone penetration tests were performed in 5 calibration chamber specimens including 5 standard cone and 18 minicone penetration tests. In specimens No. CC-1 and CC-2, only standard cone penetration tests were conducted, and in specimens CC-3, CC-4, and CC-5, both the standard cone and the minicone were used. The results for all tests are shown in Figures B.6.-B.26 in the appendix. The piezocone was used for specimens No. CC-1, CC-2 and CC-3. However, the test on specimen CC-3 was bad due to problem with the cone’s internal electronics. Tests on specimens CC-4 and CC-5 were conducted using the standard Fugro cone. Unfortunately, the data of test on specimen CC-4 was lost due to a mistake in setting up the test. Therefore, standard cone penetration test data are only available for specimens No. CC-1, CC-2, and CC-5. Fortunately, all tests using the minicone were successful, and these tests show consistency and repeatability.

A typical piezocone test result is shown in Fig. 6.8. and the results of six minicone tests and one standard cone test in specimen CC-5 are shown in Fig. 6.9. Three important conclusions can be drawn from these figures.
1.) The multiple penetration testing method is repeatable and consistent
2.) There is no scale effect between the standard cone and the minicone
3.) There seems no bias on results due to boundary effects

The key cone penetration results for all calibration chamber tests are presented in Table 6.1. This table shows the consolidation pressure, void ratio, state parameter and normalized tip resistance. In tests where multiple cone penetration tests were conducted, all data were averaged equally.

Shown in Fig. 6.10. is the logarithm of the average of tip resistance plotted against the average void ratio for calibration chamber tests consolidated at three different pressures. There is a gentle trend of decreasing tip resistance with increasing void ratios for void ratios less than 0.46. For void ratio greater than this, the tip resistance decreases more dramatically. Consolidation pressures seem to have no direct influence on the penetration results, but the data are too limited to make solid conclusions. This is discussed later in this report.

The tip resistances measured in the silty sand specimens were much lower than one would expect for clean sands. Shown in Fig. 6.11. is the tip resistance - depth relationship measured for specimen CC-1, and calibration chamber results of Monterey 0/30 at relative densities of 24% and 63%. All curves were obtained with a 10 cm² cone penetrometer in a vertical effective stress of about 1 kg/cm². Notably, the tests on the silty sand were conducted with isotropic stress conditions while the tests on Monterey #0/30 utilized Kᵥ consolidation stresses. The penetration resistances measured for Monterey #0/30 at 63% and 24% relative density were about ten and five times higher respectively than that measured for the silty sand. Reasons for such wide differences in tip resistance probably are caused by a combination factors including density, sand gradation, and degree of dissipation of excess pore pressures during cone penetration. As to the latter item, the tests performed with the Monterey #0/30 sand were fully drained in as much as the sand in a dry condition. In the
Fig. 6.8. TYPICAL PIEZOCONE TEST RESULTS IN CALIBRATION CHAMBER SPECIMEN NO. CC-1.
Fig. 6.9. TIP RESISTANCE FROM MULTI PENETRATION TESTS IN SPECIMEN NO.CC-5
Table 6.1. RESULTS OF CPT IN THE CALIBRATION CHAMBER

<table>
<thead>
<tr>
<th>NO.</th>
<th>$\sigma_c$ (kPa)</th>
<th>depth range</th>
<th>$q_c$ (avg)</th>
<th>$q_c - \sigma_m$ / $\sigma_m'$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(bars)</td>
<td>(MPa)</td>
<td></td>
</tr>
<tr>
<td>C1-100</td>
<td>100</td>
<td>0-1</td>
<td>14.0</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>2-5</td>
<td>16.0</td>
<td>1.57</td>
</tr>
<tr>
<td>C2-70</td>
<td>70</td>
<td>1-2</td>
<td>27.0</td>
<td>2.65</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>2-3</td>
<td>35.0</td>
<td>3.43</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>3-5</td>
<td>15.0</td>
<td>1.67</td>
</tr>
<tr>
<td>C3-140</td>
<td>140</td>
<td>0-2</td>
<td>16.0</td>
<td>1.57</td>
</tr>
<tr>
<td></td>
<td>140</td>
<td>2-5</td>
<td>6.5</td>
<td>6.37</td>
</tr>
<tr>
<td>C4-70</td>
<td>70</td>
<td>0-1</td>
<td>21.5</td>
<td>2.11</td>
</tr>
<tr>
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<td>70</td>
<td>1-3</td>
<td>10.0</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>3-5</td>
<td>5.0</td>
<td>0.49</td>
</tr>
<tr>
<td>C5-100</td>
<td>100</td>
<td>0-5</td>
<td>10.0</td>
<td>0.98</td>
</tr>
</tbody>
</table>
Fig. 6.10. TIP RESISTANCE VS. VOID RATIO IN THE CALIBRATION CHAMBER TESTS
Fig. 6.11. COMPARISON OF TIP RESISTANCE FOR YATESVILLE SILTY SAND AND CLEAN MONTEREY NO.0/30 SAND
6.7.2. Use of Cone Tip Resistance to Characterize Sand Behavior under Load

Empirical, semi-empirical, and analytical methods are available for interpretation of the cone tip resistance data. While experience exists with interpreting cone penetration tests in sands and clays, it is not clear that these methods are applicable for interpreting cone penetration data in silty soils. Cone penetration tests in clays are usually assumed to be undrained, and tests in clean sands are considered to be drained. However, silty sands have a permeability between that of clay and clean sand. Thus, the evaluation of material properties of the silty sand using procedures developed for the drained behavior of sand or the undrained behavior of clay has to be viewed with skepticism.

McNeilan and Bugno (1985) suggested that permeability and associated drainage effects are of primary importance to penetration testing results and that strength, density and overburden effects are of secondary importance. They recommended using the following rules: (1) For permeabilities less than about $10^{-7} - 10^{-8}$ cm/sec., the soil behaves in an undrained manner and, (2) For permeabilities exceeding about $10^{-4} - 10^{-3}$ cm/sec., the soil behaves in a nearly drained manner. However between these two limits, the soil should behave in a partially drained manner. Based on average value of $C_v$ data (Fig. 4.9. and 4.10.) and average value of $m_v$ from Table B.1., the permeability $k$ value of the Yatesville silty sand can be estimated using eq. 6.1.

\[ k = C_v \cdot m_v \cdot \gamma_w \quad [\text{eq.6.1.}] \]

Using $C_v = 0.103 \text{ in}^2/\text{min}$ and $m_v = 0.0035/\text{psi}$ the above expression give an approximate $k$ value of $0.55 \times 10^{-6}$. Permeability tests and consolidation tests conducted by Corps of Engi-
neers (1985) on undisturbed samples of Yatesville silty sand indicated permeabilities in the range of $2.4 - 36.4 \times 10^{-6}$ cm/sec. for fines contents ranging from 10 to 42%. Therefore, according to McNeilan and Bugno (1985), Yatesville silty sand falls at the boundary between undrained and partially drained behavior during penetration test. Thus, in contrast to the calibration chamber tests with Monterey #0/30 sand, the tests of this investigation were influenced by the excess pore pressures during cone penetration.

Relative density is a common soil parameter correlated to tip resistances measured in clean sand. Previous work in large calibration chambers (Veismanis, 1974; Chapman and Donald, 1981; Baldi et al., 1981; Parkin et al., 1980; and Been and Jefferies, 1987; Sweeney, 1987; and Villet and Mitchell, 1981) has provided correlations of cone penetration resistance, $q_c$, with the relative density for many soils. Most of this work has shown that no single relationship exists for all sands. Robertson and Campannella (1985) emphasized that no unique relationships exists between cone resistance, in situ effective stress and relative density since other factors, such as soil compressibility, also influence cone penetration results. Since the compressibility of silty sands is typically higher than that of clean sands, this also suggests that silty sand may have a lower tip resistance than clean sand if all other factors are equal.

Been and Jefferies (1986) and Jefferies (1988) have used the state parameter for interpretation of cone penetration data of cohesionless soils, including sands and silty sands. In their method, the tip resistance is normalized by the mean effective stress, $\sigma_{\text{m}}$. The use of the state parameter incorporates the void ratio, the effective stress, and the other characteristics of the sand represented by the slope of its steady state line.

As suggested by Jefferies (1988) (equation 2.22.), a single equation can represent the correlation of the normalized tip resistance and the state parameter by inclusion of the slope of SSL of the sand, $\lambda_{\psi}$. This correlation is presented in Fig. 6.12. It is shown that for a given value of $\psi$, sands with steeper SSL have a lower tip resistance. According to observations by other researchers (Poulos et al., 1985; Castro, 1987), the slope of the SSL increases with increasing
soil compressibility. Thus, this confirms the previous findings by researchers that at similar relative densities and overburden pressures, a more compressible sand exhibit smaller values of $q_c$.

The test results from the calibration chamber are plotted in terms of the normalized tip resistance versus the state parameter in Fig. 6.13 (linear scale) and in Fig. 6.14 (semi-log scale). In Fig. 6.13, the line representing Jefferies equation with $\lambda_{es} = 0.168$ of Yatesville SSL is also shown. The test data of Yatesville silty sand show different relationship than that suggested by Jefferies. A better fit to the data acquired in this research would be the log-linear relationship shown in Fig. 6.14. Been and Jefferies (1987) reported that the tip resistance of all sands and sands with small amount of fines strongly dependent on the state parameter (Fig. 2.23). However, drainage is an important issue for the case of silty sand. The greater the value of the state parameter, the greater the pore pressure that will be generated during the cone penetration for both drained and partially drained conditions. This would result in a lower shear strength. The slope in Fig. 6.14 is probably a function of the soil compressibility and the drainage as well. It should be noted that Jefferies equation was derived mainly from drained penetration test results, thus incorporating the drainage characteristics during penetration is recommended for soil which is partially or fully undrained upon loading.

6.7.3. Interpretation of the Sleeve Friction

It is well known through studies by Begemann (1965) that friction ratio, FR, is closely related to the soil type. Charts to determine soil types based on values of FR and $q_c$ have been proposed by Schmertmann (1978a) and Robertson and Campanella (1984).
\[ \psi = \frac{1}{[8.1 - \ln(\lambda)] \cdot \ln \left( \frac{q_e - \sigma_m}{\sigma_m} \right)} \cdot \ln \left( 8 + 0.55(\lambda - 0.01) \right) \]
Fig. 6.13. NORMALIZED TIP RESISTANCE VS. $\psi$ IN LINEAR SCALE

- $\sigma_0 = 70$ kPa
- $\sigma_0 = 100$ kPa
- $\sigma_0 = 140$ kPa

In THIS RESEARCH, the normalized tip resistance is compared with Jefferies' equation $[\lambda_{ss} = 0.168]$. The data points represent the measurements at different state parameters $\psi$. The line represents the theoretical prediction according to Jefferies' equation.
Fig. 6.14. NORMALIZED TIP RESISTANCE VS. \( \psi \) IN SEMI LOG SCALE

\( \sigma = 70 \text{ kPa} \)
\( \sigma = 100 \text{ kPa} \)
\( \sigma = 140 \text{ kPa} \)

STATE PARAMETER, \( \psi \)
Calibration chamber results for Yatesville silty sand show that the measured friction ratios are higher than those suggested by the Schmertmann or the Robertson Campanella charts. The suggested range of friction ratio for silty sand is from 1 to 4 %, while most of the data from the calibration chamber show values from 2 to 6 % and occasionally as high as 8 to 10%. Test data collected using the Hogentogler piezocone for friction ratio was biased due to an interaction between the tip and the sleeve. As load was applied to the tip, the friction sleeve output changed proportionally. In specimen CC-1, the FR went up to 10% and even higher in specimen CC-2. This high friction ratio is outside the limit in the chart of classification suggested by Robertson and Campanella (1984) and can not be used for analysis. However, the tip results were consistent and reliable.

Friction ratio is sensitive to lateral stresses and soil fabric (Huntsman, 1985; Houlsby et al., 1988). In these calibration chamber tests, a K value of 1.0 was used. Apparently this high value of K could possibly explain the large friction ratio measured.

6.7.3. Interpretation of the Generated Pore Pressure During Penetration

As previously discussed, McNeilan et al. (1985) suggested that due to the intermediate permeability, silty sand would respond in partially drained behavior. In calibration chamber tests where pore pressures were measured (specimen No. CC-1 and CC-2), excess positive pore pressure were developed. It is shown in Figures B.6. and B.7. in appendix that higher pore pressure is associated with lower penetration resistance (Fig. B.6. at a depth of 2 to 5 ft.), and a slightly negative or low pore pressure value is associated with higher tip resistance (Fig. B.6. at a depth of 1 to 2 ft., and Fig. B.7. at a depth of 2 to 3 ft.).
A classification chart proposed by Senneset and Janbu (1984) is shown in Fig. 6.15, where the pore pressure coefficient, $B_q$, is defined as the ratio of the net pore pressure, $(u_{\text{max}} - u_0)$, to the net cone tip resistance, $(q_t - \sigma_w)$:

$$B_q = \frac{(u_{\text{max}} - u_0)}{(q_t - \sigma_w)} \quad \text{[eq. 6.2.]}$$

where $u_{\text{max}}$ is measured pore pressure, $u_0$ is the hydrostatic pore pressure, $q_t$ is total cone penetration resistance and $\sigma_w$ is the total overburden pressure. A range of the data of Yatesville silty sand based on Fig. 6.6 and Fig. 6.7 is plotted on this classification chart (Fig. 6.15). The data falls in a zone below the loose and medium sand, which is a reasonable zone for silty sand. The $B_q$ values for silty sand range from a slightly negative to about 0.1.

It is also of interest to compare the pore pressure measured during cone penetration with the pore pressure generated during undrained shear in the laboratory. Based on the permeability of Yatesville silty sand, it appears that under cone penetration, this soil behave in an undrained or partially drained mode. Hence, it is expected that the pore pressure generated during a cone test would be comparable to that observed in the laboratory.

It has been discussed in the previous chapter that when a soil specimen is sheared, strain it would generate pore pressure at small strain and then depending on the state of the soil, it could decrease if dilative or increase if highly contractive. If the loading is undrained, the pore pressures generated during cone penetration would be expected to be between the maximum pore pressure during undrained shear in the laboratory and residual pore pressure at steady state. For this analysis, only penetration data for specimen No. CC-1 (Fig. 6.6 in appendix) is used where the pore pressure response is stable and constant at a depth of 2 to 5 ft. Fig. 6.16 shows the measured pore pressure ratio, $\Delta u/\sigma_w^\prime$, compared to that generated by triaxial shear. The pore pressure measured is close to the residual pore pressure, yet it is still questionable whether this actually represents the residual pore pressure. However,
Fig. 6.15. TENTATIVE CLASSIFICATION CHART BASED ON $q_c$ AND $B_q$
(Senneset and Janbu, 1984)
Fig. 6.16. Comparison of Pore Pressure Ratio generated by CPT and by Shear in Laboratory.
an approach like this may be valuable in estimating the state of the soil, therefore further investigation is suggested.

### 6.7.4. Prediction of void ratio by CPT.

Methods suggested by Jefferies (1988) allow prediction of the void ratio from cone penetration data. These require knowledge of the SSL, and for Yatesville silty sand, the SSL has been established (Fig. 5.7.) with $i_s = 0.168$. Equation 2.23 was used to predict the state parameter, $\psi$, from the normalized $q_c$. Based on the SSL and the vertical and horizontal stresses, the value of void ratio at steady state ($e_s$) can be estimated. Finally, the void ratio in situ can be calculated using equation 2.24. Table 6.2 shows the results of the calculations.

The predicted void ratio is plotted versus the measured void ratio in Fig. 6.17. It is shown that there is a systematic error in this prediction. At void ratios less than about 0.45, the measured void ratios are higher than the predicted void ratios, and vice versa for void ratios higher than 0.45. Apparently, the difference can be attributed to the fact that the correlation of $q_c$ vs state parameter for the silty sand does not follow Jefferies’ equation (Fig. 6.12).

When applying this prediction method in the field, care should be taken as slight differences in the soil gradation shift the vertical position of the SSL. Although $\psi$ is correctly estimated, the void ratio in situ cannot be based on a general SSL. Also when applying this method for the prediction of the undrained steady state strength, a slight mistake could lead to substantial differences.
<table>
<thead>
<tr>
<th>TEST #</th>
<th>depth range (ft)</th>
<th>$q_c - \sigma_m$</th>
<th>$\Psi_{\text{pred}}$</th>
<th>$e_{ss}$</th>
<th>$e_{\text{pred}}$</th>
<th>$e_{\text{measured}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>($\sigma_m$)</td>
<td></td>
<td></td>
<td>(ave)</td>
<td>(low)</td>
</tr>
<tr>
<td>C1-100</td>
<td>0-1</td>
<td>12.7</td>
<td>0.010</td>
<td>0.464</td>
<td>0.454</td>
<td>0.442</td>
</tr>
<tr>
<td></td>
<td>2-5</td>
<td>14.7</td>
<td>0.025</td>
<td>0.464</td>
<td>0.439</td>
<td>0.455</td>
</tr>
<tr>
<td>C2-70</td>
<td>1-2</td>
<td>36.8</td>
<td>0.118</td>
<td>0.490</td>
<td>0.372</td>
<td>0.433</td>
</tr>
<tr>
<td></td>
<td>2-3</td>
<td>48.0</td>
<td>0.145</td>
<td>0.490</td>
<td>0.346</td>
<td>0.419</td>
</tr>
<tr>
<td></td>
<td>3-5</td>
<td>22.8</td>
<td>0.070</td>
<td>0.490</td>
<td>0.421</td>
<td>0.470</td>
</tr>
<tr>
<td>C3-140</td>
<td>0-2</td>
<td>10.2</td>
<td>0.012</td>
<td>0.440</td>
<td>0.453</td>
<td>0.420</td>
</tr>
<tr>
<td></td>
<td>2-5</td>
<td>3.6</td>
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<td>0.440</td>
<td>0.558</td>
<td>0.488</td>
</tr>
<tr>
<td>C4-70</td>
<td>0-1</td>
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<td></td>
<td>1-3</td>
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<td></td>
<td>3-5</td>
<td>6.2</td>
<td>0.062</td>
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<td>C5-100</td>
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<td>9.2</td>
<td>0.022</td>
<td>0.467</td>
<td>0.490</td>
<td>0.480</td>
</tr>
</tbody>
</table>
Fig. 6.17. PREDICTED vs. MEASURED VOID RATIO
6.7.5. Comparison to Field Penetration Resistance at Yatesville Dam site

The silty sand used in the calibration chamber test was taken from the Yatesville dam excavation in Yatesville, Kentucky. It is interesting to compare the penetration characteristics of this material in the calibration chamber and in situ to SPT results. Cross sections of the section investigated for the dam design are shown in Fig. 6.18. Loose to medium silty sand layers overly the foundation sands. The SPT blowcounts of the loose silty sand layers range from 1 to 10 and those of medium layers from 5 to 15. Based on Seed’s et al. SPT - CPT correlations (1986), a $\frac{q_c}{N}$ ratio of 3.5 can be used for a $D_{90} = 0.1$ mm. Using this ratio, the SPT data correspond to $q_c$ values of 3.5 - 35 bars for loose silty sand and 17.5 - 52.5 bars for medium silty sand. Those values are in the same range as those obtained in the calibration chamber (5-35 kg/cm²).
Fig. 6.18. TYPICAL BORING LOG AT YATESVILLE DAM, BLAINE CREEK, KENTUCKY
The Held tests were performed with the following objectives in mind:

1. To allow project personnel to gain experience with the CPT in a field setting.

2. To develop data on silty sand that would supplement that obtained in the calibration chamber.

3. To examine effects of cone size and the relationship between blowcount from the SPT and the tip resistance from the CPT.

Analysis of the liquefaction resistance of the soil deposit using CPT data based on methods described in chapter 2 is addressed in the following chapter.

7.1. TEST SITE

The test site, known locally as Pepper's Ferry is located about 7.5 miles south-west of Blacksburg on the bank of New River along the Virginia state road 114. A map showing the location and its geology is shown in Fig. 7.1. This site has been used for the studies of in situ
tests by the geotechnical engineering group at Virginia Tech. Soil exploration and laboratory testing were conducted at this site as part of instruction in the Civil Engineering Department. Fig. 7.2. shows approximate site locations of the CPT and SPT holes that have been documented by Dickenson et al. (1988) and Castro (1989).

Dickenson et al. (1988) conducted 4 CPT’s using the minicone penetrometer and an SPT and described the site as a fluviatile terrace deposit. Three of the CPT’s suggested that the soils were a relatively uniform silty sands. The fourth CPT was located closer to the river than the other three, and it encountered a more clayey silty soil. Further work at the site by Castro (1989) showed that certain portions of the terrace were composed of clay. The sandy soils were found to be concentrated on a knoll which was approximately 30 yards from and parallel to the New River. A number of CPT’s, SPT’s and laboratory studies in the clayey zone were conducted by the Virginia Tech geotechnical group but are outside the scope of this work. According to Castro (1989), the ground water table at this site ranges from 12 ft. to 15.5 ft. from the ground surface.

Fig. 7.3. gives the blowcount records for the two SPT’s performed in the silty sand area. The two records are similar except in the upper part where boring A1 shows higher blowcounts. It can be seen that to a depth of about 7m, the blowcounts typically fall between 3 to 9 blows/ft. Below 7 m and the top of rock (12.5m), the blowcounts increase, with values reaching as high as 30 blows/ft. To the depth of 7 m, conventional classification system would suggest that the silty sands have a very loose to loose density. Using classification test data from the samples obtained in the SPT holes, it was found that the mean grain sizes ranged from 0.12 to 0.25 mm, with a fines content of 5 to 30%. The fines were observed to have essentially negligible plasticity. Water contents ranged from 3.5% in the upper unsaturated layers to 28% in the soils below the water table.
Fig. 7.1. LOCATION AND GEOLOGY OF THE PEPPER'S FERRY TEST SITE
Fig. 7.2. BORING LOCATION PLAN AT PEPPER’S FERRY SITE
SPT vs. Depth

Fig. 7.3. SPT DATA AT PEPPER'S FERRY
7.2. TEST PROCEDURE

For this work, four CPT’s were performed, three with a standard cone, and one using large cone penetrometer. The cone penetrometers used for these tests have been described in the previous chapter with the exception of the large cone. The large cone is a subtraction type and was provided by Hogentogler Company of Maryland. It has a 15 $cm^2$ projected tip area and a 225 $cm^2$ sleeve area with a capacity of 15 tons. Fig. 7.4. presents a photograph of the three cone penetrometers used for this investigation. Locations of the tests are shown in Fig. 7.2.

The CPT’s were conducted using a drill rig with a hydraulic jacking system to push the cone into the ground. Before the cone was inserted, the tip was placed in a backfilled borehole on the ground surface for temperature equilibration. When the data acquisition system and the drillrig were ready, the penetrometer was then brought back to the surface and a zero-load reading of the tip bearing and the sleeve friction were recorded. The data were collected by a portable PC powered by a field generator. The data acquisition system was driven by depth pulses from a depth trigger to record digital data at 2 cm increments. Real time plots of tip resistance and sleeve friction were generated and displayed on the PC screen while the test was in progress. After the test, the cone was brought to the surface and a final zero load reading was recorded.

Individual test results were designated as CPT-std1, CPT-std2, and CPT-std3 for the standard cone. Similarly for the minicone tests and the large cone test, the attributes “min” and “lrg” were used instead of “std”. All tests were conducted in close proximity to each other except for test No. CPT-min4 that was performed close to the river in a sloping ground.
7.3. TEST RESULTS

Figs. 7.5 and 7.6 give cone test result profiles for both tip resistance and friction ratio. Fig. 7.5. was obtained by Dickenson, et al. (1988) using the minicone, and Fig. 7.6. was obtained using the standard cone during this investigation. Except for local areas, both cones show friction ratios between 0.5 and 1 %, typical values for silty sands. In each case, the cone tip resistance is highest in the upper 2 m, and decreases slightly thereafter. The average tip resistance for either CPT is about 40 bars.

Fig. 7.7 shows a comparison for the tip resistances of the three standard cone penetrations of this investigation. Except in the upper 1.5 m or so, the tip resistances are very similar. The average tip resistance in Fig. 7.7. is approximately 40 bars. A similar plot is made in Fig. 7.8. for the three CPT's reported by Dickenson et al. (1988) from the silty sand portion of the site. As with the data in Fig. 7.7, there is a good consistency in the tip resistances with an average of 40 bars.

7.4. ANALYSIS AND DISCUSSION

The cone penetration test induces complex changes in stresses and strains around the cone tip. Although comprehensive theoretical solutions of this problem have been attempted in the last fifteen years (Durgunoglu and Mitchell, 1975; Rohani and Baladi, 1981), none of them are applicable to all soil types. The interpretation of cone penetration data therefore relies heavily on empirical correlations to obtain the required geotechnical parameters.
Fig. 7.5. PEPPER'S FERRY CPT NO. MIN-2.
Fig. 7.8. PEPPER'S FERRY CPT NO. STD-2.
Fig. 7.7. DATA OF 3 STANDARD CONE TIP RESISTANCE
Fig. 7.8. DATA OF 3 MINI CONE TIP RESISTANCE
7.4.1. Soil type and soil profile.

The two SPT holes at the site allowed samples to be obtained and normal classification tests to be conducted. The CPT allows a soil to be classified based on the relative values of FR and $q_c$ parameter.

An example of the soil identification using the suggested chart by Robertson et al. (1984) of the test No. CPT Min-2 is shown in Fig. 7.5. The plot indicates that the soil type ranges from a sandy silt to sand with majority of points falling in the category of silty sand. All of the CPT test results followed the same pattern, and the results confirm those found from the SPT holes.

7.4.2. SPT vs. CPT Results

The SPT-liquefaction potential correlation is based on SPT measurements made at sites which subsequently liquefied during earthquakes. Knowledge of the magnitude and acceleration characterizing the actual earthquake (or artificial earthquake when using a shaking table) allowed development of the correlation of liquefaction resistance and the SPT-N values. Because of much less data existed for CPT measurements, investigations have attempted to relate the SPT-N value to the CPT tip resistance. In most cases, this involved assuming that there is a simple relationship between them. In this way, charts already available could be used to convert SPT blowcount into $q_c$. This has led an emphasis on effort to establish the ratio of $q_c/N$ (Muromachi and Kobayashi, 1980; Martin and Douglas, 1981; Robertson and Campanella, 1985; and Seed and de Alba, 1986) where $q_c$ is usually given in bars and N is blowcounts per foot. Perhaps the first such relationship was introduced by Meyerhof (1956) where he proposed that $q_c = 4N$ for fine sands. Later approaches altered this relationship for
Fig. 7.9. EXAMPLE OF PLOT OF TEST RESULTS FOR CLASSIFICATION PURPOSES
fines content or the mean grain size (Muromachi, 1981; Robertson and Campanella, 1985; and Seed and de Alba, 1986).

This $q_c/N$ ratio was evaluated for the Pepper’s Ferry data. The SPT values for this study were taken from borings A1 and C1 (Fig. 7.3). The mean diameter of the silty sand was taken from classification tests on spoon samples. Because of consistency in the results, the CPT data for this analysis was based on the average values of all three cones of different sizes. The $q_c/N$ of Pepper’s Ferry is plotted with depth (Fig. 7.10) and the values suggested by Seed et al. (1986) based on the mean grain size was superimposed in the figure. It is shown that at a depth less than 10 ft., the ratio is higher than was predicted by Seed et al. method. However, at a depth more than 10 ft., Seed et al. prediction is in close agreement to the Pepper’s Ferry data. The agreement shown is within the scatter given in the other correlations by Douglas (1982), Jamiolkowsky et al. (1985) and Farrar (1986).

7.4.4. Scale effect

The possibility of a scale effect in penetration tests using different sizes of cones has been addressed by De Beer (1963). While there is some uncertainty on this issue, it is generally thought that if the soil particles are small enough not to directly obstruct the cone, there is no scale effect. Some evidence for scale effects in calibration chambers, e.g. Last (1984) is now attributed to boundary effects (Eid, 1987). According to Schmertmann (1978), cone penetrometers with diameters of 5 to 40 cm² should have no scaling effect, however a small cone can be more sensitive to the presence of layers in a soil deposit, and if the grain size of the soil being penetrated is large relative to the cone tip, then the cone will over register and yield higher tip resistances. Schmertmann did not quantify how large the grain size of the soil would have to be to influence the results obtained with a small cone. In this study, the
Fig. 7.10. $\frac{q_c}{N}$ VARIATION WITH DEPTH AT PEPPER’S FERRY
ratios of the diameters of the cones to the mean grain size ($D_{50}$) of Pepper's Ferry silty sand were about 290, 240 and 160 for the large, standard and minicone respectively.

The three minicone test results (CPT-min1, CPT-min2 and CPT-min3) and the three standard cone test results of Pepper's Ferry site were averaged and superimposedly plotted in Fig. 7.11. and Fig. 7.12. Fig. 7.11. shows the cone tip resistances and Fig. 7.12. shows the friction ratios of the averaged standard cone, minicone and large cone results. These figures show that there was no appreciable effect of the cone size and for practical purposes, they yield the same results. The variations in the data of the tip resistances are likely due to the variation of densities of soil layers from one test hole to the other. On the other hand, the friction ratios show almost no differences.
Fig. 7.11. AVERAGE TIP RESISTANCES OF STANDARD CONE, MINICONE AND LARGE CONE
Fig. 7.12 AVERAGE FRICTION RATIOS OF STANDARD CONE, MINICONE AND LARGE CONE
8.1. INTRODUCTION

The purpose of this chapter is to examine the applicability of the available correlations with the test results obtained during the course of the research. Data from the calibration chamber is linked to the laboratory determined cyclic strength of Yatesville silty sand and the results are compared to the available correlation charts and other data of clean sand tested in the same calibration chamber. Analysis of the liquefaction potential at Pepper’s Ferry site is performed based on various earthquake magnitudes to determine the earthquake shaking that would cause liquefaction at this site. Also, the relationship between liquefaction and the state parameter is reviewed. A new correlation of the undrained steady state shear strength of silty sand to cone tip resistance is proposed.
8.2. EVALUATION BASED ON CYCLIC STRESS RATIO

The charts for liquefaction evaluation based on CPT developed by Robertson et al. (1985), Seed et al. (1986) and Shibata et al. (1988) have been generally supported by field data. To examine in more detail the applicability of these methods of predictions, two approaches were used.

(1) Examine the available correlations to the test results of Yatesville silty sand

(2) Use the available methods for liquefaction analysis at Pepper’s Ferry site.

8.2.1. Correlation of Cyclic Strength of Yatesville Silty Sand and the CPT.

The cyclic strength data for Yatesville silty sand were presented in chapter 5 and the characteristics of its cone penetration resistance were studied in chapter 6. Both results will be linked to determine the correlation of both quantities. To facilitate comparison of the available methods, an earthquake magnitude of 7.5 was chosen. The cyclic strength of Yatesville silty sand was determined from the laboratory data (Fig. 5.32.). The curve corresponding to the samples prepared from a slurry and OCR = 1 was representative of the calibration chamber samples. For an earthquake magnitude 7.5., Seed et al. (1984) suggested use of the cyclic strength corresponding to liquefaction at 15 cycles. From Fig. 5.32., this corresponds to a cyclic stress ratio, $\frac{\sigma_{cc}}{2\sigma'_c}$ of 0.235. A correction factor of 0.57 was applied to relate the laboratory cyclic triaxial test data to the field stress ratio in terms of $\frac{\tau}{\sigma'_{vo}}$ (Seed and Peacock, 1971). Hence a value of stress ratio, $\frac{\tau}{\sigma'_{vo}} = 0.134$ was used as the cyclic resistance of Yatesville silty sand under a 7.5 magnitude earthquake. This earthquake magnitude was chosen to facilitate the comparison to the available charts which are mostly based on a 7.5 magnitude earth-
quake. The range of void ratio of samples tested in the laboratory was 0.36 - 0.45 and the corresponding penetration resistance was found 30 bars (Fig. 6.11).

Fig. 8.1. shows chart incorporating the methods by Robertson & Campanella (1985), Seed and de Alba (1986) and Shibata and Terapaksa (1988). The curves proposed by Shibata et al. (1988) are very close to those of Robertson & Campanella. Curves of Shibata et al. and Seed et al. for $D_{50} = 0.1$ mm are essentially the same; however the curves proposed by Seed and de Alba (1986) generally fall to the left of the first two methods for bigger particle sizes. This means that for a similar value of cone tip resistance, the Seed and de Alba method predicts a higher cyclic strength of the soil than the other two methods. Test results for Yatesville silty sand plotted in the proposed zone of liquefaction by all three methods, and exactly on the boundary for $D_{50} = 0.1$ mm in the Seed et al. and Shibata et al. methods.

8.2.2. Comparison of The Tip Resistance of Silty Sand to Clean Sand at a Similar Cyclic Stress Ratio.

In order to compare the Yatesville silty sand data to clean sand, data for Monterey #0/30 is used for this purpose. This sand was studied by Sweeney (1987) for liquefaction analysis. The behavior of Monterey #0/30 under cyclic loading was presented in Fig. 5.35. as a results of study by Muzzy (1983) and Milestone (1985). The cyclic strength of Monterey #0/30 was related to the cone penetration data from the calibration chamber by Sweeney (1987) for an earthquake magnitude of 6.6. In this study, a similar method to that of Sweeney (1987) was also applied to relate the cone tip resistance and the cyclic stress ratio for an earthquake magnitude of 7.5. Again, 15 cycles was chosen to represent the specified earthquake magnitude. This enables a comparison of the tip resistance of a clean sand to a silty sand corresponding to a similar value of cyclic stress ratio. In a previous section, the cyclic strength of
Fig. 8.1. COMPARISON OF BOUNDARY CURVES FOR LIQUEFACTION POTENTIAL ANALYSIS
Yatesville silty sand was determined to be 0.134 at 15 cycles and this correspond to a tip resistance of 30 bars. For clean sand, the relationships of cyclic stress ratio to the cone tip resistance were evaluated following these steps:

1. The stress ratio was correlated against relative density based on data from Muzzy (1983) and Milstone (1985). Fig. 5.35. was used to produce Fig. 8.2.

2. The tip resistance was correlated with the relative density and vertical effective stress. This was done by Sweeney (1987) and is shown in Fig. 8.3. The tip resistance data was based on the minicone since it was not biased by the boundary conditions of the calibration chamber.

3. An equivalent relative density was taken to be comparable with similar cyclic strength of silty sand at 15 cycles which was determined in previous section equals to 0.134. Referring to Fig. 8.2., this correspond to 28% relative density in clean sand.

4. A comparable tip resistance of clean sand can be estimated based on this relative density at vertical effective stress = 1 ksc. In this step (Fig. 8.3.) a tip resistance = 85 ksc. was obtained to correspond to the specified cyclic stress ratio. Thus, it can be said that at similar cyclic strength and earthquake magnitude, the silty sand has critical tip resistance of 30 bars while that of clean sand shows 85 bars.

The results are also shown in Fig. 8.1. The data of Monterey #0/30 plot in the proper location in all three curves. It should be noted however that the methods proposed by Shibata et al. (1988) and Robertson et al. (1985) do not make a distinction for grain size over 0.25 mm while the Seed and de Alba method (1986) does.
Fig. 8.2. CORRELATION OF RELATIVE DENSITY AND STRESS RATIO FOR MONTEREY #0/30 AT 15 CYCLES
Fig. 8.3. CORRELATION OF TIP RESISTANCE AND OVERBURDEN PRESSURE FOR MONTEREY #0/30 SAND
8.2.3. Analysis of Liquefaction Potential at Pepper’s Ferry site

In this section, the grain size and the CPT data of Pepper’s Ferry are used to determine the magnitude of an earthquake that would cause liquefaction at this site based on methods discussed in chapter 2. According to the boundaries set by Tsucida (1970), the silty sand at Pepper’s Ferry can be classified as very liquefiable (Fig. 8.4.).

The CPT data were used in two ways. In the first one, the tip resistance and the friction ratio were plotted on the classification chart suggested by Robertson & Campanella (1985). This is shown in Fig. 7.5. where the soil was classified as silty sand. This classification also indicates that the soil is susceptible to liquefaction, agreeing with the classification based on gradation analysis. All of the CPT test results plotted in similar way indicating the same category of the type of soil shown in Fig. 7.5.

Secondly, the CPT data were used to determine the magnitude of earthquake that will cause liquefaction at the Pepper’s Ferry site based on methods the methods described in chapter 2. An average \( q_{c1} = 44 \) bars was determined from the average tip resistance shown in Fig. 7.8. Some steps were needed to analyze the liquefaction potential of this site at various different earthquake magnitudes.

1. An average \( D_{50} = 0.18 \) mm was used for Pepper’s Ferry silty sand based on Borings A1 and C1 (Fig. 7.7.).

2. A curve was established corresponding to this mean grain size suggested by charts of Robertson & Campanella (1985) and Seed & de Alba (1986) by interpolation of the corresponding curves of different grainsizes. For use of the Shibata et al. method, this is taken care of by using the correction factor \( C_2 = D_{50}/0.25 \).

3. The curves were expanded for different earthquake magnitudes using the adjustment
Fig. 8.4. RANGE OF SOIL GRADATION OF PEPPER'S FERRY SITE FROM BORING A1 AND C1
factors suggested by Seed and Idriss (1982) that is shown in Fig. 8.5. As a result, a family of curves applicable for the particular mean grain size and for different earthquake magnitudes were established.

(4) The cyclic stress ratio were calculated corresponding to a certain maximum surface acceleration using equation 2.3. In this analysis, an $a_{max} = 0.15$ g was applied. To determine the values of $\sigma_0$ and $\sigma_0'$, the water table was assumed at 12 ft. depth, and the evaluation was performed at 20 ft.

(5) For the corresponding stress ratio, the values of critical tip resistances were read from the curves established in step (3). For the case of Shibata et al. method, the stress ratio was calculated by equation 2.13 and correspondingly the critical tip resistance can be obtained by using equation 2.21.

(6) The results of calculations were plotted in terms of $q_{ct-crit}$ against the earthquake magnitudes. The average tip resistance of Pepper's Ferry site is also shown in the figure. Liquefaction was determined when $q_c < q_{ct-crit}$.

The results of the analysis are shown in Fig. 8.6, where three curves of critical cone tip resistance to allow liquefaction are plotted vs. earthquake magnitude as determined by three different methods: Robertson & Campanella (1985), Seed & de Alba (1986) and Shibata et al. (1988). The average tip resistance for Pepper's Ferry site is compared to the three curves in Fig. 8.6., leading to the conclusions: (1). The Seed & de Alba critical tip resistance is lower than the other two methods; (2). The Robertson & Campanella method and Shibata et al. method predict that Pepper's Ferry site would liquefy at an earthquake magnitude of about 6.8; and (3). According to Seed & de Alba method, the site would not liquefy even at an earthquake magnitude of 8.5.
Fig. 8.5. CORRECTION FOR DIFFERENT EARTHQUAKE MAGNITUDE
Fig. 8.6. LIQUEFACTION POTENTIAL OF PEPPER'S FERRY SITE AT DIFFERENT EARTHQUAKE MAGNITUDES
8.3. EVALUATION BASED ON THE STATE PARAMETER

In a technical memorandum from the Public Works Research Institute of Japan (PWRI), Farrar (1986) presented a method of correlating the cyclic stress ratio of sandy materials with the state parameter as defined by Been and Jefferies (1985). The state parameter was evaluated based on the equation proposed earlier by Been et al. (1986) as follows:

\[
\psi = -0.092 \ln \left( \frac{q_c - \sigma^m}{\sigma^m} + 17 \right) / 31 \quad \text{[eq. 8.1.]} 
\]

It should be noted that equation 8.1 does not take into account the slope of the steady state line, \( \lambda_{ss} \). To develop a correlation in terms of the state parameter, the \( q_{c1} \) versus cyclic shear strength relationship was used. Farrar (1986) used \( q_{c1} \) for clean sands from the chart proposed by Seed and converted them to state parameters using eq. 8.1 assuming a \( K_s \) values of 0.5, with a typical sand unit weight and a ground water location at the ground surface. Then, the state parameter points calculated in SPT intervals were plotted with liquefaction occurrence as predicted by SPT \( N_{(60)} \). The results of Farrar’s analysis is shown in Fig. 8.7. His findings suggest that there is a relationship between the state parameter and occurrence of liquefaction. Soils with positive state parameter are considered prone to liquefaction. However, it is also possible to liquefy a soil even with a moderately negative state parameter.

The calibration chamber samples had a range of state parameter corresponding to cyclic stress ratio of 0.134 as discussed in section 8.2.1. These data are plotted in Fig. 8.7 and it can be shown that the data fall in the liquefiable area. For Pepper’s Ferry silty sand, the state parameter was calculated using equation 8.1 and related to the cyclic stress ratio obtained in the laboratory assuming that the range of void ratios in the laboratory is similar to that in the field. The cyclic stress ratio of Pepper’s Ferry silty sand was determined to be 0.185 Fig. 5.33, where at 15 cycles and with a correction factor of \( C_r = 0.57 \) yield a value of \( \tau/\sigma^m = \)
Fig. 8.7. STATE PARAMETER AND THE OCCURRENCE OF LIQUEFACTION (Farrar, 1986)
The state parameter of Pepper's Ferry site was calculated at a depth of 20 ft.
assuming water table at 12 ft. and a $K_s = 0.5$. Substituting all quantities into equation 8.1.
give state parameter, $\psi$ equals to -0.11. The Pepper's Ferry data falls outside the liquefiable zone.

A similar correlation presented in Fig. 8.8. using the data published by Shibata et al. (1988).
To convert the cone tip resistance into the state parameter, equation 2.23. This equation requires the value of the slope of the SSL, $\lambda_{ss}$ and the effective and total horizontal stresses be known. The horizontal stresses were calculated based on a $K_s$ value of 0.5. However, there was no information of the values of, $\lambda_{ss}$ in the published literature. To estimate the slope of SSL, it was assumed that the slope, $\lambda_{ss}$ was a function only of the mean grain size of the soil. For sands with $D_{50} \geq 0.25 mm$, $\lambda_{ss}$ equals to 0.05 was assumed and for silty sands with $D_{50} < 0.25 mm$, a value of $\lambda_{ss} = 0.1$ was assumed. However, as has been previously discussed, this slope is influenced not only by the mean grain size but also by the shape of the particles and the mineralogy. Hence the use of this relationship is only tentative until further investigations. For this analysis, this simple assumption is regarded as being sufficient.

Only data of liquified cases were considered. Results of the calculations were tabulated in Table D.1 and plotted in Fig. 8.8. It was assumed that soils with $D_{50} < 0.15 mm$ are silty sands and soils with $D_{50} \geq$ are clean sands. The figure suggest that in terms of state parameter, there is not much distinction of the cyclic strength of clean sands and silty sands.

Test results from Pepper's Ferry and results of the calibration chamber tests are also incorporated in this Figure. The same values of state parameters and cyclic strength ratios are used for Yatesville silty sand. But in this plot, the state parameter of Pepper's Ferry site was calculated using equation 2.23. instead of equation 8.1. In eq. 2.23., the slope of the SSL, $\lambda_{ss}$, is incorporated. The calculation yields a state parameter of -0.17, which is more negative than the previous one. It is interesting to note that all the calculated state parameters in Table 8.1.
Fig. 8.8. PROPOSED CHART FOR LIQUEFACTION EVALUATION BASED ON STATE PARAMETER VALUES.
were negative, thus this agree with Farrar (1986) conclusion that even at a negative state, the soil deposit still had tendency to liquefy. This implies that when using state parameter for liquefaction evaluation, the conclusion based on pore pressure generation concept could be contradictory with that based on steady state shear strength concept when the state parameter of the soil falls between -0.30 to 0.

8.4. EVALUATION BASED ON THE STEADY STATE SHEAR STRENGTH

De Alba and Seed (1987) and Poulos (1988) have suggested correlations of undrained shear strength of the soil with the normalized N-SPT values. The undrained shear strength was back calculated from earthquake induced failures. Fig. 2.15 and 2.16 show these correlations. The correlation presented by Poulos shows much higher undrained shear for a given N-SPT than that presented by de Alba and Seed (1987).

One of the goals of this research was to develop a similar correlation to that given for SPT and the residual shear strength using CPT results. The Virginia Tech calibration chamber is a unique tool which can be used for this purpose. The undrained steady state shear strength for Yatesville silty sand from triaxial tests are readily available for a range of void ratios (Fig. 5.6.) and the cone tip resistance data can be correlated conveniently to these values based on the corresponding void ratios measured in the calibration chamber specimens. The tip resistance was normalized using a $C_v$ value such as suggested by Seed et al. (1982) or by the mean effective stress, $\sigma_{\text{eff}}$, such as suggested by Been and Jefferies (1986). These correlations are shown in Fig. 8.9. and 8.10. There is a wide scatter of the undrained steady state shear strength data due to variation in void ratio in the calibration chamber and due to the fact that $S_{\text{cm}}$ values are very sensitive to the change in void ratio. The figures suggest that at a lower cone tip resistance (normalized cone tip resistance less than 20, the steady state shear
strength increases, however it stays about the same for higher normalized cone tip resist-
ances.

In Fig. 8.9., data from Poulos (1988) based on SPT and $S_{uw}$ measured in laboratory are also
shown. These data are incorporated in Fig. 8.9. by converting the SPT to CPT using a factor
of $\frac{q_c}{N} = 3.5$. Data from Poulos (1988), which is based on laboratory measured steady state
undrained shear strength plot in the same scatter as the Yatesville silty sand. The range of
case histories data from de Alba and Seed (1987) is also plotted in Fig. 8.9. by converting the
SPT into CPT by the same factor. However, since de Alba and Seed's chart used an equiv-
alent clean sand $N_t$, their $N_t$ values were adjusted first to be comparable with the silty sand
data by substracting a $\Delta N_t$ value of 4 such as suggested in Table 2.2. The residual shear
strength of de Alba and Seed (1987) were assumed analogous to the steady state shear
strength, $S_{uw}$. These residual shear strength were back analyzed from various case histories.
It is shown in this figure, that the laboratory measured $S_{uw}$ is much higher than the 'field' $S_{uw}$.
The difference is as high as 2 to 5 times higher for 'field' $S_{uw}$. It is also interesting to note that
Hsing and Seed (1988) reported the same issue concerning this. Their laboratory measured
$S_{uw}$ on the lower San Fernando Dam silty sands also shows 2 to 5 times higher than the resi-
dual shear strength back analyzed from stability analysis.

Alternatively, similar correlations were also established using the normalized tip resistance
as well as normalised steady state shear strength of the soil (Fig. 8.11. and 8.12). To nor-
malize $S_{uw}$, the mean effective stress, $\sigma_m'$ was used. The dashed lines represent the range of
the data. It has to be noted that unlike the chart by de Alba and Seed (1987), the tip resistance
in Fig. 8.9. - 8.12. were not corrected for fines content. So the applicability of this chart is
limited only to silty sands of similar properties with the silty sand tested.
Fig. 8.9. UNDRAINED STEADY STATE SHEAR STRENGTH vs $q_c$
YATESVILLE SILTY SAND
FINES CONTENT = 40%

Fig 8.10. UNDRAINED STEADY STATE SHEAR STRENGTH vs NORMALISED CONE TIP RESISTANCE
Fig. 8.11 Normalised Undrained Steady State Shear Strength vs \( q_{c1} \)

YATESVILLE SILTY SAND
FINES CONTENT = 40%
Fig. 8.12 NORMALISED UNDRAINED STEADY STATE SHEAR STRENGTH vs NORMALISED CONE TIP RESISTANCE

YATESVILLE SILTY SAND
FINES CONTENT = 40%
9.1. SUMMARY

Earthquake induced soil behavior has been viewed from two different aspects. The first focuses on the generation of pore pressures that can ultimately transform the soil into fluid-like state known as liquefaction. The basic mechanism of liquefaction according to this view is the progressive development of excess pore water pressure due to the application of cyclic shear stresses induced by the upward propagation of shear waves from the underlying rock formation. Two approaches were addressed in chapter 2. The first one uses relative density as a measure of the liquefaction potential which is represented by the cyclic stress ratio (Seed and Idriss, 1982; Robertson and Campanella, 1985; Ishihara, 1985; Seed and de Alba, 1986; and Shibata and Terapaksa, 1988) and the other approach is based on assumption that the pore pressure generation during cyclic loading is controlled by the magnitude of the cyclic shear strain, thus, shear modulus is of more concern (Dobry et al., 1982).

The second aspect is the condition to cause flow failure of liquefied soil under the action of the driving shear stresses. In this case, the controlling soil parameter is the undrained steady state shear strength, $S_{us}$. Liquefaction failure happens if the existing shear stresses, $\tau$, is
higher than $S_{0}$. The role of the earthquake is to trigger this failure and the pore pressure generated by the earthquake push the soil state closer to the steady state condition. This method has been suggested by Poulos et al. (1985).

An alternative way of evaluating the liquefaction potential of a soil deposit is by measuring the state parameter of the soil in the ground. Using the steady state line as reference, one can measure the proximity of the state of soil element to this line which is termed the state parameter. Been and Jefferies (1986) correlated the cone penetration test results and the state parameter based on large body of CPT data in the calibration chamber. This parameter could be related to the behavior of the material during shear and has been used to evaluate the liquefaction potential of a soil deposit.

Variability in natural soil deposits and the difficulty associated with taking undisturbed samples of loose saturated cohesionless soil have caused the practicing engineers to rely more on the results of in situ tests, usually either the more common SPT or the CPT. The SPT has been used due to the availability of a large data base to correlate with liquefaction performance. However, due to the problems inherent in the SPT, the CPT is gaining more attention as a tool to provide a more continuous information of the soil profile.

An important issue of the gap in knowledge of evaluating liquefaction potential by in situ tests is the influence of fines content. As the fines content increases, the cone penetration resistance drops significantly without changing much the liquefaction resistance. Direct correlations established from the performance during earthquake is not sufficient due to the limited number of data. Efforts to establish correlation of the cone penetration resistance with the laboratory test results have had problems due to the sample disturbance. To overcome this situation, the use of calibration chamber was proposed to remove the uncertainty of the in situ soil density and the stress or strain boundaries.

SUMMARY AND CONCLUSIONS
This research has focused on the use of the calibration chamber for evaluating the cone penetration resistance of silty sand containing significant amount of fines. An important element in the success of the calibration chamber tests is the fabrication of uniform sample. Three factors were considered in the fabrication technique. The sample has to be uniform, it has to represent as close as possible the soil fabric in situ, and it has to be applicable both in small scale (triaxial specimen) and in large scale (calibration chamber specimen). Based on this investigation, consolidation from a slurry was selected for fabricating the sample in the calibration chamber as well as in the small scale triaxial samples to produce soil specimens having mechanically similar characteristics.

A series of laboratory tests were conducted to study the soil behavior under monotonic and cyclic loading. In all, 15 isotropically consolidated triaxial tests were conducted on Yatesville and Peppers Ferry silty sands; 33 cyclic triaxial tests were conducted on Yatesville silty sand including 22 tests with OCR = 1 and 9 tests with OCR = 2, of which the samples were fabricated by consolidation from a slurry. To investigate the effect of soil fabric, 2 samples were prepared by compaction. For Peppers Ferry silty sand, 9 cyclic triaxial tests were conducted with all having OCR = 1 and fabricated by consolidation of slurry.

Study of the cone penetration resistance of silty sands was conducted in the calibration chamber and at the Peppers Ferry site. The use of the calibration chamber has the advantage over in situ test conditions in removing the uncertainty for measurements of soil density and determination of the boundary conditions. 23 cone penetration tests were conducted using both minicone and the standard cone. About 6000 kg of slurry was mixed and consolidated for each test; and then after the test, it was carefully excavated and the densities were measured. The excavated material was then reused for the following test. In this way, the soil was the same for a series of the calibration chamber tests.

The study of the cone penetration resistance at the Pepper's Ferry site was performed as a complement to the data obtained in the calibration chamber. The field cone penetration tests...
consisted of 4 minicone tests, 3 standard cone tests and one CPT using a 15 cm² cone penetrometer. One borehole was supplemented with SPT and samples were taken to study their properties in the laboratory. In addition, the SPT data from Yatesville dam are presented for comparison.

The last step in this research was to link the cone penetration resistance of silty sands to their cyclic and static behavior determined in the laboratory. Data from the calibration chamber tests were compared with the available correlation charts and used to develop a new relationship of the CPT and the steady state shear strength of silty sand. CPT data from Pepper’s Ferry were utilized for liquefaction potential analysis of the site.

9.2. CONCLUSIONS

(1). Laboratory experiments showed that pluviation of silty sand with significant amounts of fines is not suitable for fabrication of triaxial specimen or calibration chamber specimen. A pluviated sample is very loose and does not represent the normal range of density in the field. Consolidation can be regarded as being the best way of obtaining a uniform saturated sample of silty sand for a large scale. This method is similar to the natural alluvial deposition process and applicable to both triaxial specimens and calibration chamber specimens. The only demerit of this technique is that extensive time is needed for the completion of the consolidation.

(2). Laboratory studies on the behavior of Yatesville and Pepper’s Ferry silty sands were conducted using monotonic and cyclic loading triaxial tests. The cyclic triaxial tests were performed to find the parameters relating to the cyclic resistance of the soils, and to study the effects of fines as opposed to the clean sands. Consolidated undrained triaxial tests were conducted to measure the steady state shear strength and related parameters, and to inves-
tigate the significance of the state parameter on the behavior of silty sands. Several conclu-
sions can be drawn from the laboratory study:

2.1. The cyclic strength of silty sands is similar to those of clean sand. The effects of non-
plastic fines are not significant.

2.2. Sample preparation influences the cyclic strength of the material. Samples prepared
by compaction and overconsolidated samples show higher resistance to liquefaction. At
cyclic stress ratio of 0.3, Yatesville silty sand liquefied at 10 cycles for samples prepared
from slurry with OCR = 1. At OCR = 2, the corresponding number of cycles to liquefaction
increases to 19 cycles and a compacted sample liquefies at 43 cycles. These findings
confirmed previous conclusions drawn by other researchers (Ladd, 1974; Mulilis et al.,

2.3. The steady state line (SSL) is a unique property of cohesionless material including
sands and silty sands (Castro, 1969; Poulos, 1981; Castro et al., 1982; Mohammad and
Dobry, 1984). Test results on Yatesville silty sand, Pepper’s Ferry silty sand and two other
silty sands from the Lower San Fernando Dams show that sample preparation does not
affect the SSL. According to Castro, the slope of the steady state line, \( \lambda_{ss} \), is related to the
compressibility, particle shape, and the mineralogy of the soil. The slope for silty sands
show a higher values of \( \lambda_{ss} \) than one would expect for a clean sand. This confirms findings
by other researchers that finer material normally exhibit a steeper SSL.

2.4. Results from the laboratory study suggest that the state parameter is a good means
for normalizing the behavior of silty sands. The stress strain behavior of all silty sands
investigated strongly correlated with the state parameter, \( \psi \). Been et al. (1985) have sug-
gested similar conclusions. In this study, it was found that the normalized steady state
shear strength and the generated pore pressure during shear also depend on the state

SUMMARY AND CONCLUSIONS
parameter. These findings have important implications in that the steady state shear strength could be predicted by means of the state parameter.

2.5. Observation on the dependency of the stress strain behavior of the silty sands investigated lead to a conclusion that samples at state parameters of 0.0 to 0.03 may exhibit contractive or dilative behavior.

(3). Conclusions which may be drawn from the study in the calibration chamber include:

3.1. Cone tip resistances were very low (5 - 30 bars), presumably due to the undrained conditions generated by the presence of fines.

3.2. The normalized tip resistance is a function of the state parameter. This conclusion has been suggested by Been and Jefferies (1986) and Jefferies (1988); however, data from the calibration chamber for Yatesville silty sand shows a steeper slope on the normalized qc vs. ψ plot than that suggested by Been et al. (1986).

3.3. There is no significant effect of the overburden pressure on the tip resistance of Yatesville silty sand. This may be attributed to the relatively low permeability of the material compared to clean sand. The behavior of the silty sand during penetration is either fully undrained or partially drained. A quantification of this aspect is recommended for further research.

3.4. A comparison of the CPT tip resistances in the calibration chamber to that of the SPT in the field on Yatesville silty sand shows a high degree of consistency. This support the idea that the calibration chamber successfully approximated field conditions.

(4). Field CPT results show that the use of the different sizes of cones do not affect the results and that CPT provided repeatable and reliable test results. Liquefaction potential analyses using the Robertson and Campanella (1985) and Shibata et al. (1988) methods for the Pepper's
Ferry site predicted that the site would liquefy at an earthquake magnitude of 6.8. A comparable analysis using the Seed and de Alba approach (1986) suggested the site would not liquefy even at an earthquake magnitude of 8.5.

(5). The correlation of the liquefaction resistance to the state parameter based on field data from Shibata and Terapaksa (1988) was reviewed. It was found that soils at moderately negative state still possess the potential to liquefy.

(6). A correlation of the steady state shear strength to the CPT was attempted. The correlation has a trend similar to that suggested by Poulos (1988). However, there is much scatter in the correlation, possibly due to the variations in the void ratio. The undrained steady state shear strength is very sensitive to changes in void ratio. Consequently, this correlation needs to be refined. The correlation of the CPT to $S_\text{u}$ as measured in the laboratory shows higher values than that of $S_\text{u}$ back analyzed from the liquefaction flow failure case histories such that presented by de Alba and Seed (1987). This matter needs further research.


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Appendix A. ICU TRIAXIAL TESTS
### TABLE A.1.

**TEST CONDITIONS BEFORE AND AFTER CONSOLIDATION STAGE**

<table>
<thead>
<tr>
<th>Test#</th>
<th>$\sigma'_c$ (psi)</th>
<th>$e_o$</th>
<th>$e_c$</th>
<th>$\gamma_d$ (pcf)</th>
<th>$m_{vi} \times 10^{-4}$ (psi⁻¹)</th>
<th>$t_{50}$ (min)</th>
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<tbody>
<tr>
<td>YSL-10</td>
<td>10</td>
<td>0.4722</td>
<td>0.4604</td>
<td>113.17</td>
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<td>e</td>
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<td>$m_{v_i}$ (10^{-4}) (psi⁻¹)</td>
<td>$t_{50}$ (min)</td>
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<td>7.31</td>
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### TABLE A.3. RESULTS OF ICU TRIAXIAL TESTS OF BM-3 SAN FERNANDO DAM (Hsing & Seed, 1988)

<table>
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<tr>
<th>Test #</th>
<th>$e_c$</th>
<th>$\sigma_{m}'$ (psi)</th>
<th>$\sigma_{3r}'$ (psi)</th>
<th>$S_{us}$ (psi)</th>
<th>$P_{ss}$ (psi)</th>
<th>$q_{ss}$ (psi)</th>
<th>$\phi_s$ (deg)</th>
<th>state $\psi$</th>
<th>Behavior</th>
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<td>BS-3.1</td>
<td>0.655</td>
<td>65.2</td>
<td>19.3</td>
<td>21.9</td>
<td>46.3</td>
<td>27.0</td>
<td>35.7</td>
<td>0.039</td>
<td>A</td>
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<td>BS-3.2</td>
<td>0.627</td>
<td>65.0</td>
<td>30.8</td>
<td>36.7</td>
<td>76.6</td>
<td>45.8</td>
<td>36.7</td>
<td>0.011</td>
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<td>BS-3.3</td>
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<td>BS-3.4</td>
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**LEGEND:**
A: CONTRACTIVE BEHAVIOR WITH A PEAK SHEAR STRESS PRIOR TO STEADY STATE
B: CONTRACTIVE BEHAVIOR WITH NO PEAK SHEAR STRESS PRIOR TO STEADY STATE
C: DILATIVE BEHAVIOR WITH A PEAK SHEAR STRESS PRIOR TO STEADY STATE
D: DILATIVE BEHAVIOR WITH NO PEAK SHEAR STRESS PRIOR TO STEADY STATE
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Table A.5. RESULTS OF ICU TESTS ON SACRAMENTO RIVER SAND (Hsing and Seed, 1988)

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TABLE A.6. RESULTS OF ICU TRIAXIAL TESTS OF BANDING SAND (Castro et al., 1982)

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D : DILATIVE BEHAVIOR WITH NO PEAK SHEAR STRESS PRIOR TO STEADY STATE
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<td>5.9</td>
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<td>6.7</td>
<td>27.4</td>
<td>0.004</td>
<td>A</td>
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</tbody>
</table>

**LEGEND:**

A : CONDUCTIVE BEHAVIOR WITH A PEAK SHEAR STRESS PRIOR TO STEADY STATE
B : CONDUCTIVE BEHAVIOR WITH NO PEAK SHEAR STRESS PRIOR TO STEADY STATE
C : DILATIVE BEHAVIOR WITH A PEAK SHEAR STRESS PRIOR TO STEADY STATE
D : DILATIVE BEHAVIOR WITH NO PEAK SHEAR STRESS PRIOR TO STEADY STATE
TEST NO: YSL-10

MATERIAL = YATESVILLE SILTY SAND

CONSOLIDATION PRESSURE, $\sigma_c$ = 10 psi

DRY DENSITY, $\gamma_d$ = 113.17pcf.

INITIAL VOID RATIO, $e_0$ = 0.4722

VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.4604

STRAIN RATE = 4.41 $\%$/hr.

STATE PARAMETER, $\psi$ = -0.0308

Fig.A.1. ICU TRIAXIAL TEST No. YSL-10
TEST NO : YSL-20

MATERIAL = YATESVILLE S11-T

CONSOLIDATION PRESSURE, \( \sigma_c \) = 20 psi
DRY DENSITY, \( \gamma_d \) = 113.45 pcf.
INITIAL VOID RATIO, \( e_0 \) = 0.4477
VOID RATIO AFTER CONSOLIDATION, \( e_c \) = 0.4192
STRAIN RATE = 4.35 %/hr.

STATE PARAMETER, \( \psi \) = -0.0213

Fig.A.2. ICU TRIAXIAL TEST No. YSL-20
TEST NO: YSL-30

MATERIAL = YATESVILLE SILTY SAND

CONSOLIDATION PRESSURE, $\sigma_c$ = 30 psi

DRY DENSITY, $\gamma_d$ = 113.39 pcf.

INITIAL VOID RATIO, $e_i$ = 0.4693

VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.4346

STRAIN RATE = 4.77 %/hr.

STATE PARAMETER, $\psi$ = +0.0237

Fig.A.3. ICU TRIAXIAL TEST No. YSL-30
TEST NO: YSL-40

MATERIAL = YATESVILLE SILTY SAND
CONSOLIDATION PRESSURE, $\sigma_c$ = 40 psi
DRY DENSITY, $\gamma_d$ = 116.86 pcf.
INITIAL VOID RATIO, $e_i$ = 0.4257
VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.3865
STRAIN RATE = 5.01 %/hr.
STATE PARAMETER, $\psi$ = -0.0034

Fig. A.4. ICU TRIAXIAL TEST No. YSL-40
TEST NO : YCOM-10

MATERIAL = YATESVILLE SILTY SAND
CONSOLIDATION PRESSURE, $\sigma_c$ = 10 psi
DRY DENSITY, $\gamma_d$ = 117.77 pcf.
INITIAL VOID RATIO, $e_0$ = 0.4147
VOID RATIO AFTER CONSOLIDATION, $e' = 0.4041$
STRAIN RATE = 5.31 %/hr
STATE PARAMETER, $\psi$ = -0.0871

Fig.A.5. ICU TRIAXIAL TEST No. YCOM-10
TEST NO: YCOM-15

MATERIAL = YATESVILLE SILTY SAND

CONSOLIDATION PRESSURE, $\sigma_c$ = 15 psi

DRY DENSITY, $\gamma_d$ = 107.1 pcf.

INITIAL VOID RATIO, $e_i$ = 0.5518

VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.5401

STRAIN RATE = 5.31

STATE PARAMETER, $\psi$ = +0.0785
TEST NO : YCOM-30

MATERIAL = YATESVILLE SILTY SAND
CONsolidation Pressure, $\sigma_c$ = 30 psi
Dry density, $\gamma_d$ = 113.91 pcf.
Initial void ratio, $e_i$ = 0.4626
Void ratio after consolidation, $e_f$ = 0.4290
Strain rate = 5.31 %/hr.
State parameter, $\psi$ = +0.0181
TEST NO: YCOM-30A

MATERIAL = YATESVILLE SILTY SAND
CONsolidation pressure, \( \sigma_c \) = 30 psi
Dry density, \( \gamma_d \) = 111.78 pcf.
Initial void ratio, \( e_0 \) = 0.4905
Void ratio after consolidation, \( e_e \) = 0.4499
Strain rate = 5.31 \%/hr.
State parameter, \( \psi \) = +0.0381
TEST NO : PSL-10

MATERIAL = PEP. FERRY SILTY SAND
CONSOLIDATION PRESSURE, $\sigma_c$ = 10 psi
DRY DENSITY, $\gamma_d$ = 105.23 pcf.
INITIAL VOID RATIO, $e_0$ = 0.5832
VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.5741
STRAIN RATE = 7.05 \%/hr
STATE PARAMETER, $\psi$ = -0.1160

Fig.A.9. ICU TRIAXIAL TEST No. PSL-10
TEST NO: PSL-30

MATERIAL = PEP. FERRY SILTY SAND

CONSOLIDATION PRESSURE, $\sigma_c$ = 30 psi

DRY DENSITY, $\gamma_d$ = 102.7 pcf.

INITIAL VOID RATIO, $e_o$ = 0.6228

VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.5843

STRAIN RATE = 7.21 %/hr

STATE PARAMETER, $\psi$ = -0.0421

Fig.A10. ICU TRIAXIAL TEST No. PSL-30
TEST NO: PCOM-10

MATERIAL = PEP. FERRY SILTY SAND
CONSOLIDATION PRESSURE, $\sigma_c$ = 10 psi
DRY DENSITY, $\gamma_d$ = 100.4 pcf.
INITIAL VOID RATIO, $e_0$ = 0.6426
VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.6286
STRAIN RATE = 8.64 %/hr.
STATE PARAMETER, $\psi$ = -0.0615

Fig.A.11. ICU TRIAXIAL TEST No. PCOM-10
TEST NO : PCOM-20

MATERIAL = PEP. FERRY SILTY SAND
CONSOLIDATION PRESSURE, $\sigma_c$ = 20 psi
DRY DENSITY, $\gamma_d$ = 100.33 pcf.
INITIAL VOID RATIO, $e_i$ = 0.6606
VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.6387
STRAIN RATE = 8.63 %/hr.
STATE PARAMETER, $\psi$ = -0.0112

Fig.A.12. ICU TRIAXIAL TEST No. PCOM-20
TEST NO: PCOM-30

MATERIAL
= PEP, FERRY SILTY SAND

CONSOLIDATION PRESSURE, $\sigma_c$
= 30 psi

DRY DENSITY, $\gamma_d$
= 103.49 pcf.

INITIAL VOID RATIO, $e_i$
= 0.6099

VOID RATIO AFTER CONSOLIDATION, $e_c$
= 0.5851

STRAIN RATE
= 8.59 %/hr.

STATE PARAMETER, $\psi$
= -0.0412

Fig.A.13. ICU TRIAXIAL TEST No. PCOM-30
TEST NO : PCOM-35

MATERIAL = PEP. FERRY SILTY SAND
CONSOlidATION PRESSURE, $\sigma_c$ = 35 psi
DRY DENSITY, $\gamma_d$ = 103.76 pcf.
INITIAL VOID RATIO, $e_v$ = 0.6057
VOID RATIO AFTER CONSOLIDATION, $e_c$ = 0.5803
STRAIN RATE = 8.51 %/hr.
STATE PARAMETER, $\psi$ = -0.0371

Fig. A.14. ICU TRIAXIAL TEST No. PCOM-35
TEST NO: PCOM-40

MATERIAL = PEP. FERRY SILTY SAND

CONSOLIDATION PRESSURE, \( \sigma_c \) = 40 psi

DRY DENSITY, \( \gamma_d \) = 102.58 pcf.

INITIAL VOID RATIO, \( e_i \) = 0.6242

VOID RATIO AFTER CONSOLIDATION, \( e_c \) = 0.5767

STRAIN RATE = 8.27 %/hr.

STATE PARAMETER, \( \psi \) = -0.0323

Fig. A.15. ICU TRIAXIAL TEST No. PCOM-40
Fig. A.16. $S_{us}$ vs $\sigma_{3r}'$ FOR YATESVILLE SILTY SAND
Fig. A.17. $S_u$ vs $\sigma_{3r}$ for Pepper's Ferry Silty Sand.
Fig. A.18. $S_u$ vs $\sigma_{3r}'$ FOR BATCHMIX NO. 3, LOWER SAN FERNANDO DAM
Fig. A.19.a. $S_{sa}$ vs $\sigma_{3r}$ FOR BATCHMIX NO. 7, LOWER SAN FERNANDO DAM
Fig. A.19.b. $S_{us}$ vs $\sigma'_{3r}$ for Batchmix No. 7, LOWER SAN FERNANDO DAM

DATA FROM HSING & SEED, 1988
Fig. A.20. $S_u$ vs. $\sigma'_3$ for undisturbed samples, of Lower San Fernando Dam
Fig. A.21 STEADY STATE ENVELOPE FOR BATCH MIX NO. 3, SAN FERNANDO DAM

DATA FROM HSING & SEED, 1988

SAN FERNANDO DAM
BULK SAMPLE NO. 3

ALL SAMPLE PREPARED BY
MOIST TAMMING AND
ISOTROPICALLY CONSOLIDATED
Fig. A 22 STEADY STATE ENVELOPE FOR BATCH MIX No. 7, SAN FERNANDO DAM

DATA FROM HSING & SEED, 1988

SAN FERNANDO DAM
BULK SAMPLE NO. 7

△ MOIST TAMPING
▲ WET PLUVIATION
Fig. A.23. STEADY STATE ENVELOPE FOR BANDING #6 SAND (Castro et al., 1982)
Fig. A.24. STEADY STATE ENVELOPE FOR SACRAMENTO SAND (Hsing & Seed, 1988)
Appendix B. CPT IN THE CALIBRATION CHAMBER
Table B.1. Compression and Modulus of Yatesville Silty Sand in the Calibration Chamber

<table>
<thead>
<tr>
<th>specimen</th>
<th>$\sigma'_e$ (kpa)</th>
<th>$\Delta H$ (cm)</th>
<th>$\frac{\Delta V}{V_o}$</th>
<th>$m_{nl} \times 10^{-3}$ (psi$^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CC-1</td>
<td>100</td>
<td>1.89</td>
<td>0.038</td>
<td>2.65</td>
</tr>
<tr>
<td>CC-2</td>
<td>70</td>
<td>2.50</td>
<td>0.051</td>
<td>5.01</td>
</tr>
<tr>
<td>CC-3</td>
<td>140</td>
<td>3.10</td>
<td>0.064</td>
<td>3.15</td>
</tr>
<tr>
<td>CC-4</td>
<td>70</td>
<td>2.20</td>
<td>0.044</td>
<td>4.38</td>
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<td>CC-5</td>
<td>100</td>
<td>1.60</td>
<td>0.033</td>
<td>2.26</td>
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</table>
Table B.1. CALIBRATION CHAMBER SPECIMEN NO. CC-1

\( \sigma_e = 100 \text{kPa} \)

<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>sample no.</th>
<th>( \gamma_d ) (pcf)</th>
<th>e (%)</th>
<th>w (%)</th>
<th>Sr (%)</th>
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<tbody>
<tr>
<td>0 - 1</td>
<td>C11.1</td>
<td>114.8</td>
<td>0.4515</td>
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<td>0.4320</td>
<td>12.2</td>
<td>75.4</td>
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<td>0.4412</td>
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<td>C12.1</td>
<td>114.7</td>
<td>0.4522</td>
<td>15.7</td>
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</tr>
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<td></td>
<td>C12.2</td>
<td>114.3</td>
<td>0.4578</td>
<td>16.8</td>
<td>97.8</td>
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<td>115.5</td>
<td>0.4421</td>
<td>15.9</td>
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<td>0.4711</td>
<td>17.4</td>
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<td>3 - 4</td>
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<td>4 - 5</td>
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Table B.2. CALIBRATION CHAMBER SPECIMEN NO. CC-2

$\sigma_c = 70$ kPa

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<th>DEPTH (ft)</th>
<th>sample no.</th>
<th>$\gamma_d$ (pcf)</th>
<th>$e$ (%)</th>
<th>$w$ (%)</th>
<th>Sr (%)</th>
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<td>0.5270</td>
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<td>0.4911</td>
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$e_{av} = 0.433$  $q_c(ave) = 27$ bars
$e_{av} = 0.4185$  $q_c(ave) = 35$ bars
$e_{av} = 0.4702$  $q_c(ave) = 15$ bars
Table B.3. CALIBRATION CHAMBER SPECIMEN NO. CC-3

$\sigma_c = 140$ kPa

<table>
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<th>DEPTH (ft)</th>
<th>sample no.</th>
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<th>e (%)</th>
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Table B.4. CALIBRATION CHAMBER SPECIMEN NO. CC-4

\( \sigma_e = 70 \) kPa

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<th>( \gamma_d ) (pcf)</th>
<th>e</th>
<th>w (%)</th>
<th>Sr (%)</th>
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</thead>
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### Table B.5. CALIBRATION CHAMBER SPECIMEN NO. CC-5

\( \sigma_c = 100 \text{ kPa} \)

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<th>( e ) (%)</th>
<th>( w ) (%)</th>
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Fig. B.1. CONSOLIDATION DATA OF SPECIMEN NO. CC-1
SPECIMEN No. CC-2
\( \sigma_c = 70 \text{ kPa} \)

Fig. B.2. CONSOLIDATION DATA OF SPECIMEN NO. CC-2
Fig.B.3. CONSOLIDATION DATA OF SPECIMEN NO. CC-3

SPECIMEN No. CC-3

$\sigma_c = 140$ kPa
Fig. B.4. CONSOLIDATION DATA OF SPECIMEN NO. CC-4

SPECIMEN No. CC-4
\( \sigma_c = 70 \text{ kPa} \)

TIME (hours)

COMPRESSION (mm)

10000
1000
10
1

15 20 25 30 35 40 45 50
Figure B.5: Consolidation Data of Specimen No. CC-5

Specimen No. CC-5

\[ \sigma_c = 100 \text{ kPa} \]
Fig. B.6. CONE PENETRATION TEST NO. CC1-STD
Fig. B.7. CONE PENETRATION TEST NO. CC2-STD
Fig. B.8. CONE PENETRATION TEST NO. CC3-MIN1
Fig. B.9. CONE PENETRATION TEST NO. CC3-MIN2
Fig. B.10. CONE PENETRATION TEST NO. CC3-MIN3
TIP RESISTANCE [bars]

DEPTH [cm]

FRICITION RATIO [%]

Fig.B.11. CONE PENETRATION TEST NO. CC3-MIN4
Fig. B.12. CONE PENETRATION TEST NO. CC3-MIN5
Fig. B.13. CONE PENETRATION TEST NO. CC3-MIN6
Fig. B.14. CONE PENETRATION TEST NO. CC4-MIN1
Fig.B.15. CONE PENETRATION TEST NO. CC4-MIN2
Fig. B.16. CONE PENETRATION TEST NO. CC4-MIN3
Fig. B.17. CONE PENETRATION TEST NO. CC4-MIN4
Fig. B.18. CONE PENETRATION TEST NO. CC4-MINS
Fig B.19. CONE PENETRATION TEST NO. CC4-MIN6
Fig. B.20. CONE PENETRATION TEST NO. CC5-MIN1
Fig. B.21. CONE PENETRATION TEST NO. CC5-MIN2
Fig. B.22. CONE PENETRATION TEST NO. CC5-MIN3
Fig. B.23. CONE PENETRATION TEST NO. CC5-MIN4
TIP RESISTANCE [bars]

FRICTION RATIO [%]

DEPTH [cm]

Fig.B.24. CONE PENETRATION TEST NO. CC5-MIN5
Fig. B.25. CONE PENETRATION TEST NO. CC5-MIN6
Fig. B.26. CONE PENETRATION TEST NO. CC5-STD.
Appendix C.  CPT AT PEPPER’S FERRY
TIP RESISTANCE (bars)

Fig. C.1. PEPPER'S FERRY CPT NO. STD-1.
Fig. C.2. PEPPER'S FERRY CPT NO. STD-2.
TIP RESISTANCE (bars)

Fig. C.3. PEPPER’S FERRY CPT NO. STD-3.
TIP RESISTANCE (bars)

PEPPERS FERRY CPT NO. MIN-1

FRICTION RATIO (%)

Fig.C.4. PEPPER’S FERRY CPT NO. MIN-1.
Fig. C.5. PEPPER'S FERRY CPT NO. MIN-2.
Fig. C.6. PEPPER'S FERRY CPT NO. MIN-3.
Fig. C.7. PEPPER'S FERRY CPT NO. MIN-4.
TIP RESISTANCE (bars)

Fig. C.8. PEPPER'S FERRY CPT NO. LRG-1.

DEPTH (m)

FRICION RATIO (%)
Appendix D. STATE PARAMETER AND THE OCCURRENCE OF LIQUEFACTION
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<th>Earthquakes</th>
<th>Site</th>
<th>$\sigma_m$ (kg/cm²)</th>
<th>$\sigma_m'$ (kg/cm²)</th>
<th>$q_c$ (kg/cm²)</th>
<th>$D_{50}$ (mm)</th>
<th>$\frac{\tau}{\sigma_o'}$</th>
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### Table D.1. STATE PARAMETER AND THE OCCURRENCE OF LIQUEFACTION

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The vita has been removed from the scanned document