LIQUEFACTION RESISTANT ON MONTEREY NO.0/30 SAND

by

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ABSTRACT

A series of isotropically consolidated undrained cyclic triaxial tests were conducted to determine the effect of relative density, confining pressure, stress ratio and frequency on the liquefaction resistance of Monterey No.0/30 Sand and behaviors of soil samples on the way to liquefaction. Fifteen cyclic triaxial tests were performed. Twelve samples were tested at 0.5Hz and additional three samples were prepared at 30% relative density and tested under 30 psi effective confining pressure at 1Hz, 1.5Hz and 2.0Hz, respectively to check the frequency effect. For the 12 samples, six were prepared at 30% relative density and other six at 50% relative density, and each of the six sample group, half were consolidated at 15 psi effective confining pressure and other half at 30psi. Three different stress ratios of 0.15, 0.25 and 0.4 were used in cyclic triaxial tests. Soil specimens were subjected to sinusoidal wave loading starting with tension phase.

Soil specimen preparation was very important process for cyclic triaxial test results. Required specimen dimensions, relative densities and degree of saturation were closely produred. Before the cyclic triaxial test, the pore pressure B parameter must be higher 0.95. De-aired water was used to seep through soil specimen to enhance the saturation, and then, back pressure was applied to further increase B parameter and degree of saturation.
Axial load, excess pore water pressure and axial displacement were continuously recorded during the application of cyclic loading. The recorded data were processed to attain stress-strain behavior, excess pore water pressure, stress path (or q-p′ plots) and Young’s modulus degradation during the cyclic loading test. Finally, liquefaction resistance was presented in terms of cyclic stress ratio versus number of the loading cycles to liquefaction.

The form and content of this abstract are approved. I recommend its publication.

Approved: Nien-Yin Chang
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1. Introduction

1.1 Problem Statement

Soil liquefaction is a phenomenon in which soil loses much of its strength or stiffness for a short time. Nevertheless it is long enough for liquefaction to be the cause of many failures, deaths and major financial losses.

Soil liquefaction is one of the most detrimental forms of earthquake-induced ground failure that can result in catastrophic damage to engineering structures. Ground liquefaction failure usually takes place in saturated loose granular soils. When shaken by earthquakes, a saturated loose granular soil will tend to density. Since the soil is saturated, the densification requires that water be expelled out of the soil mass so that soil particles can become more densely packed. If the process of expelling water cannot occur immediately, the soil particles will tend to become waterborne and induce a rapid rise in pore water pressure. When the induced pore water pressure becomes high enough to counterbalance the total stress acting on soil particles, the granular soil will lose all its shear strength and is, in a broad sense, designated as “in a state of liquefaction failure.”

1.2 Objectives and Scope

The major objective of this study is to investigate the effectors affected on liquefaction resistance, pore water pressure generation characteristics, and post liquefaction resistance of soils. To achieve the objectives, cyclic triaxial tests and the University of Colorado at Denver’s hollow cylindrical sample tests program for testing soil samples which are Monterey No.0/30 Sand. Statistical analysis of the test results was conducted through which following tasks were performed: (1) identification of factors which exert significant influence on the liquefaction
resistance of soils; (2) investigation of pore pressure generation characteristics of soils, (3) investigation factors on post liquefaction (or residual) strength of soils; and finally (4) refinement of current procedure for evaluating liquefaction resistance of sands.

1.3 Research Approach

The approach pursued in this research consisted of the following activities: (1) collect and report current methodologies for liquefaction potential evaluation; (2) report factors affecting liquefaction resistance of sand; (3) conduct a reconnaissance cyclic triaxial test program to evaluate the liquefaction potential of Monterey No. 0/30 Sand; (4) conduct hollow cylinder tests on Monterey No. 0/30 Sand to compare test results between hollow cylinder tests and cyclic triaxial tests.

1.4 Important of This Research

The four most important parts of this research are: (1) to collect methods for liquefaction potential analysis; (2) to collect main factors the governing liquefaction in the field; (3) factors affect cyclic triaxial strength; (4) to perform triaxial tests to determine the stress-strain relationships and stress path of Monterey No. 0/30 sand on different stress ratio, relative density, mean effective stress and cyclic deviatoric-stress amplitude. (5) to perform triaxial tests and the University of Colorado at Denver ‘s hollow cylindrical sample tests, and also compare test results.
2. Field Methodologies for Liquefaction Potential Evaluation

2.1 General

Preliminary assessments may often be made to determine whether a given site is likely or not likely to liquefy in response to earthquake ground motions. The previous occurrence of liquefaction in site soils, knowledge of embankment placement techniques that have historically performed well or poorly when shaken, the seismicity of the site, and degree of saturation are some of the factors that may indicate the potential for future liquefaction.

The importance of adequate site characterization to seismic stability analysis cannot be overstated. Much can and should be accomplished by acquiring and examining existing site data from the geological literature, historical records, earlier field investigations and even sensing imagery before additional subsurface investigation is planned or undertaken. The following information is essential to initial assessment of the potential for earthquake-induced ground failure:

(1) Site topography

(2) Soil profile, including general classification soil properties and the origin of site soils

(3) Water level records, representative of both current and historical fluctuations

(4) Evidence from project records, aerial photographs, or previous investigations of past ground failure at the site or at similar (geologically and seismologically) nearly areas (including historical records of liquefaction, topographical evidence of landslides, sand boils, effects of ground movement on trees and other vegetation, subsidence, and sand intrusions in the subsurface)

(5) Seismic history of the site
(6) Geologic history of the site, including age of site soils, glacial preconsolidation or preconsolidation by now-eroded overburden, and lateral extent and continuity of soil deposits.

A subsurface investigation should be performed in two phases, distinguished by coverage and purpose. The first of these should include Standard Penetration Tests (SPT) for measuring penetration resistance and obtaining disturbed split-spoon samples for classification and water content determination. Coverage of the site with SPT borings should be adequate to (1) establish general soil conditions, distributions of soil types, homogeneity and ground water elevations; (2) identify soils that, if shaking were sufficiently intense, might liquefy; and (3) assist in specifying the locations of additional boring and geophysical surveys aimed at detailed seismic response evaluation. The second phase of subsurface investigation likely includes surveys and undisturbed sampling borings to: (1) refine preliminary interpretation of stratigraphy and the extent of potentially liquefiable soils; (2) measure in situ densities and dynamic properties for input to dynamic response analysis; (3) recover undisturbed soil samples for laboratory testing.

2. 2 Standard Penetration Test (SPT)

2.2.1 General

The Standard Penetration Test (SPT) is a soil-sampling procedure that is in worldwide use and is generally accepted as providing some correlation with in-place properties of a soil. The SPT requires that a 2-in. (51mm) split spoon sampler be used in conjunction with a 140-lb (63.6kg) drive weight. The SPT reports the number of blows N to drive the sample 1ft (0.3m) into undisturbed soil by using the 140-lb weight falling 30in. (0.76m).
In the United States and most other countries, the standard penetration test (SPT) has been the most commonly used in situ test for characterization of liquefaction resistance; factors that tend to increase liquefaction resistance (e.g. density, prior seismic straining, overconsolidation ratio, lateral earth pressures, and time under sustained pressure) also tend to increase SPT resistance. Seed et al. (1983) compared the corrected SPT resistance and cyclic stress ratio for clean sand (Figure 2.1) and silty sand (Figure 2.2) sites at which liquefaction was or was not observed in earthquakes of $M = 7.5$ to determine the minimum cyclic stress ratio at which liquefaction could be expected in a clean sand of a given SPT resistance.

2.2.2 Cyclic Resistance Ratio from the Standard Penetration Test

The cyclic resistance ratio represents the liquefaction resistance of the in situ soil. The most commonly used method for determining the liquefaction resistance is to use the data obtained from the standard penetration test. The advantages of using the standard penetration test to evaluate the liquefaction potential are as follows:

1. **Groundwater table:** a boring must be excavated in order to perform the standard penetration test. The location of the groundwater table can be measured in the borehole. Another option is to install a piezometer in the borehole, which can then be used to monitor the groundwater level over time.

2. **Soil type:** In clean sand, the SPT sampler may not be able to retain a soil sample. But for most other types of soil, the SPT sampler will be able to retrieve a soil sample. The soil sample retrieved in the SPT sampler can be used to visually classify the soil and to estimate the percent fines in the soil. In addition, the soil specimen can be
returned to the laboratory, and classification tests can be performed to further assess the liquefaction susceptibility of the soil.

3. Relationship between N value and liquefaction potential: in general, the factors that increase the liquefaction resistance of a soil will also increase the \((N_1)_60\) from the standard penetration test. For example, a well-graded dense soil that has been preloaded or aged will be resistance to liquefaction and will have high values of \((N_1)_60\). Likewise, a uniformly graded soil with a loose and segregated soil structure will be more susceptible to liquefaction and will have much lower values of \((N_1)_60\).

The presence of fines can affect SPT resistance and therefore must be accounted for in the evaluation of liquefaction resistance (Seed et al., 1985; Ishihara and Kosecki, 1996; Koester, 1994). Examination of Figures 2.1 and 2.2 shows that the liquefaction resistance of sands is not influenced by fines unless the fines comprise more than 5% of the soil. At higher fines contents, the fines tend to inhibit liquefaction [i.e., the CSR required to initiate liquefaction (for a given \((N_1)_60\) value)]. The plasticity of the fines can also influence liquefaction resistance; the adhesion of plastic fines tends to resist the relative movement of individual soil particles and thereby reduce the generation of excess pore pressure during earthquakes. Laboratory tests (Ishihara and Koseki, 1996) indicate little influence at plasticity indices below 10, and a gradual increase in liquefaction resistance at plasticity indices greater than 10. Ishihara (1996) suggested that the effects of plasticity could be accounted for by multiplying the CSR by the factor

\[
F = \begin{cases} 
1.0 & \text{PI} \leq 10 \\
1.0 + 0.022(\text{PI} - 10) & \text{PI} > 10 
\end{cases}
\]
Since most sandy soils in alluvial deposits and man-made fills have plasticity indices less than about 15, the effect of fines plasticity is usually small. Because strong-motion duration (hence equivalent number of uniform stress cycles) increases with earthquake magnitude, the minimum cyclic stress ratio required to initiate liquefaction decreases with increasing magnitude. The minimum cyclic stress ratio for other magnitudes may be obtained by multiplying the cyclic stress ratio for M= 7.5 earthquakes by the factors shown in Table 2.1.
Figure 2.1 Relationship between cyclic stress ratios causing liquefaction and $(N_1)_{60}$ values for clean sands in $M = 7.5$ earthquakes. (After Seed et al. (1975). Influence of SPT procedures in soil liquefaction resistance evaluations, *Journal of Geotechnical Engineering*, Vol. 111, No. 12. Reprinted by permission of ASCE.)
Figure 2.2 Relationship between cyclic stress ratios causing liquefaction and \((N_1)_{60}\) values for silty sands in \(M = 7.5\) earthquakes. (After Seed et al. (1975). Influence of SPT procedures in soil liquefaction resistance evaluations, *Journal of Geotechnical Engineering*, Vol. 111, No. 12. Reprinted by permission of ASCE.)
Table 2.1 Magnitude Correction Factors for Cyclic Stress Approach

<table>
<thead>
<tr>
<th>Magnitude, M</th>
<th>CSR&lt;sub&gt;M&lt;/sub&gt; / CSR&lt;sub&gt;M= 7.5&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.25</td>
<td>1.50</td>
</tr>
<tr>
<td>6</td>
<td>1.32</td>
</tr>
<tr>
<td>6.75</td>
<td>1.13</td>
</tr>
<tr>
<td>7.5</td>
<td>1.00</td>
</tr>
<tr>
<td>8.5</td>
<td>0.89</td>
</tr>
</tbody>
</table>

A large data base of SPT blowcounts, normalized to account for the effects of different overburden pressure and performance conditions, has been correlated to occurrence and non-occurrence of liquefaction in a wide variety of soils (Seed, Idriss and Arango, 1983, Seed, et al. 1985, Farrar, 1988).

The SPT remains the tool of choice for preliminary in situ investigation of liquefaction potential as a result of its empirical correlation to field performance. The term “standard” is of dubious relevance, as the standard procedure specified for SPT performance by the American Society for Testing and Materials (1967) is not so rigid as to prevent variations in practice. Other countries have also developed indigenous versions of the test, unconstrained by the US regulation.
Figure 2.3. SPT Setup.
Based on the standard penetration test and field performance data, Seed et al. (1985) concluded that there are three approximate potential damage ranges that can be identified:

Table 2.2 Relationship between ($N_{1}$)$_{60}$ and potential damage

<table>
<thead>
<tr>
<th>$N_{1}$$_{60}$</th>
<th>Potential damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-20</td>
<td>High</td>
</tr>
<tr>
<td>20-30</td>
<td>Intermediate</td>
</tr>
<tr>
<td>&gt;30</td>
<td>No significant damage</td>
</tr>
</tbody>
</table>

Figure 2.2 presents a chart that can be used to determine the cyclic resistance ratio of the in situ soil. This figure was developed from investigations of numerous sites that had liquefied or did not liquefy during earthquakes. Use Figure 2.2 to determine the cyclic resistance ratio of the in situ soil, as follows: 1. Standard penetration test ($N_{1}$)$_{60}$ value: note in Figure 2.2 that the horizontal axis shows data from the SPT test, which must be expressed in terms of the ($N_{1}$)$_{60}$ values. 2. Percent fines: once ($N_{1}$)$_{60}$ value has been calculated, the next step is to determine or estimate the percent fines in the soil. For a given ($N_{1}$)$_{60}$ value, soils with more fines have a higher liquefaction resistance. 3. Cyclic resistance ratio for an anticipated magnitude 7.5 earthquake.
2.3 Cone Penetration Test (CPT)

2.3.1 General

The standardized cone-penetration test (CPT) involves pushing a 1.41-inch diameter 55°
to 60° cone through the underlying ground at a rate of 1 to 2 cm/sec. CPT soundings can be very
effective in site characterization, especially sites with discrete stratigraphic horizons or
discontinuous lenses. CPT (ASTM D-3441, adopted in 1974) is a valuable method of assessing
subsurface stratigraphy associated with soft materials, discontinuous lenses, organic materials
(peat), potentially liquefiable materials (silt, sands and granule gravel) and landslides. The Cone
rigs can usually penetrate normally consolidated soils and colluvium, but have also been
employed to characterize d weathered Quaternary and Tertiary-age strata. The cone is able to
delineate even the smallest (0.64 mm/1/4-inch thick) low strength horizons, easily missed in
conventional (small-diameter) sampling programs. Some examples of CPT electronic logs are
attached, along with hand-drawn lithologic interpretations. Most of the commercially-available
CPT rigs operate electronic friction cone and piezocone penetrometers, whose testing procedures
are outlined in ASTM D-5778, adopted in 1995. These devices produce a computerized log of tip
and sleeve resistance, the ratio between the two, induced pore pressure just behind the cone tip,
pore pressure ratio (change in pore pressure divided by measured pressure) and lithologic
interpretation of each 2 cm interval are continuously logged and printed out.

The CPT is a promising subsurface investigation tool for a variety of applications,
particularly in the shallow, soft soils prone to earthquake-induced liquefaction. The electrical
friction cone penetrometer (several variations, depending on instrumentation design) have
replaced the earlier, mechanical version (both types are sketched in Figure 2.1), due primarily to
the ability to obtain continuous, direct measurement of the resistance of soil to penetration and
friction. These two parameters have been correlated with soil type and behavioral properties (Douglas, Olsen and Martin, 1994, Olsen and Farr, 1996, and Olsen and Malone, 1994). One failing of the CPT is the inability to obtain physical samples; on the other hand, borings usually require circulation of drilling fluid and much greater labor and time (and accordingly, expense) to advance through similar depths of investigation.

In deference to the large field performance data base on liquefaction potential that has evolved using the SPT, researchers have usually elected to convert CPT data into equivalent SPT values and take advantage of existing correlations. Adaptation of CPT data in this manner is supported by Douglas, Olsen and Martin (1981), who concluded from a detailed study of influential factors common to both the SPT and CPT:

(1) the SPT and CPT are similarly affected by certain soil properties, such that CPT results are directly relatable to SPT results for liquefaction potential;

(2) CPT profiles provide much finer resolution of stratigraphy than do SPT results (and liquefaction failure may occur in thin layers that could lead to sliding); and

(3) The typically large variation of test results associated with the actual performance of an SPT is substantially avoided with the CPT, which is more automated (see also Federal Highway Administration, 1978).

Olsen (1994) proposed normalization of measured CPT data to a function of the effective overburden stress, followed by conversion to continuous, normalized SPT data. The chart shown in Figure 2.10 illustrates the interrelationship developed to predict normalized (to 1 tsf effective overburden stress) SPT blow counts, $N_1$, the exponent $n$ ranges from about 0.6 in coarse sands to 1.0 in clays (Olsen and Malone, 1994). These normalized SPT data may then be
compared to laboratory cyclic strength test data or the field performance data base for various soils.

2.3.2 Cyclic Resistance Ratio from the Cone Penetration Test

As an alternative to using the SPT test, the CPT can be used to determine CRR of the in situ test. The tip resistance from the CPT test can also be used as a measure of liquefaction resistance.

In CPT-based liquefaction evaluations, the tip resistance is normalized to a standard effective overburden pressure of 1 ton/ft² (96 kPa) by

\[
q_{c1} = q_c \left(\frac{p_a}{\sigma'_{vo}}\right)^{0.5} \quad \text{or} \quad q_{c1} = \left[\frac{1.8}{(0.8 + \sigma'_{vo})}\right] q_c
\]

(2.7)

where \(\sigma'_{vo}\) is in tons/ft² (Kayen et al., 1992). Adjustment for magnitudes other than 7.5 can be made using the CSR correction factors presented in Table 2.1. Kayen et al. (1992) found that liquefaction observations in the 1989 Lorna Prieta earthquake agreed well with the curves of Robertson and Campanella (1985) and Mitchell and Tseng (1990).

For silty sands (> 5% fines), the effects of fines can be estimated by adding the following tip resistance increments to the measured tip resistance to obtain an equivalent clean sand tip resistance (Ishihara, 1993)

The cyclic resistance ratio (CRR) represents the capacity of the soil to resist liquefaction. The relationship recommended by Youd et al. (2001) for computing CRR from CPT measurements can be expressed as (Robertson and Wride 1998):

If \((q_{c1N})_{cs} < 50\), \(CRR = 0.833 \left[\frac{(q_{c1N})_{cs}}{1000}\right] + 0.05\)

If \(50 \leq (q_{c1N})_{cs} < 160\), \(CRR = 93 \left[\frac{(q_{c1N})_{cs}}{1000}\right]^3 + 0.08\)

Where \((q_{c1N})_{cs}\) is the clean-sand cone tip resistance normalized to atmospheric pressure.
The stress-normalized cone tip resistance \((q_{c1N})\) is calculated using the following equation (Robertson and Wride 1998):

\[
q_{c1N} = C_Q \left( \frac{q_c}{P_a} \right) = \left( \frac{P_a}{\sigma_v'} \right)^n \left( \frac{q_c}{P_a} \right)
\]

where \(q_c\) is the measured cone tip resistance in the same units as \(P_a\), where \(P_a\) is a reference pressures assumed to be atmospheric pressure (about 100 kPa) in the same units \(\sigma_v'\) and \(n\) is an exponent that depends on soil type. To avoid unreasonably high values at shallow depths, Youd et al. (2001) recommended that \(C_Q\) be limited to a maximum value of 1.7. For cone measurements made with a pressure transducer behind the cone tip, values of \(q_c\) are corrected for the effect of pore pressures (Lunne et al. 1997). This correction is particularly significant in silty soils. The exponent \(n\) is a variable that depends on soil type and is assumed as 0.5 for granular soils and 1.0 for clay.

Several investigators have noted that liquefaction resistance of soils increases with age (e.g., Seed, 1979; Youd & Hoose, 1977; Youd & Perkins, 1978; Arango et al. 2000; Leon et al. 2006.) However, because the processes causing increased liquefaction resistance with age were poorly understood and proposed correction factors for age had not been verified, Youd et al. (2001) did not recommended age correction factors at the time of their study. In an effort to account for the affect of age on CRR, the following correction equation has been proposed (Andrus et al. 2004):

\[
CRR_a = CRR \times K_{a2}
\]

where \(CRR_a\) is the age-corrected cyclic resistance ratio, and \(K_{a2}\) is a factor to correct for influence of age. The value of \(K_{a2}\) is 1.0 for soils less than a few thousand years old. For older soils, Andrus et al. (2004) suggested using the lower bound of the relationship between cyclic strength and time proposed by Arango et al. (2000).
Table 2.3 Relationship between fines content and tip resistance increment

<table>
<thead>
<tr>
<th>Fines Content (%)</th>
<th>Tip Resistance Increment (tons/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 5</td>
<td>0</td>
</tr>
<tr>
<td>≤ 10</td>
<td>12</td>
</tr>
<tr>
<td>≤ 15</td>
<td>22</td>
</tr>
<tr>
<td>≤ 35</td>
<td>40</td>
</tr>
</tbody>
</table>

Figure 2.4 – Manufacturing and operating tolerances of cones, taken from ASTM D5778.
Figure 2.5- Schematic section through a piezocone head, showing the piezo-element and friction sleeve. Taken from ASTM D5778.
2.4 Piezometric Cone Penetrometer Test

Miniaturized instrumentation was installed into electric cone penetrometer devices such that pore pressures might be measured both as the probe is pushed into the soil and to monitor ambient head to determine the precise depth to the ground water table. Early studies indicated that penetration of a cone penetrometer would increase pore water pressures in contractive, potentially liquefiable soils and induce negative pore water pressure in dilative deposits (Schmertmann, 1978).

As additional CPT push rod segments are added, excess pore pressure dissipates to ambient levels at the penetrometer tip. Forrest, Ferritto and Wu (1981) report a study of one such device, comparing CPT results (cone tip penetration resistance and piezometric level only) with laboratory cyclic triaxial strengths in waterfront deposits where dissipation rates correlated with permeability; it was postulated that such information could implicate liquefiable soils, though no such attempts were directly made.

Cooper and Franklin (1982) and Norton (1983b) described a piezometer cone penetrometer that measured both cone tip penetration resistance and sleeve frictional resistance as do typical electric CPT devices, with the addition of a pore pressure transducer at the tip.

2.5 Other Techniques

2.5.1 General

A number of additional in situ testing techniques show promise as tools to assist in site characterization for liquefaction potential evaluation. Any or all in situ techniques may someday prove useful in the assessment of cyclic strength of fine-grained soils, since the soils of concern are difficult to sample.
The self-boring pressure meter was used to evaluate liquefaction potential of sand through correlation with the dilation angle parameter. Dilation angle, defined as the inverse sine of the slope of a volume expansion-versus-shear strain curve, may be measured either from drained laboratory triaxial of simple shear tests or from in situ pressure meter tests. Pilot tests on a hydraulic fill dam yielded reasonably similar estimation of liquefaction resistance from SPT blow count-based and pressure meter-based techniques.

Electrical resistivity and conductivity geophysical survey methods have been applied to characterize in situ properties using either surface or borehole sensor arrays (Department of the Army, 1999). They studied electrical anisotropy of soil deposits, developing a structural index that may correlate to cyclic strength. Erchul and Gularte (1982) investigated densification in liquefying sand deposits in the laboratory using electrical resistivity; they proposed extending the method to evaluate field deposits and monitor compaction efficiency.

Chen Yunmin and Chen Pen-peng (2005) determined the response of reconstituted laboratory soil specimens to a given low-amplitude P-wave excitation, demonstrating a relationship between acoustic signature so measured and liquefaction resistance. The study was aimed at evaluation of liquefaction potential in marine deposits where sampling is particularly difficult and a data base exists for acoustic response.

Seismic wave velocities (P-wave and shear, S-wave) are routinely determined through field geophysical surveys to obtain input for dynamic response analysis (Department of the Army, 1999). Dobry, et al. (1981) described the use of shear wave velocity to estimate a threshold earthquake acceleration for liquefaction.
Measured shear wave velocities can be normalized to a standard effective overburden pressure of 1 ton/ft\(^2\) (96 kPa) by

\[
V_{sl} = V_s (\sigma'_{vo})^{-1/n}
\]  

(2.1)

where \(\sigma'_{vo}\) is in tons/ft\(^2\) and \(n\) is taken as 3 (Tokimatsu et al., 1991) or 4 (Finn, 1991; Kayen et al., 1992). Stokoe et al. (1988) used the cyclic strain approach and equivalent linear ground response analyses to explore the relationship between peak ground surface acceleration (for stiff soil site conditions) and shear wave velocity. The results were used to develop bounds for the conditions under which liquefaction could be expected; the results agreed well with observed behavior in two earthquakes in the Imperial Valley of California (Figure 2.6). Tokimatsu et al. (1991) used the results of laboratory tests to develop curves showing the CSR required producing cyclic strain amplitude of 2.5% in various numbers of cycles as a function of corrected shear wave velocity (Figure 2.7).

The observation that the shear wave velocity of sand is insensitive to factors (e.g., soil fabric, overconsolidation ratio, prior cyclic straining) that are known to influence liquefaction resistance suggests that shear wave velocity measurements alone may not be sufficient to evaluate the liquefaction potential of all soil deposits (Jamiolkowsky and LoPresti, 1992; Verdugo, 1992).

Tokimatsu, Yoshimi and Uchida (1996) proposed a method to evaluate in situ liquefaction resistance of dense sands that may eventually prove adaptable to other soils, wherein: (1) shear wave velocities are determined by geophysical survey; (2) high-quality samples are obtained by in situ freezing; (3) laboratory initial shear modulus, \(G_{max}\), is determined by low amplitude cyclic shear testing (type of equipment unspecified) and compared to that calculated
from field shear wave velocity; (4) laboratory $G_{\text{max}}$ is adjusted (increased) by application of low amplitude (equipment again unspecified) preshearing until field and laboratory values match, and (5) cyclic triaxial tests are performed to measure liquefaction resistance of thawed specimens.

Adjusted specimen liquefaction resistance is claimed to represent in situ behavior, Stokoe, et al. (1988) developed charts relating shear wave velocity to maximum surface acceleration, $a_{\text{max}}$, that predict liquefaction potential in clean sands (e.g., Figure 2.11).

Figure 2.6 Chart for evaluation of liquefaction potential from shear wave velocity and peak ground acceleration (0 cycles). (After Stokoe et al., 1988.)
Figure 2.7 Correlations between cyclic stress ratio required to produce cyclic strain amplitude of 2.5% in clean sand and shear wave velocity. (After Tokimatsu et al., 1991.)

2.5.2 Cyclic Resistance Ratio from the Shear Wave Velocity

The shear velocity of the soil can also be used to determine the factor of safety against liquefaction. The shear wave velocity can be measured in situ by using several different geophysical techniques, such as the up hole, down-hole, or cross-hole methods. Other methods that can be used to determine the in situ shear wave velocity include the seismic cone penetrometer and suspension logger (Woods 1994).

The shear wave velocity is corrected for the overburden pressure by using the following equation (Sykora 1987, Robertson et al. 1992):

\[ V_{sl} = V_s C_v = V_s \left( \frac{100}{\sigma_{vo}} \right)^{0.25} \]  

(2.8)

Where \( V_{sl} \) = corrected shear wave velocity
$C_v = \text{correction factor to account for overburden pressure. } C_v = (100/\sigma_{vo}')^{0.25}$

$\sigma_{vo}' = \text{vertical effective stress kPa}$

$V_s = \text{shear wave velocity measured in field}$

Figure 2.9 relationship between cyclic resistance ratio and corrected shear wave velocity for clean sand, silty sand, and sandy for $M=7.5$ earthquake (From Andrus and Stokoe 2000, required with permission of the American Society of Civil Engineers.)

Figure 2.9 showed that the corrected shear wave velocity $V_{s1}$, and then by intersecting the appropriate fines content curve, the cyclic resistance ratio is obtained.

An advantage of using the shear wave velocity to determine the factor of safety against liquefaction is that it can be used for very large sites where an initial evaluation of the liquefaction potential is required. Disadvantages of this method are that soil samples are often
not obtained as part of the testing procedure, thin strata of potentially liquefiable soil may not be identified, and the method is based on small strains of the soil, whereas the liquefaction process actually involves high strains.

In addition, as indicated in Fig. 2.9, there are little data to accurately define the curves above a CRR of about 0.3. Furthermore, the curves are very steep above a shear wave velocity of 200 m/s, and a small error in measuring the shear wave velocity could result in a significant error in the factor of safety.

2.6 Cyclic Stress Ratio Caused by Earthquake

If it is determined that the soil has the ability to liquefy during an earthquake and the soil is below or will be below the groundwater table, then the liquefaction analysis is performed. The first step in the simplified procedure is to calculate the cyclic stress ratio, also commonly referred to as the seismic stress ratio (SSR) that is caused by the earthquake.

To develop the CSR earthquake equation, it is assumed that there is a level ground surface and a soil column of unit width and length, and that the soil column will move horizontally as a rigid body in response to the maximum horizontal acceleration $a_{\text{max}}$ exert by the earthquake at ground surface. The weight $W$ of the soil column is equal to $\gamma_t z$, where $\gamma_t =$ total unit weight of the soil and $z =$ depth below ground surface. The horizontal earthquake force $F$ acting on the soil column (which has a unit width and length) is:

$$F = ma = (W/g) a = (\gamma_t z/g) a_{\text{max}} = \sigma_{vo} (a_{\text{max}}/g)$$

where $F =$ horizontal earthquake force acting on soil column that has a unit width and length, lb or kN

$m =$ total mass of soil column, lb or kg, which is equal to $W/g$. 

25
\[ W = \text{total weight of soil column, lb or kN.} \] For the assumed unit width and length of soil column, the total weight of the soil column is \( \gamma_t z \)

\[ \gamma_t = \text{total unit weight of soil, lb/ft}^3 \text{ or kN/m}^3 \]

\[ z = \text{depth below ground surface of soil column} \]

\[ a = \text{acceleration, which is the maximum horizontal acceleration at ground surface caused by the earthquake} (a=a_{\text{max}}), \text{ ft/s}^2 \text{ or m/s}^2 \]

\[ a_{\text{max}} = \text{maximum horizontal acceleration at ground surface that is induced by the earthquake, ft/s}^2 \text{ or m/s}^2. \]

\[ \sigma_{vo} = \text{total vertical stress at bottom of soil column, lb/ft}^2 \text{ or kPa. The total vertical stress} = \gamma_t z \]

Since the soil element is assumed to have a unit base width and length, the maximum shear force \( F \) is equal to the maximum shear stress \( \tau_{\text{max}} \),

\[ \tau_{\text{max}} = F = \sigma_{vo} \left( a_{\text{max}}/g \right) \quad (2.3) \]

dividing both sides of the equation by the vertical effective stress \( \sigma_{vo}' \) gives

\[ \left( \frac{\tau_{\text{max}}}{\sigma_{vo}'} \right) = \left( \frac{\sigma_{vo}}{\sigma_{vo}'} \right) \left( a_{\text{max}}/g \right) \quad (2.4) \]

Seed and Idriss (1971) incorporated a depth reduction factor \( \gamma_d \) into the above equation:

\[ \left( \frac{\tau_{\text{max}}}{\sigma_{vo}'} \right) = \gamma_d \left( \frac{\sigma_{vo}}{\sigma_{vo}'} \right) \left( a_{\text{max}}/g \right) \]

For the simplified method, Seed et al. (1975) converted the typical irregular earthquake record to an equivalent series of uniform stress cycles by assuming the following:

\[ \tau_{\text{cyc}} = 0.65 \tau_{\text{max}} \quad (2.5) \]

where \( \tau_{\text{cyc}} = \text{uniform cyclic shear stress amplitude of earthquake, lb/ft}^2 \text{ or kPa} \)

\[ \text{CSR} = \left( \frac{\tau_{\text{max}}}{\sigma_{vo}'} \right) = 0.65 \gamma_d \left( \frac{\sigma_{vo}}{\sigma_{vo}'} \right) \left( a_{\text{max}}/g \right) \quad (2.6) \]
Figure 2.8 Reduction factor to estimate the variation of cyclic shear stress with depth below level or gently sloping ground surfaces. (After Seed and Idriss, 1971)

Figure 2.8 presents the range in values for the depth reduction factor $\gamma_d$ versus depth below ground surface.

Another option is to assume a linear relationship of $\gamma_d$ versus depth and use the following equation (Kayen et al. 1992): $\gamma_d = 1-0.012z$
2.7 Factor Safety against Liquefaction

The final step in the liquefaction analysis is to calculate the factor of safety against liquefaction. If the cyclic stress ratio caused by the anticipated earthquake is greater than the cyclic resistance ratio of the in situ soil, then liquefaction could occur during the earthquake, and vice versa. The factor of safety against liquefaction (FS) is defined as follows:

\[ \text{FS} = \frac{\text{CRR}}{\text{CSR}} \]  

(2.9)

The higher the factor of safety, the more resistant the soil is to liquefaction. However, soil that has a factor of safety slightly greater than 1.0 may still liquefy during an earthquake.
Figure 2.10 CPT prediction of overburden pressure-corrected SPT blow-count (Olsen and Malone, 2000)
Figure 2.11 Chart to predict liquefaction in clean sands from shear wave velocity and maximum acceleration (Stokoe, et al. 1988)
3. Laboratory Methods for Liquefaction Resistance Evaluation

3.1 Undisturbed Sampling

Soil samples are disturbed both mechanically and by change in their effective stress state during sampling and transporting to testing facilities. The term “undisturbed” is liberally interpreted to imply sampling activities that minimize mechanical disturbance for the purposes of this study. As concerns liquefaction potential evaluation, Marcuson and Franklin (1979) review techniques and apparatuses that are still commonly applied to sample granular soils. Significant conclusions reported in that reference include: (1) fixed-piston, thin-walled tube samplers used in boreholes supported by appropriately mixed drilling mud or fluid generally yield high quality samples of many sands; (2) the use of radiographs of samples within sampling tubes permits judgement of sampling disturbance for selection of representative specimens; (3) undisturbed gravel specimens can be successfully obtained only by hand carving larger block samples; and (4) in situ freezing of a larger-than-required volume of soil for subsequent trimming produces very high quality (with regard to mechanical disturbance) soil samples, as long as the freezing front is propagated in a manner that assures free drainage.

Marcuson and Franklin (1979) reported that fixed piston sampling operations tend to produce the best samples so obtained when used in medium dense sands. Tube sampling was observed to densify loose sands and dilate dense sands. The implication is that cyclic strength test results on tube sampled specimens, if interpreted directly, would be unconservative in the case of sands that were loose in situ, and overconservative in dense sands.

Singh, Seed and Chan (1982) examined in situ freezing techniques for undisturbed sampling of saturated sands. Few studies have addressed the efficacy of freezing in silty or
clayey soils. Tani and Yasunaka (1988) studied the effects of in situ freezing to sample sands with up to 6% particles finer than 74 micrometers (i.e., passing the US Standard No. 200 sieve). Their results indicated that there was no change in cyclic triaxial liquefaction resistance for alternately frozen and thawed specimens. Samples were taken from the body of a small earth dam that experienced moderate settlement due to liquefaction within either the embankment or its foundation or both. Tani and Yasunaka (1988) claimed from a small number (unstated, but by data plots apparently less than 10) of tests that samples taken by in situ freezing were thus representative of “true liquefaction resistance”.

A number of studies have examined the effects of methods of reconstitution to prepare representative specimens of soils that are difficult to sample (e.g., Mulilis, Chan, and Seed, 1975, Marcuson and Townsend, 1976, Ladd, 1977). No one method of reconstitution best represents natural deposition processes and preserves in situ fabric.

3.2 Laboratory Test

3.2.1 Monotonic Tests

Liquefaction response may be indicated by monotonic, undrained triaxial compression test results. Figure 3.1 illustrates typical stress-strain and pore pressure response such as might results from isotropically consolidated, monotonic triaxial compression tests on sand specimens prepared to void ratios either side of and very close to the critical void ratio, that is, that void ratio at which a soil can deform continuously at constant shearing stress (Casagrande, 1936). A study state of deformation, defined as the state in which a mass of soil is continuously deforming at constant volume, constant normal effective stress, constant shear stress, and constant rate of
shear strain (Poulos, Castro and France, 1985) may develop if and only if a soil is contractive (i.e., its void ratio is greater than the critical value).

Poulos, Castro and France (1985) describe a liquefaction evaluation procedure to determine whether a soil deposit is susceptible to liquefy and subsequently achieve a steady state of deformation and associated flow-slide failure, based primarily on the in situ void ratio of the soil and the available driving shear stress on failure surfaces. It is contended for stability evaluation that if the factor of safety against liquefaction, defined as the ratio of undrained steady state shear strength (assumed to be the lowest in situ shear strength in a contractive soil in situ) to the shear stress required to maintain static stability, is less than 1, the soil mass is considered to be in unstable equilibrium. There must be a disturbance sufficient to load the soil to the point of steady state strength, by either monotonic or cyclic loading as depicted in Figure 3.2 to result in liquefaction.

The procedure promoted by Poulos, Castro and France (1985) consists of five steps: (1) determination of in situ void ratio, by either fixed-piston sampling, freezing and coring, or test pit sampling; (2) developing a relationship between void ratio at the steady state and effective stress from strain-controlled, undrained triaxial compression tests on remolded specimens; (3) measuring the undrained steady state shear strength of “undisturbed” specimens by similar triaxial tests; (4) correcting measured undrained steady state strengths to correspond to the in situ void ratio; and (5) calculating the in situ driving shear stress and factor of safety. Step (4) is accomplished as follows (Figure 3.3): the steady state strength of an undisturbed specimen, \( S_{us} \), is plotted against its actual void ratio, \( e_l \); a dashed line is extended, parallel to the remolded specimen steady state line, to intersect a horizontal line drawn through the estimated field void ratio, \( e_f \) (determined in step (1)); and the estimated in situ steady state strength, \( (S_{us})_f \), is
determined from the abscissa where it is crossed by a vertical line drawn through this intersecting point.

Poulos, Castro and France (1985) caution that the distinction between contractive and dilative behavior is very sensitive to void ratio. Figure 3.4 illustrates that a density change of only 2 pcf drastically alters the stress-strain response of a uniform sand. The maximum shear stress at the steady state of deformation (i.e., the steady state shear strength) may change significantly with very slight change in void ratio Figure 3.5.

3.2.2 Cyclic Triaxial Tests

Ideally, the best cyclic test to evaluate response of soils to earthquake shaking would be one that correctly simulates the loading to which the soil would be subjected in situ. It is commonly believed that at least a single component of earthquake ground motion is adequately reproduced in one form or another of the cyclic simple shear test. Various configurations of cyclic simple shear, cyclic triaxial, large-scale shake table, and cyclic torsional shear (on solid or hollow specimens) apparatuses have been employed to study liquefaction resistance.

In a cyclic triaxial test, a cylindrical specimen of soil encased in a rubber membrane is placed in a chamber, subjected to confining fluid pressure and back pressure, and then loaded axially until failure. The axial load may be applied to the sample through a rigid top platen. The axial force can be compression or extension: thus the axial stress can be either major or minor principal stress. Usually the top platen is laid over a porous stone which allows fluid to flow in and out of the specimen. The axial deformation of the specimen is directly monitored by the movement of the piston which is in contact with or connected to the top platen. The lateral deformation is not usually measured. Transducers are used for pore pressure measurement.
In a cyclic triaxial test, a sample is consolidated under an initial isotropic confining pressure. The confining pressure is kept constant and axial load is either increased (compression test) or decreased (extension test) during a test. Thus, two of three principal stresses are always equal during a test. In a compression test the intermediate principal stress is equal to the minor principal stress; and the axial stress is equal to the major principal stress. In an extension test the major and the intermediate principal stress are equal, while the axial stress is equal to minor principal stress.

A variety of modified tests can be conducted in a conventional triaxial apparatus. Bishop and Henkel (1962) proposed several modified triaxial test. To simulate field conditions, a test can be performed by keeping the axial stress constant, while decreasing the confining pressure. Consolidation can be conducted under hydrostatic condition or at any ratio of axial-to-lateral stress. A triaxial test can be conducted at any ratio of principal stresses while keeping their mean stress constant. By conducting these tests, a wide variety of stress paths can be obtained.

Historically, the most common cyclic loading technique for investigating liquefaction resistance involves the performance of the cyclic triaxial test, as a consequence of such factors as availability of equipment and relative ease of preparing undisturbed specimens. This is in spite of wide recognition of the inability of the test to accurately represent field earthquake stresses (Seed and Idriss, 1982a). Figure 3.6 and 3.7 are a schematic drawing of the cyclic triaxial test apparatus and a sample recording of load, deformation, and pore pressure response, respectively. Cyclic strength curves such as are typically generated from cyclic triaxial data are shown in Figure 3.8. Instructions for performance of cyclic triaxial tests may be found in Engineer Manual 1110-2-1906 (Department of the Army, 1970).
Previous studies have demonstrated that cyclic triaxial strengths (in fact, strengths determined from any unidirectional loading test) are higher than those expected to produce equivalent effects in the field (Seed, 1976). Reduction factors were developed to adjust laboratory cyclic test strengths to estimate field liquefaction resistance. The current study and additional research efforts reported in the literature indicate that estimation of field cyclic strengths from laboratory cyclic test results may not be possible by universal application of simple factors.

3.2.3 Cyclic Hollow Cylinder Tests

A hollow cylindrical test device (HCTD) is an extremely valuable tool for studying constitutive behaviour under generalized stress conditions. The HCTD allows independent control of the magnitudes of the three principal stresses and rotation of the major-minor principal stress axes while recording the specimen deformational and pore pressure responses.

In a hollow cylinder test, a hollow cylindrical soil specimen is enclosed in between an inner membrane and an outer membrane. The confining pressure can be independently applied on both inner and outer chambers; therefore, inner and outer pressures can be controlled either equally or unequally. The axial load and torque are applied on the top of specimen and transmitted by a top cap or a pedestal to the specimen.

When each of these boundary stresses can be controlled independently, both the principal stress direction and the relative magnitude of the intermediate principal stress can be controlled, thus the HCA can facilitate more generalized stress path testing than the conventional test apparatus. It is also possible to control (or measure) the pore water pressure and apply back pressure, so that drainage conditions can be controlled and both drained and undrained tests can
be performed. As a result, the HCA offers an opportunity of extending the stress path approach to include simulation of both principal stress rotation and variation in intermediate principal stress, as well as conducting fundamental research into the effect of principal stress rotation under a reasonably generalized stress state.

Idealized stress conditions in a hollow cylindrical element subjected to axial load, $W$, torque, $M_T$, internal pressure, $P_i$, and external pressure, $P_o$.

During shearing, the torque, $M_T$, develops shear stresses, $\tau_{0z}$ and $\tau_{z\theta}$ ($\tau_{0z} = \tau_{z\theta}$) in vertical and horizontal planes, the axial load, $W$, contributes to a vertical stress, $\sigma_z$. $P_i$ and $P_o$ determine $\sigma_r$, $\sigma_\theta$. The relationship between $\sigma_r$ and $\sigma_\theta$, is established by the differences between $P_i$ and $P_o$.

$$\sigma_\theta = \sigma_r + r \left( d\sigma_r / dr \right)$$

where $r$ is the radial distance to a point in the hollow cylinder, and $d\sigma_r$ and $d\sigma_\theta$ are the radial and circumferential stress increments respectively. When $P_i = P_o$, $\sigma_r$ becomes identical to $\sigma_\theta$.

The state of stress in a hollow cylinder test is defined with reference to cylindrical coordinates, in terms of the stress components.

$$\begin{bmatrix} \sigma_r & 0 & 0 \\ 0 & \sigma_\theta & \tau_{\theta z} \\ 0 & \tau_{z\theta} & \sigma_z \end{bmatrix}$$

Since the stresses will not be uniform across the wall of the cylinder for various loading conditions, to consider the hollow cylinder as an element, it becomes necessary to calculate average stresses, $\sigma_z$, $\sigma_r$, $\sigma_\theta$, $\tau_{0z}$. Hight et al. (1983) used the following expressions:
Average vertical stress \( \sigma_z' = \frac{W}{\pi} \left( r_o^2 - r_i^2 \right) + \left( \frac{P_o r_o^2 - P_i r_i^2}{(r_o^2 - r_i^2)} \right) \)

Average radial stress \( \sigma_r' = \frac{P_o r_o + P_i r_i}{r_o + r_i} \)

Average circumferential stress \( \sigma_\theta' = \frac{P_o r_o - P_i r_i}{r_o - r_i} \)

Average shear stress \( \tau_{0z}' = \frac{3M_T}{2\pi} \left( r_o^3 - r_i^3 \right) \)

In hollow cylinder tests, the radial stress, \( \sigma_r' \), is usually equal to the intermediate principal stress (\( \sigma_2 \)). The major and minor principal stresses, \( \sigma_1 \) and \( \sigma_3 \), are observed from the average stress components \( \sigma_r' \), \( \sigma_\theta' \), and \( \tau_{0z}' \), and as following:

\[
\sigma_2 = \left[ (\sigma_r' + \sigma_\theta')/2 \right] + \sqrt{\left[ (\sigma_r' - \sigma_\theta')/2 \right]^2 + (\tau_{0z}')^2}
\]

\[
\sigma_1 = \sigma_r'
\]

\[
\sigma_3 = \left[ (\sigma_r' + \sigma_\theta')/2 \right] - \sqrt{\left[ (\sigma_r' - \sigma_\theta')/2 \right]^2 + (\tau_{0z}')^2}
\]

By regarding the specimen as a single element, the state of strain is presented in cylindrical coordinates in terms of the following components:

\[
[\varepsilon] = \begin{bmatrix}
\varepsilon_r & 0 & 0 \\
0 & \varepsilon_\theta & \frac{\gamma_{\theta z}}{2} \\
0 & \frac{\gamma_{z \theta}}{2} & \varepsilon_z
\end{bmatrix}
\]

Also, it is necessary to calculate the average strains. According to the paper of Hight *et al.* (1983), the average strains are calculated using the following equations:

Average axial strain \( \varepsilon_z = w/H \)
Average radial strain  
\[ \varepsilon_r = - \frac{(u_o-u_i)}{(r_o-r_i)} \]

Average circumferential strain  
\[ \varepsilon^\theta = - \frac{(u_o+u_i)}{(r_o+r_i)} \]

Average shear strain  
\[ \gamma_{\theta z} = \frac{2 \theta (r_o^3-r_i^3)}{3H(r_o^3-r_i^3)} \]

Since the average values of  \( \varepsilon_z \) and  \( \gamma_{\theta z} \) are based on strain compatibility only, the expressions for the average strains are valid and independent of the constitutive law of the material. The average values of  \( \varepsilon_r \) and  \( \varepsilon^\theta \) are based on a linear variation of radial displacement across the wall of the specimen. In the hollow cylinder test, the radial strain  \( \varepsilon_r \) is usually the intermediate principal strain,  \( \varepsilon_2 \). The major and minor principal strains can be observed from the average strain components:

\[
\varepsilon_1 = \left[ \frac{(\varepsilon_z + \varepsilon^\theta)}{2} \right] + \sqrt{\left[ \left( \frac{(\varepsilon_z - \varepsilon^\theta)}{2} \right)^2 + \left( \frac{\gamma_{\theta z}}{2} \right)^2 \right]} \\
\varepsilon_2 = \varepsilon_r \\
\varepsilon_3 = \left[ \frac{(\varepsilon_z + \varepsilon^\theta)}{2} \right] - \sqrt{\left[ \left( \frac{(\varepsilon_z - \varepsilon^\theta)}{2} \right)^2 + \left( \frac{\gamma_{\theta z}}{2} \right)^2 \right]} 
\]

Parameters  \( \alpha \) and  \( b \) are two variables of stress path to describe fundamentally different aspects in the applied state of state of stress.  \( \alpha \), is the inclination of major principal stress direction with respect to the vertical axis, which can be varied from 0 to 90°. It can be computed from the known average stress components

\[
\tan 2 \alpha = 2 \frac{\tau_{\theta z}'}{(\sigma_r' - \sigma_\theta')}
\]

\( b \) is defined as the relative magnitude of the intermediate principal stress, which can be varied from 0 to 1:

\[
b = \frac{(\sigma_2-\sigma_3)}{(\sigma_1-\sigma_3)}
\]
For the particular case of equal internal and external pressure, $P_i = P_o = P$, and are usually assumed to be equal to $P$. From Average radial stress $\sigma_r = (P_o r_o + P_i r_i) / (r_o + r_i)$, $\sigma_2$ is equal to $P$ as well. Therefore, changes in the $\alpha$ angle are accompanied by changes in magnitude of $b$. When $P_i = P_o$:

$$b = \sin^2 \alpha \quad \text{(Hight. et al., 1983)}$$

The direction of strain increment $\alpha_{dc}$ can be calculated from the incremental strain components

$$\tan 2\alpha_{dc} = \frac{d \gamma_{\theta z}'}{(d \varepsilon_z - d \varepsilon_0)}$$

The amount of non-coaxiality was defined as the difference between the directions of principal stress and of principal strain increments as, $\alpha_{dc} - \alpha$.

### 3.3 Methods for Liquefaction Resistance Analysis

#### 3.3.1 General

If soil types are present that may threaten the safety or function of a critical facility, detailed liquefaction potential analysis may be justified; the level of effect depends on such factors as the earthquake hazard and an assessment of associated risk. Current analysis methods may be broadly classed as either: (1) empirical procedures developed from field observations of the response of soil deposits shaken by previous earthquakes, wherein comparisons are drawn between soil types for which data have been collected and soils at the site in question; and (2) methods based on laboratory strength tests (most commonly, cyclic) on representative site soil samples to determine whether laboratory strengths will be exceeded by stresses generated in the field by the design earthquake.

#### 3.3.2 Empirical Approaches

The most prevalent empirical methodologies for assessing liquefaction potential of various soils are those based on correlation of liquefaction occurrence data with SPT blow
counts, following the lead initially developed by Seed and Idriss (1971), refined as per Seed, Idriss and Arango (1983) and later published widely by Seed, et al. (1985). Liquefaction occurrence data are represented in Figure 2.1 and 2.2 as observed in the Americas, Japan and the PRC in level ground deposits of clean sands and sandy soils containing fines, respectively, as a consequence of earthquakes having Richter magnitudes of approximately 7.5. If the design earthquake magnitude is other than 7.5, Figure 3.11 may be used to determine the appropriate clean sand relationship. Numbers shown near each data point in Figure 2.2 indicated the fines content determined for each.

The ordinate of these plots is the ratio of the estimated average cyclic shear stress developed on horizontal planes in the soil deposit to the initial effective overburden stress prior to the earthquake. The abscissa is the measured SPT blow count corrected for two factors, as follows: (1) \( N_1 \) is the equivalent blow count that would have been obtained if the effective overburden pressure were 1 tsf, and is calculated as \( N_1 = C_N \times N \), where \( C_N \) is found from relationships such as shown in Figure 3.10; and (2) \( (N_1)_{60} \) reflects adjustment of the blowcount to an equivalent value such as would have been measured if the SPT equipment delivered 60% of the theoretical maximum free-fall energy, as is average for US systems.

The liquefaction potential of a deposit where corrected blow counts (or equivalent blow counts converted from CPT data as described earlier) are available is evaluated by comparing the estimated earthquake-induced shear stresses in the soil deposit with the cyclic strength. Earthquake-induced shear stresses may be either calculated from one-, two, or in cases where geometry dictates, three-dimensional dynamic response analysis, or estimated by the following expression:

\[
\frac{[\tau_h]}{a_v} = \frac{\sigma_0'}{\sigma_0} = 0.65 \left( \frac{a_{\text{max}}}{g} \right) \times x_d \times \left( \frac{\sigma_0'}{\sigma_0} \right) \quad (3.1)
\]
Where \([\tau_h]a_v\) is the average cyclic shear stress developed by shaking on a horizontal plane in the deposit, here assumed equivalent to the ordinate of Figures 2.1 and 2.2; \(\sigma'_o\) and \(\sigma_o\) are the initial effective and total overburden stresses, respectively, at the depth in question; \(a_{\text{max}}\) is the maximum acceleration at the ground surface; \(g\) is the acceleration of gravity in units consistent with \(a_{\text{max}}\); and \(\gamma_d\) is a stress reduction factor that varies from a value of 1 at the ground surface to 0.9 at a depth of 30 ft. Figure 3.11 diagrams the procedure just discussed.

Ishihara (1985) reported the evolution of Japanese SPT correlations with liquefaction potential through 1985, derived both from laboratory studies and observations of field performance at site where SPT data is available. Farrar (1998) documents a study of SPT, cone penetrometer, and dilatometer correlations at six Japanese sites where liquefaction has occurred. Fine-grained soils are of great interest, due to the fact that they usually exhibit lower penetration resistance. Laboratory tests and field observations both confirm that liquefaction resistance is generally increased by the presence of fines, particularly clayey fines, thus correlations derived from clean sand tests and observations are not necessarily appropriate. Formulae that attribute added cyclic strength to given deposit based only on fines content may over predict strengths deposits where the fines fractions are non plastic.

Tokimatsu and Yoshimi (1983) supported continued correlation of SPT data to cyclic strength, the view of recent improvements in and trends toward standardization of SPT field procedures, presenting the following attributes:

(a) The SPT is a true in situ test, reflecting stress history, soil fabric, and horizontal effective stress, and is sensitive to the joint effects of relative density and overburden stress.
(b) SPT data are available as obtained from sites where liquefaction has occurred (before and after earthquakes), reflecting real in situ stress conditions, and a large body of data is now available at sites where liquefaction may occur in the future.  

(c) The SPT is essentially an undrained shear strength test.  

(d) Soil samples (disturbed) are retained for index properties determination.  

(e) Many SPT’s can be performed at a given site for low cost, affording economic coverage.  

Ishihara (1977) proposed the following expression based on a program of cyclic triaxial tests on undisturbed sand specimens:

\[ R_{\text{liq}} = 0.0676 \sqrt{N_1} \quad (3.2) \]

Where \( R_{\text{liq}} \) is the undrained cyclic triaxial strength defined as the cyclic stress ratio required to cause 5 percent double amplitude axial strain in 20 cycles; and \( N_1 \) is again a corrected SPT blow count, determined by normalizing the measured blow count value to an effective overburden pressure of 1 kgf/cm\(^2\), as follows (based on the formula of Meyerhof, 1957): where \( N \) is the actual measured SPT blow count; and \( \sigma_v \) is the effective overburden pressure in kgf/cm\(^2\) at the depth of the SPT measurement.

\[ N_1 = C_N \times N \quad \text{for} \quad C_N = 1.7 / (\sigma_v + 0.7) \quad (3.3) \]

Tatsuoka, et al. (1978) studied the effects of grain size on SPT penetration resistance with a series cyclic triaxial tests on undisturbed samples of young alluvial sand deposits and uncompacted hydraulic fills. The following expressions were derived from their test data:

\[ R_{\text{liq}} = 0.0676 \sqrt{N_1} + 0.225 \log_{10} (0.35/D_{50}) \quad \text{for} \quad 0.04 \text{ mm} \leq D_{50} \leq 0.6 \text{mm} \quad (3.4a) \]
\[ R_{\text{liq}} = 0.0676 \sqrt{N_1} - 0.5 \quad \text{for} \quad 0.6 \text{ mm} \leq D_{50} \leq 1.5 \text{ mm} \]  \hspace{1cm} (3.4b)

Where \( D_{50} \) is mean grain size. Cyclic triaxial strength increases with decreasing mean grain size for a given blow count by an amount described by the logarithmic term in the above equation.

The measure of \( D_{50} \) requires both sieve and hydrometer grain size analyses. Hydrometer analysis may not be done routinely, thus a formulation based on fines content might be more widely applicable. Tatsuoka, et al. (1980) proposed the following expression to this purpose, again based on laboratory cyclic triaxial test results:

\[ R_{\text{liq}} = 0.0676 \sqrt{N_1} + 0.0035C \]  \hspace{1cm} (3.5)

Where \( C \) is the fines content (i.e., percent finer than 0.074mm).

Ishihara (1979) proposed an alternative fines content relationship, based on the results of results of cyclic triaxial tests on similar materials retained in an Osterberg-type sampler:

\[ R_{\text{liq}} = 0.009(N_1+13+6.5 \log_{10} C) \]  \hspace{1cm} (3.6)

Where \( N_1 \) is again defined by \( N_1 = C_N \times N \), but where \( C_N \) is instead defined (after Peck, Hanson and Thornburn, 1974) as:

\[ C_N = 0.77 \log_{10} \left( \frac{20}{\sigma_v} \right) \]  \hspace{1cm} (3.7)

Where \( \sigma_v \) is tones per square foot. Equation (3.7) is only valid for overburden pressures greater than or equal to 0.25 tons per square foot (24kpa). Ishihara (1985) noted that the above formulae were developed using tube-sampled sandy materials, and contended that they should be applied only in the cases of loose to medium dense clayey and silty sands in view of likely
sampling disturbance effects. Matsumoto, Sasaki, and Kondoh (1988) conducted a program of 102 cyclic triaxial tests on undisturbed samples of soils ranging from coarse sands to sandy (maximum fines content of 96 percent), concluding that the following term be added to Equations 3.4a and 3.5b to correct for fines contents greater than 40 percent:

\[ R_{corr} = 0.004C - 0.16 \]  

(3.8)

Tatsuoka, et al. (1980) considered alluvial and hydraulic fill materials. Tailings materials containing low plasticity fines may exhibit cyclic strengths similar to those observed in loose sands. Ishihara, Yasuda, and Yokota (1981) developed the following expression based on results of cyclic triaxial tests on undisturbed specimens from 15 Japanese tailings dam sites:

\[ R_{liq} = 0.0676 \sqrt{N_1} + 0.085 \log_{10} (0.5/D_{50}) \]  

(3.9)

Where \( N_1 \) is the SPT blow count normalized as for Equation 3.3 Figure 3.12 compares relationships proposed by Tatsuoka, et al. (1980) with that developed from the data of Ishihara, Yasuda and Yokota (1981), and shows that the strengthening effect of decreasing particle size is less pronounced in the mine tailing studied than in alluvial materials.

The average relationship determined for tailings sampled at a site known as the Kuroko mine is also shown in Fig 3.12; the Kuroko tailings are reported to be typically plastic, and apparently retain cyclic strength across a wide range of mean grain size. The same study examined the influence of plasticity index on cyclic strength of various tailing containing an average of about 30 percent fines (plotted in Fig 3.12). Assuming that the \( I_p/35 \) strengthening effect (\( I_p \) is plasticity index) implied by the linear fit shown in Fig.3.12 holds in soils with more than 30 percent fines, Ishihara, Yasuda and Yokota (1981) suggest that Equations (3.5) and (3.6), respectively, be modified to include the plasticity effect as follows:
\[ R_{liq} = 0.0676 \sqrt{N_1} + 0.0001 I_p C \] (3.10)

and

\[ R_{liq} = 0.009(N_1+13) + 0.00167 I_p \log_{10} C \] (3.11)

The choice as to which expression to use depends on the selection of the respective \( C_N \) correction factor.

Xie (1979) reported the following empirical criterion from a PRC seismic design code in effect since 1974 to establish critical SPT blow counts, \( N_{crit} \), for liquefaction resistance, based on field performance:

\[ N_{crit} = N [1 + 0.125 (d_s - 3) - 0.05(d_w - 2)] \] (3.12)

where \( d_s \) is the depth to the sand layer in question, in meters; \( d_w \) is the depth to the water table, in meters; and \( N \) is a reference blow count, determined from earthquake shaking intensity. Data on liquefaction of clayey soils consequent to the 1976 Tangshan earthquake permitted generation of several proposed modifications to the 1976 code to account for the presence of clay fines. Such modifications always reduced the critical blow count required for soils containing clay, for example (shi, 1984):

\[ N_{crit} = N [1 + 0.125 (d_s - 3) - 0.05(d_w - 2) - 0.1 (p_c - 3)] \] (3.13)

or, another correlation:

\[ N_{crit} = N [1 + 0.125 (d_s - 3) - 0.05(d_w - 2)] (3/p_c)^{0.5} \] (3.14)

where \( p_c \) is the percent clay fines by weight (finer than 0.005 mm).
Seed, et al. (1985) maintain that the blow count values used to develop Equations 3.13 and 3.14 were measured using PRC equipment that produced about 60 percent of the theoretical maximum energy input to the drill stem; it was not necessary to correct the blow counts for energy equivalence for inclusion into the field performance data base. Seed, et al. (1985) report yet another such correction (by Taiping, et al., 1984) where the bracketed factor in Equation (3.4) is instead reduced by 0.07p_c.

3.3.3 Cyclic Stress–Based Approach

Seed and Idriss (1982a) describe a procedure for liquefaction potential evaluation of level-ground soil deposits based on direct comparison of earthquake stresses and cyclic strengths, the basic steps of which follow:

(1) The cyclic shear stress histories induced by earthquake ground shaking at varying depth or locations in a soil deposit are computed by dynamic response analysis and converted to equivalent numbers of uniform stress cycles (Annaki and Lee, 1977, Seed, et al. 1975). This procedure accounts for intensity and duration of shaking, as well as variation of shear stress history with depth. The equivalent number of uniform shear stress cycles for a given magnitude of earthquake motion may be obtained from the relationship shown in Fig 3.14.

(2) Laboratory cyclic loading tests are performed on representative undisturbed samples at various effective confining pressures to determine the cyclic stresses required to cause liquefaction in the number of stress cycles deemed equivalent for the design earthquake from step (1)
(3) Laboratory cyclic strengths determined in step (2) are compared to cyclic stresses from step (1) to ascertain where within the profile examined liquefaction may be caused by the design earthquake, conceptualized in Fig 3.15.

The above procedure is adaptable to evaluate liquefaction potential of earth embankment structures such as large earth dams, but dynamic response analysis may be more complicated. The effects of pre-earthquake effective overburden and shear stresses on liquefaction resistance must also be considered. Subsequent chapters address both previous and current laboratory studies of these effects.

A preliminary assessment of dynamic stresses generated in an earth dam may be made from one-dimensional shear wave propagation analyses (e.g., as performed using the SHAKE computer code or its revisions, originated by Schnabel, Lysmer and Seed, 1972) on idealized columns of soil developed at several locations throughout a critical cross section of the embankment, including foundation deposits to bedrock (the source of upward propagating shear waves) where possible. More detailed (and thus, costly) two- or three-dimensional finite element dynamic response and laboratory cyclic test-based analysis may be justified if liquefaction occurrence is not clearly precluded by the combination of stresses and estimated cyclic strengths.

As a result of improved understanding of residual strength of liquefied soils and the availability of large-deformation response analyses techniques, failure is not longer considered a necessary consequence of liquefaction in portions of earth dams or their foundations in lieu additional analysis. The strength of liquefied zones may contribute sufficient resistance to slope instability following earthquake shaking to prevent catastrophic release of a reservoir or even loss of function.
3.3.4 Cyclic Strain-Based Approach

Dabrey, et al. (1999) proposed a method to assess liquefaction potential based on the premise that densification and subsequent pore water pressure buildup will not occur in sands if the cyclic shear strain amplitude developed during earthquake shaking remains below a threshold value. The following basic steps were proposed:

1. Determine the amplitude and number of equivalent uniform cycles of shear strains generated with depth by the design earthquake.

2. Compare shear strains induced by the earthquake to the “threshold” shear strain by pore pressure buildup, determined for a number of clean sands to be approximately $10^{-2}\%$.

3. If earthquake-induced shear strains are expected to exceed the threshold level, further experimental evidence must be used to determine liquefaction occurrence.

The threshold strain concept only permits assessment of whether pore pressures will begin to accumulate, and the threshold strain level so defined is apparently a function of soil type. Hynes-Griffin (1988) performed strain-controlled cyclic triaxial tests on 15-inch diameter gravel specimens that revealed a characteristic threshold strain of $3 \times 10^{-3}\%$, roughly half of the sand threshold strain determined by Dabrey, et al. (1999).
Figure 3.1 Stress-strain and pore pressure response in monotonic triaxial tests on sands

(Department of the Army)
Figure 3.2 Schematic stress-strain curve for liquefaction failure by cyclic or monotonic loading (Marcuson, Hynes and Franklic, 1990, after Poulos, Castro and France, 1985)
Figure 3.3 Procedure for determining steady-state strength for soil at the field void ratio condition (Marcuson, Hynes and Franklic, 1990, after Poulos, Castro and France, 1985)
Figure 3.4 Effect of slight density variation on stress-strain behavior of clean sand in undrained triaxial compression (Poulos, Castro and France, 1985)
Figure 3.5 Typical steady state line for clean sand (Poulos, Castro and France, 1985)
Figure 3.6 Schematic of cyclic triaxial test equipment (Marcuson and Krinitzsky, 1976)
Figure 3.7 Typical analog recordings of load, deformation, and pore pressures during a cyclic triaxial test (Department of the Army, 1990)
Figure 3.8 Cyclic triaxial strength curves for Monterey No.0 sand (Department of the Army, 1990)
Figure 3.9 Chart for estimation of liquefaction resistance of clean sands using SPT data, for earthquakes of various Richter Magnitudes (Koester and Franklin, after Seed, et al. 1985)
Figure 3.10 Chart to determine SPT overburden pressure correction factor, $C_N$ (Seed, et al. 1985)
Figure 3.11 Current procedure for empirical liquefaction potential evaluation using the SPT (Koester and Franklin, 1985)
Figure 3.12 Chart used by Ishihara, Yasuda and Yokota (1981) to develop corrections to SPT-based liquefaction resistance correlations to account for mean particle size
Figure 3.13 Relation between cyclic strength and plasticity index for tailing with 30% fines (Ishihara, Yasuda and Yokota, 1981)
Figure 3.14 Relation between relative cyclic strength and number of cycles required to cause liquefaction (showing ordinate ratios for various magnitude earthquakes) (Seed, Idriss and Arango, 1983)
Figure 3.15 General concept of stress-based liquefaction potential evaluation (Seed and Idriss, 1982)
4. Factors affecting liquefaction resistance of sand

4.1 General

A monograph on the subject of ground motions and soil liquefaction by Seed and Idriss (1988a) provided a comprehensive list of the factors most often studied as influential on cyclic strength of soil, divided into three categories: (1) soil properties, including dynamic shear modulus and damping characteristics, unit weight, grain characteristics, relative density and soil structure (fabric); (2) environmental factors, such as mode of soil deposition, seismic history, geologic history (aging), coefficient of lateral earth pressure at rest, $K_o$, overconsolidation ratio, depth to water table, and effective confining pressure; and (3) earthquake characteristics, specifically ground shaking intensity and duration.

This chapter shows many factors in cyclic triaxial test and in the field. There are two categories in cyclic triaxial test. One is testing procedures affecting cyclic triaxial strength. Another is factors affecting triaxial strength. Main factors affecting liquefaction resistance of soils in the field are also discussed.

4.2 Testing Procedures Affecting Cyclic Triaxial Strength

4.2.1 Specimen Preparation Method

The effect of sample preparation on the cyclic strength of soils was presented by Ladd (1974). An electro hydraulic closed-loop loading system was used in his tests. Samples were prepared by two different specimen preparation methods to investigate their effect on the cyclic strength of three materials with different gradation. The specimen prepared by the wet tamping method was found to be always stronger than the specimen prepared by the dry vibration method.
Silver, et al. (2000) also proved that the cyclic strength of the specimen prepared by using the dry vibration method was on the order of half the strength of the specimen prepared by using the wet tamping method. The cyclic strength of the specimen prepared with the dry method did not increase significantly with increasing stress ratios.

Mulilis, et al. (1977) presented the most comprehensive studies regarding specimen preparation effects on the cyclic triaxial test. Six procedures with different specimen preparation methods were used in the stress-controlled cyclic triaxial tests. The effect of the method of sample preparation on the liquefaction characteristics was found to be significantly different. Differences in the cyclic stress ratio causing initial liquefaction of Monterey No. 0/30 Sand were found to be in the order of 100%. Generally speaking, the weakest specimens were formed by pluviating sand through air, while the strongest specimens were those formed by vibrating the soil in a moist condition.

Silver, et al. (2000) compared the cyclic strengths of specimens prepared by moist vibration, moist tamping, dry tamping, and dry vibration. The same conclusion as that of Mulilis, et al. (1977) was found.

Furthermore, Mulilis (1978) presented the data obtained on specimens of Monterey No. 0/30 Sand prepared by the moist rodding and the dry rodding methods. An increase of cyclic strength of approximately 50 percent at 10 cycles to cause initial liquefaction was noted. In the same publication, the effect of tamping foot size was also examined. However, no significant effect on the tamping foot size was found.
4.2.2 Effect of Reconstitution versus Intact Specimens

As specimen preparation procedure had a strong influence on cyclic triaxial strength (Mulilis, 1978), dilemma may arise as to what reconstitution method should be adopted for comparison here. Limited data using moist tamping and pluviation device through water to reconstitute specimens have shown that undisturbed specimens were slightly stronger than reconstituted specimens (Ishihara et al., 1978; Mulilis et al., 1978). It should be noted that cyclic triaxial strength of undisturbed specimens are subjected to such factors as degree of in-situ cementation and amount of disturbance during sampling.

4.2.3 Effect of Load Wave Forms

It has been found that wave forms of cyclic loading affect liquefaction resistance. Mulilis et al. (1978) compared the effects of rectangular, triangular, and sine wave loading as shown in Figure 4.1. In Figure 4.1, the order of increasing strength was rectangular, triangular, and sine, with triangular and sine wave loading strengths being 13 and 30% stronger than rectangular wave loading, respectively. Results of similar trend were also reported by other researchers (Lee and Fitton, 1989; Silver et al., 2000). The effect of loading wave form has been extensively studied by researchers and the results from these studies are quite similar.

Silver, et al. (2000) performed a series of cyclic triaxial tests using three different wave forms; (1) a sine wave; (2) a square wave with a very rapid rise time; and (3) a square wave with a degraded rise time whereby the unloading and loading portions of the wave did not have an instantaneous change in velocity. Results from these tests show that the cyclic soil strength is significantly affected by the shape of the loading wave as shown in Fig 4.1. Specimens tested using a fast rise time square wave showed cyclic strength values approximately 15% less than
those tested using a sine wave loading or a degraded square wave pattern. Examination of the pore pressure response recorded during a sharp square wave loading indicated that the instantaneous change in velocity caused a stress wave to propagate through the specimen. This stress wave was reflected in the form of pore pressure spikes. The more rapid buildup of pore pressure associated with the sharp square wave caused the sample to liquefy in a fewer number of cycles.

It was observed that if the rise time in the rectangular wave form was degraded such that the wave form did not have an instantaneous change of velocity in either the loading or unloading portion of the cycle, then the strength of specimens which were tested using the degraded wave form was approximately the same as that of specimens which were tested using the sine wave form. Due to the rapid jump in pore pressure associated with severe square wave loading, Silver recommended that a degraded square wave with a rise time of approximately 10% of the loading period or a sine wave loading be used in cyclic triaxial testing.

4.2.4 Effect of Frequency on Cyclic Strength

The effect of frequency over a range of 1/12 to 60 Hz on cyclic strength has been inconclusive with some researchers (Lee and Fitton, 1989; Lee and Focht, 1995) reported that slower loading frequencies produced slightly (< 10%) lower strength while others (Wulilis, 1975; Wong et al., 1975) reported otherwise. A study on the effect of frequency ranging from 0.00011 to 1 Hz showed that below 0.01Hz. Cyclic strength was independent of frequency effect while above 0.01 Hz, cyclic strength tend to increase with increasing frequency (Samuelson, 1981).

4.2.5 Effect of Specimen Size
A previous study concluded a height-to-diameter ratio of 2 is usually required. Lee and Fitton (1989) reported little effect on cyclic strength between specimen size of 1.4 and 2.8 inches in diameter. Larder (1999) however, reported lower liquefaction resistance in specimens with 2.8 inches diameter than those with 1.4 inches diameter due to the effect of membrane penetration. Another study by Wang et al. (2002) involved specimen size of 2.8 and 12 inches in diameter showed similar membrane penetration effect.

4.2.6 Effect of Frictionless Caps and Bases

The cap and base friction of the triaxial specimen might be different for sample of different diameters. The effect of caps and bases friction on cyclic strength has been reported to be insignificant (Mulilis, 1975).

4.2.7 Effect of Membrane Compliance

To minimize this effect, a relatively thick membrane was used to reduce the amount of initial penetration into the irregular sample surfaces.

Martin, et al. (1978) investigated the effect of system compliance on uniform sands. They concluded that membrane compliance affected well graded samples. In addition, samples containing a small proportion of gravel samples. In addition, samples containing a small proportion of gravel would produce a relatively large void on the sample surface, leading to a large increase in the apparent resistance to liquefaction.

4.3 Factors Affecting Cyclic Triaxial Strength

4.3.1 Effect of Relative Density
In one of the earliest laboratory cyclic triaxial study, Seed and Lee (1966) concluded that void ratio of a saturated sand strongly affected its liquefaction resistance – the higher the void ratio or the lower the relative density, the more easily liquefaction will occur. Lee and Seed (1967) reported that cyclic stress required causing initial liquefaction increased linearly to approximately 60% relative density. Other study showed that the stress ratio to cause liquefaction in 10 cycles is linear with relative density to approximately $D_r = 70\%$ (Mulilis, 1975). The paramount importance of relative density as a parameter of liquefaction resistance was evidenced in various empirical correlations based on observations during previous earthquakes for the evaluation of liquefaction potential (Kishida, 1969; Castro, 1975; Seed and Idriss, 1981; Tokimatsu and Yoshimi, 1983). In these correlations, SPT-N value, which has been shown to relate to relativity density (Gibbs and Holtz, 1957) of soil, is invariably used as an indicator of soil strength liquefaction. In all these correlations, the lower the SPT-N values or the lower the relative density, the lower the liquefaction resistance.

### 4.3.2 Effect of Confining Stress ($\sigma_3$)

Seed and Lee (1966) reported that liquefaction resistance of a saturated sand was affected by the confining pressure acting on the sand- the lower the confining pressure the more easily liquefaction will develop. The effect of confining pressure on liquefaction resistance of soils as concluded above is consistent with the fact that soil strength increases with confining pressure. However, confusion may arise if cyclic stress ratio instead of absolute cyclic stress amplitude is used to designate intensity of cyclic loading. In using equivalent uniform stress cycle concept (Seed et al., 1975; Annaki and Lee, 1977) for soil liquefaction analysis, it is convenient to express in-situ cyclic loading in terms of cyclic stress ratio which is a ratio of cyclic shear stress amplitude to effective overburden pressure. In a one-dimensional simplification, a magnitude of
earthquake induced cyclic shear stress in a soil is in direct proportion to effective overburden pressure it is subjected to (Seed and Idriss, 1967). In laboratory triaxial condition, effective overburden pressure in the field can be simulated by effective confining pressure if in-situ coefficient of lateral earthquake, K is equal to unity. Applicability of laboratory triaxial condition for different in-situ K values was discussed by Seed and Peacock (1970). Therefore, in the event of an earthquake shaking, soils under higher effective overburden pressure or effective confining pressure will in general experience higher shear stress amplitude and vice versa. Due to this confining pressure dependency, stress ratio, being a confining pressure normalized parameter is apparently a better indicator for liquefaction resistance under earthquake loading. Observations in laboratory have confirmed that cyclic stress ratio required to cause liquefaction decreases with increasing confining pressure (Castro and Poulos, 1976; Mulilis et al., 1977). It can be concluded that when cyclic stress ratio is used to designate cyclic loading intensity the lower the confining pressure the stronger the liquefaction resistance. As a matter of fact, the difference here is whether absolute cyclic stress amplitude or cyclic stress ratio is used as loading intensity. Use of absolute stress amplitude to indicate liquefaction resistance may be appropriate in the study of static loading induced liquefaction, cyclic stress ratio is nevertheless more realistic when earthquake induced liquefaction is of concern.

4.3.3 Effect of Cyclic Stress Amplitude and Number of Cyclic Stress Cycles

In their laboratory study, Seed and Lee (1966) concluded that the larger the stress or strain, the lower the number of cycles required to induce liquefaction. Also the more the number of stress cycles to which the sand is subjected the more likely the liquefaction failure will occur. These two factors are directly related to the magnitude of cyclic loading. The effect of earthquake magnitude on liquefaction resistance of soils is apparent based on concept of
cumulative damage proposed by Miner (1945). Applicability of Miner’s (1945) cumulative
damage concept in soil liquefaction analysis was confirmed in studies concerning the validity of
equivalent uniform stress cycle concept (Seed et al., 1975; Annaki and Lee, 1997).

4.3.4 Effect of Particle Size and Gradation

Studies conducted by several researchers (Leed and Fitton, 1969; Wong et al., 1975;
Ishihara et al., 1978) suggested that cyclic strength is the lowest with mean grain size, \( D_{50} \) near
0.1 mm. increase or decrease in \( D_{50} \) from 0.1mm tends to increase cyclic strength. Wang et al.
(2002) also found that contrary to their expectation, well-graded material was somewhat weaker
than uniformly graded material. This unexpected observation was attributed to possible higher
densification tendency and smaller membrane penetration effect in well-graded material which
favored pore pressure generation.

4.3.5 Effect of Pre-straining

Fanner et al. (2003) found that once a specimen has liquefied and reconsolidated to a
denser structure, despite this densification, the specimen is much weaker to reliquefaction.
Similar observation was also reported by Lee and Focht (1975). Study conducted by Mori et al.
(1977) showed that specimens with prestraining by applying several loading cycles without
causing liquefaction then releasing excess pore pressure for consolidation exhibited stronger
cyclic strength than those specimen without prestraining.

4.3.6 Effect of Lateral Earth Pressure (\( K_0 \)) and Over consolidation Ratio

A Study on a dense sand by Lee and Focht (1999) indicated an increase in cyclic stress
ratio of about 30% for an OCR of 3. Ishihara et al. (1978) showed that cyclic strength increased
as OCR and fines content increased. For specimens with no fines, a strength increase of 30% was observed for an increase in OCR from 1 to 2, while for the same OCR increase an 80% increase in cyclic strength was observed for specimens with 100% fines. Similar results produced from cyclic simple tests were reported by Seed and Peacock (1971).

R. Segaldo et al. (1999) showed that the effect of $K_o$ on cyclic resistance of clean, uncemented, normally consolidated sand with $D_R$ of 30-95% can reasonably be taken into account by normalization with respect to the mean consolidation effective stress. When a change in $K_o$ is associated with overconsolidation, there is an additional increase in cyclic resistance that is probably due to a prestraining effect on the fabric or grain structure of the sand. The experimental data suggest that this additional increase in cyclic resistance ranges from about 10-40% at an OCR of 2 to about 25-100% at an OCR of 4. This range in the data may be partly due to differences in soil, testing equipment, or stress path during consolidation and cyclic loading, indicating further research is necessary to quantify this effect more accurately.

### 4.3.7 Effect of Consolidation Ratio, $K_c$

To simulate stress condition in an embankment, anisotropic consolidation of specimen is required (Seed et al. 1975). In their earlier study regarding level ground liquefaction, Seed and Peacock (1970) pointed out that cyclic triaxial test can produce desired stress changes only by consolidating the specimen initially under isotropic condition. Under this condition, a constant normal stress and a controlled and continuously changing shear stress may be imposed along a 45 degree plane in the specimen. If any other consolidation pressure is used, there will be no plane in the specimen which will receive desired symmetrical changes in shear stress. In case of initially anisotropic stress condition, cyclic simple shear test can better simulate one-dimensional cyclic loading condition. However, stress variations due to earthquake can be very complicated.
in an embankment. One-dimensional simplification is not appropriate and no proper test can be devised unless stress variations during earthquake can be realistically simulated.

Castro and Poulos (1999) found that samples consolidated under higher $K_c$ would require a smaller increment in stresses to cause liquefaction, because at a higher $K_c$, the specimen is closer to failure.

4.4 Main Factors Governing Liquefaction in the Field

There are many factors that govern the liquefaction process for in situ soil. Based on the results of laboratory tests as well as field observations and studies, the most important factors that govern liquefaction are as follows:

4.4.1 Earthquake intensity and duration

In order to have liquefaction of soil, there must be ground shaking. The character of the ground motion, such as acceleration and duration of shaking, determines the shear strains that cause the contraction of the soil particles and the development of excess pore water pressures leading to liquefaction. The most common cause of liquefaction is due to the seismic energy released during an earthquake. The potential for liquefaction increases as the earthquake intensity and duration of shaking increase.

Although data are sparse, there would appear to be a shaking threshold that is needed to produce liquefaction. These threshold values were a peak ground acceleration $a_{\text{max}}$ of about 0.10g and local magnitude $M_L$ of about 5 (National Research Council 1985, Ishihara 1985). Thus, a liquefaction analysis would typically not be needed for those sites having a peak ground acceleration $a_{\text{max}}$ less than 0.10g or a local magnitude $M_L$ less than 5.
4.4.2 Groundwater Table

The condition most conducive to liquefaction is a near-surface groundwater table. Unsaturated soil located above the groundwater table will not liquefy. If it can be demonstrated that the soils are currently above the groundwater table and are highly unlikely to become saturated for given foreseeable changes in the hydrologic regime, then such soils generally do not need to be evaluated for liquefaction potential.

At sites where the groundwater table significantly fluctuates, the liquefaction potential will also fluctuate. Generally, the historic high groundwater level should be used in the liquefaction analysis unless other information indicates a higher or lower level is appropriate (Division of Mines and Geology 1997).

Poulos et al. (1985) stated that liquefaction can also occur in very large masses of sands or silts that are dry and loose and loaded so rapidly that the escape of air from the voids is restricted. Such movement of dry and loose sands is often referred to as running soil or running ground. Although such soil may flow as liquefied soil does, in this text, such soil deformation will not be termed liquefaction. It is best to consider that liquefaction only occurs of soils that are located below the groundwater table.

4.4.3 Soil Types

In term of the soil types most susceptible to liquefaction, Ishihara (1985) states: “the hazard associated with soil liquefaction during earthquakes has been known to be encountered in deposits consisting of fine to medium sand and sands containing low plasticity fines.” Occasionally, however, cases are reported where liquefaction apparently occurred in gravelly soils.
Thus, the soil types susceptible to liquefaction are nonplastic (cohesionless) soil. An approximate listing of cohesionless soils from least to most resistant to liquefaction is clean sands, nonplastic silty sands, nonplastic silt, and gravels. There could be numerous exceptions to this sequence. For example, Ishihara (1985, 1993) described the case of tailings derived from the mining industry that were essentially composed of ground-up rocks and were classified as rock flour. Ishihara (1985, 1993) stated that the rock flour in a water-saturated state did not possess significant cohesion and behaved as if it were a clean sand. These tailings were shown to exhibit as low a resistance to liquefaction as clean sand.

Seed et al. (1983) stated that based on both laboratories testing and field performance, the great majority of cohesive soils will not liquefy during earthquakes. Using criteria originally stated by Seed and Idriss (1982) and subsequently confirmed by Youd and Gilstrap (1999), in order for a cohesive soil to liquefy, it must meet all the following three criteria:

- The soil must have less than 15 percent of the particles, based on dry weight, that are finer than 0.005 mm (i.e., percent finer at 0.005 mm < 15 percent).
- The soil must have a liquid limit (LL) that is less than 35 (that is, LL < 35).
- The water content $\omega$ of the soil must be greater than 0.9 of the liquid limit [that is, $\omega > 0.9 \times (LL)$].

If the cohesive soil does not meet all three criteria, then it is generally considered to be not susceptible to liquefaction. Although the cohesive soil may not liquefy, there could still be a significant undrained shear strength loss due to the seismic shaking.

4.4.4 Soil Relative Density $D_r$
Based on field studies, cohesionless soils in a loose relative density state are susceptible to liquefaction. Loose nonplastic soils will contract during the seismic shaking which will cause the development of excess pore water pressure.

For dense sands, the state of initial liquefaction does not produce large deformations because of the dilation tendency of the sand upon reversal of the cyclic shear stress. Poulos et al. (1985) stated that if the in situ soil can be shown to be dilative, then it need not evaluated because it will not be susceptible to liquefaction. In essence, dilative soils are not susceptible to liquefaction because their undrained shear strength is greater than their drained shear strength.

4.4.5 Particle Size Gradation

Uniformly graded nonplastic soils tend to form more unstable particle arrangements and are more susceptible to liquefaction than well-graded soils. Well-graded soils will also have small particles that fill in the void spaces between the large particles. This tends to reduce the potential contraction of the soil, resulting in less excess pore water pressure being generated during the earthquake. Kramer (1996) states that field evidence indicates that most liquefaction failures have involved uniformly graded granular soils.

4.4.6 Placement Conditions or Depositional Environment

Hydraulic fills (fill placed under water) tend to be more susceptible to liquefaction because of the loose and segregated soil structure created by the soil particles falling through water. Natural soil deposits formed in lakes, rivers, or the ocean also tend to form a loose and segregated soil structure and are more susceptible to liquefaction. Soils that are especially susceptible to liquefaction are formed in lacustrine, alluvial, and marine depositional environments.
4.4.7 Drainage Conditions

If the excess pore water pressure can quickly dissipate, the soil may not liquefy. Thus highly permeable gravel drains or gravel layers can reduce the liquefaction potential of adjacent soil. Most laboratory studies on soil liquefaction simulate only element behavior and usually impose a perfectly undrained condition. The possibility of partial dissipation and redistribution of excess pore pressure during earthquake shaking has been suggested by Seed (1987). At extreme condition looser zone may reach a virtually liquid state and exhibit a lower than perfectly undrained residual strength. Absolute undrained condition in laboratory may lead to unconservative results because it is unable to consider this aspect of liquefaction phenomenon. Recently, geotechnical centrifuge testing has become widely used (Hushmand et al., 1988; 1989; Law, 1991; Ko and Mclean, 1991). In-situ liquefaction behavior can now be better studied with the aid of centrifuge modeling which has revealed valuable information towards a better understanding of field liquefaction characteristics.

4.4.8 Confining Pressure

The greater the confining pressure, the less susceptible the soil is to liquefaction. Conditions that can create a higher confining pressure are a deeper ground water table, soil that is located at a deeper depth below ground surface, and a surcharge pressure applied at ground surface. Case studies have shown that the possible zone of liquefaction usually extends from the ground surface to a maximum depth of about 50 ft (15m). Deeper soils generally do not liquefy because of the higher confining pressures.

This does not mean that a liquefaction analysis should not be performed for soil that is below a depth of 50 ft (15m). In many case, it may be appropriate to perform a liquefaction
analysis for soil that is deeper than 50 ft (15m). In addition, a liquefaction analysis should be performed for any soil deposit that has been loosely dumped in water (i.e., the liquefaction analysis should be performed for the entire thickness of loosely dumped fill in water, even if it exceeds 50 ft in thickness). Likewise, a site where alluvium is being rapidly deposited may also need a liquefaction investigation below a depth of 50 ft (15m). Considerable experience and judgment are required in the determination of the proper depth to terminate a liquefaction analysis.

4.4.9 Particle Shape

The soil particle shape can also influence liquefaction potential. For example, soils having rounded particles tend to densify more easily than angular-shape soil particles. Hence a soil containing rounded soil particles is more susceptible to liquefaction than a soil containing angular soil particles.

4.4.10 Aging and Cementation

Newly deposited soils tend to be more susceptible to liquefaction than older deposits of soil. It was shown that the longer a soil is subjected to a confining pressure, the greater the liquefaction resistance (Ohsaki 1969, Seed 1979a, Yoshimi et al. 1989). Table 3.1 presents the estimated susceptibility of sedimentary deposits to liquefaction versus the geologic age of the deposit.

The increase in liquefaction resistance with time could be due to the deformation or compression of soil particles into more stable arrangements. With time, there may also be the development of bonds due to cementation at particle contacts.
Aging and cementation can strengthen in-situ liquefaction resistance of soil deposits. Field studies on previous liquefaction failure in Japan and China (Fu & Tatsuoka, 1984; Kuribayashi & Tatsuoka, 1975; Ohsaki, 1970) have shown high liquefaction susceptibility in recent deposits and reclaimed land. In the 1976 Tangshan earthquake, it was reported that recent alluvial river fans experienced severe liquefaction failure while virtually no sign of liquefaction failure was observed in oldest river fans (Chang, 1990).

4.4.11 Historical Environment

It has also been determined that the historical environment of the soil can affect its liquefaction potential. For example, older soil deposits that have already been subjected to seismic shaking have an increased liquefaction resistance compared to a newly formed specimen of the same soil having an identical density (Finn et al. 1970, Seed et al. 1975).

Liquefaction resistance also increases with an increase in the overconsolidation ratio (OCR) and the coefficient of lateral earth pressure at rest $k_0$ (Seed and Peacock 1971, Ishihara et al. 1978). An example would be the removal of an upper layer of soil due to erosion. Because the underlying soil has been preloaded, it will have a higher overconsolidation ratio and it will have a higher coefficient of lateral earth pressure at rest $k_0$. Such a soil that has been preloaded will be more resistance to liquefaction than the same soil that has not been preloaded.

4.4.12 Building Load

The construction of a heavy building on top of a sand deposit can decrease the liquefaction resistance of the soil. For example, suppose a mat slab at ground surface supports a heavy building. The soil underlying the mat slab will be subjected to shear stresses caused the building load. These shear stresses induced into the soil by the building load can make the soil
more susceptible to liquefaction. The reason is that a smaller additional shear stress will be required from the earthquake in order to cause contraction and hence liquefaction of the soil. For level-ground liquefaction discussed in this chapter, the effect of the building load is ignored. Although building loads must be included in all liquefaction-induced settlement, bearing capacity, and stability analyses.

Figure 4.1 Effect of loading wave form on cycles to initial liquefaction for moist-tamped specimens
Table 4.1 Estimated susceptibility deposits to liquefaction during strong seismic shaking based on geologic age and depositional environment

<table>
<thead>
<tr>
<th>Type of deposit</th>
<th>General distribution of cohesionless sediments in deposits</th>
<th>Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by age of deposit)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt;500 years</td>
<td>Holocene</td>
</tr>
<tr>
<td>Alluvial fan and plain</td>
<td>widespread</td>
<td>Moderate</td>
</tr>
<tr>
<td>Delta and fan-delta</td>
<td>widespread</td>
<td>High</td>
</tr>
<tr>
<td>Dunes</td>
<td>widespread</td>
<td>High</td>
</tr>
<tr>
<td>Marine terrace/plain</td>
<td>widespread</td>
<td>Unknown</td>
</tr>
<tr>
<td>Talus</td>
<td>widespread</td>
<td>Low</td>
</tr>
<tr>
<td>Tephra</td>
<td>widespread</td>
<td>High</td>
</tr>
<tr>
<td>Colluviums</td>
<td>variable</td>
<td>High</td>
</tr>
<tr>
<td>Glacial till</td>
<td>Variable</td>
<td>Low</td>
</tr>
<tr>
<td>Lacustrine and playa</td>
<td>Variable</td>
<td>High</td>
</tr>
</tbody>
</table>
Table 4.1 (con’t)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Loess</th>
<th>Floodplain</th>
<th>River channel</th>
<th>Sebka</th>
<th>Residual soils</th>
<th>Tuff</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High</td>
<td>High</td>
<td>High</td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
</tr>
<tr>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Low</td>
<td>Very low</td>
<td>Very low</td>
</tr>
</tbody>
</table>

|                  | Locally variable | Moderate       | Low           | Very low       | Very low     |
|                  | Very high       | High          | Low           | Very low       | Very low     |

|                  | Locally variable | High          | Moderate       | Low           | Very low       | Very low |
|                  | High            | Moderate       | Low           | Very low       | Very low     |

|                  | Locally variable | High          | Moderate       | Low           | Very low       |
|                  | High            | Moderate       | Low           | Very low       | Very low     |

|                  | Locally variable | High          | Moderate       | Low           | Very low       |
|                  | High            | Moderate       | Low           | Very low       | Very low     |

|                  | Rare           | Low           | Low           | Very low       | Very low     |

|                  | Rare           | Low           | Low           | Very low       | Very low     |

(b) Coastal zone

<table>
<thead>
<tr>
<th>Variable</th>
<th>Beach-large waves</th>
<th>Beach-small waves</th>
<th>Delta</th>
<th>Estuarine</th>
<th>Foreshore</th>
<th>Lagoonal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Widespread</td>
<td>Widespread</td>
<td>Very high</td>
<td>High</td>
<td>High</td>
<td>Locally variable</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>High</td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>High</td>
</tr>
<tr>
<td></td>
<td>Low</td>
<td>moderate</td>
<td>Low</td>
<td>Low</td>
<td>Very low</td>
<td>Moderate</td>
</tr>
<tr>
<td></td>
<td>Very low</td>
<td>Very low</td>
<td>Very low</td>
<td>Very low</td>
<td>Very low</td>
<td>Low</td>
</tr>
</tbody>
</table>

(c) Artificial

<table>
<thead>
<tr>
<th>Variable</th>
<th>Compacted fill</th>
<th>UnCompacted fill</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low</td>
<td>High</td>
</tr>
<tr>
<td></td>
<td>Unknown</td>
<td>Unknown</td>
</tr>
<tr>
<td></td>
<td>Unknown</td>
<td>Unknown</td>
</tr>
</tbody>
</table>

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5 Cyclic Triaxial Test Procedure, Program and Results for Sand

5.1 Introduction

In a cyclic triaxial test, a cylindrical specimen of soil encased in a rubber membrane is placed in a chamber, subjected to a confining fluid pressure, and then applying dynamic loading axially until soil failure. The axial load may be applied to the sample through a rigid top platen. The axial force can be compression or extension: thus the axial stress can be either major or minor principal stress. Usually the top platen is laid over a porous stone which allows fluid to flow in and out of the specimen. The axial deformation of the specimen is directly monitored by the movement of the piston which is in contact with or connected to the top platen. The lateral deformation is not usually measured. Transducers are used for pore pressure measurement.

In a cyclic triaxial test, a sample is consolidated under an initial isotropic confining pressure. The confining pressure is kept constant and axial load is either increased (compression test) or decreased (extension test) during a test. Thus, two of three principal stresses are always equal during a test. In a compression test the intermediate principal stress is equal to the minor principal stress; and the axial stress is equal to the major principal stress. In an extension test the major and the intermediate principal stress are equal, while the axial stress is equal to minor principal stress.

A variety of modified tests can be conducted in a conventional triaxial apparatus. Bishop and Henkel (1962) proposed several modified triaxial test. To simulate field conditions, a test can be performed by keeping the axial stress constant, while decreasing the confining pressure. Consolidation can be conducted under hydrostatic condition or at any ratio of axial-to-lateral stress. A triaxial test can be conducted at any ratio of principal stresses while keeping their mean stress constant. By conducting these tests, a wide variety of stress paths can be obtained.
5.2 Relative Density Control

To ensure that representative and accurate results were obtained from the triaxial test, a specified procedure was determined and followed.

Relative density is expressed as: \[ D_r = \frac{\gamma_{\text{max}}}{\gamma_d} \times \frac{\gamma_{\text{d}} - \gamma_{\text{min}}}{\gamma_{\text{max}} - \gamma_{\text{min}}} \times 100\% \] \[5.1\]

Where: 
- \( D_r \) = relative density (in percent) 
- \( \gamma_{\text{max}} \) = maximum dry density (unit weight) of the soil 
- \( \gamma_{\text{min}} \) = minimum dry density (unit weight) of the soil 
- \( \gamma_{\text{d}} \) = in-place dry density (unit weight) of the soil

To determine the air dry weight of soil necessary to create the sample, the following equation was used:

\[ \gamma_{\text{d}} = \frac{w_s}{\pi (D_s^2) (H_s)/4} \] \[5.2\]

Where \( \gamma_{\text{d}} \) = unit dry weight of the sample at desired relative density
- \( w_s \) = dry weight of specimen
- \( D_s \) = diameter of specimen
- \( H_s \) = height of specimen

The diameter and height of the specimen are theoretical final measurements and were determined by calculating the mold size and subtracting the membrane thickness. Several trial samples were prepared initially to ensure proper density was being achieved.

5.3 Preparation of Triaxial Samples

Extreme care must be exercised during the complete process of sample preparation. All necessary equipment and supplies are properly cleaned and arranged on the countertop for easy access and are checked for their working conditions. All samples are prepared by following the procedure outlined below.
(1) Membrane quality control. Before use, each membrane is carefully examined to see if there is any hole. To detect any hole in a membrane, it is sealed onto the base pedestal at one end and onto the loading cap at the other end with silicon grease and rubber 0-rings. The rubber 0-ring shown in Figure 5.1. The membrane is slowly inflated by filling it with water. After the membrane is inflated with water, the outside surface is dried, so that any water drops seeping through the membrane can be detected. After a membrane is thoroughly checked and found to contain no hole, its inner and outer surface are then dried to prevent any soil particles from sticking to the surface during sample preparation process.

(2) Average membrane thickness. Thickness of the rubber membrane was measured at each end of the membrane using the vernier caliper shown in Figure 5.2. Each membrane must be carefully measured for its thickness. For different measurements are taken, two at the top corners and two at the bottom corners, for its double thickness. An average double thickness is calculated by averaging the four measurements.

(3) Obtain two dry porous stones. Moist porous stone may result in sample non-uniformity due to capillary if not properly dry; the result may be a non-uniform sample.

(4) Use the dry porous stone as a template and cut two discs of filter paper to a size slightly smaller than the stone. During the placement of soil, an over-size filter paper disc tends to be lifted up by the surrounding tuber membrane and result in a void between the filter paper disc and the stone. On the other hand, if a filter paper disc is too much smaller in diameter than the porous stone, a soil will come in direct contact
with the porous stone around the perimeter of the filter paper disc. This may result in the blockade of the porous stone drainage paths.

(5) Measure the total height of the cell base, two porous stones, two filter paper discs and loading cap using a precision caliper shown in Figure.5.3. Stack them sequentially and check the final stack for level. Measure the total height from the countertop to nearest 0.001” at four different locations. Average these four measurements. This average height is termed an initial height without sample and will be used in subsequent steps to determine the final sample height.

(6) Determine the water content (w). the water content of a soil mixed can be determined using a sample procedure out-lined as follows:

a. Weight a small amount of a moist soil \(w_i\) in a pre-weighed bowl.

b. Beat the soil in a microwave oven for about 10 minutes to remove all of moisture.

Determine the final weight \(w_f\) of the dry soil. The water content is determined as:

\[
\text{w} = \frac{(w_i - w_f)}{w_f}
\]

The water content of a moist soil mix can be maintained relatively constant by sealing the container and then placing it in a 100-percent moisture room.

(7) Determine the amount of soil required to achieve a desired void ratio.

a. At a desired void ratio, \(e_d\), a desired dry unit weight of a soil mix, \(r_d\), can be determined as:

\[
\text{r}_d = \frac{G_s \times w}{1 + \text{ed}}
\]

Where \(G_s\) is the weighted specific gravity of the soil mix.

b. The volume of the sample diameter as: 

\[
V = \pi D^2 H / 4
\]

Where \(D\) is the estimated sample diameter and \(H\) is the estimated sample height. \(D\) can be obtained by subtracting the double membrane thickness from the average
inner diameter of the sample perpendicular directions. The average inner diameter of the mold is obtained by taking the average of two diameter measurements at two perpendicular directions. A caliper is used in this diameter measurement. A proper required sample height H, should allow an easy access to the top porous stone and filter paper. Usually it is preferred to expose the porous stone slightly above the sample mold. Thus, H depends on the height of the mold, and it should be a little less than the measured height from the bottom of the lower porous stone to the top of the mold. This height will allow for the easy removal of the stone and the filter paper disc during the final stage of sample preparation.

c. The weight of dry soil required is calculated as: \( W_d = \gamma_d \times V \)

d. Since moisy tamping method is adopted for compacting moist soil, the required weight of moist soil, \( W_{\text{wet}} \), is calculated as: \( W_{\text{wet}} = W_d \times (1 + W) \) where W is the water content of the moist soil mix.

(8) Apply a thin layer of grease a high vacuum silicon as sealant around the side of a base pedestal shown in Figure 5.4.

(9) Place a porous stone and then a piece of filter paper disc on top of the base pedestal.

(10) Stretch the membrane over the base pedestal so that it adheres uniformly on the side wall of the base pedestal. Carefully smooth out the membrane over the base pedestal and eliminate any air voids along the interface. Make sure that the membrane seats straight up on the base shown in Figure 5.5.

(11) Carefully place two 0-rings in the 0-ing grooves on the base pedestal groves. This will help secure the membrane to the base pedestal and prevent leakage.
(12) Apply a thin layer of grease sealant to the edges of the split mold used to from a sample.

(13) Place the mold around the base pedestal

(14) Tighten the two halves of the split mold together using pipe clamps shown in Figure 5.6.

(15) Wrap the top portion of the membrane over the top of the mold and fold the excess membrane round the outside of the mold to ensure a smooth surface when the vacuum is applied to the mold. This step involves stretching membrane with four fingers, two from each hand. The rubber membrane should be stretched equally outward, and wrap around the top of the mold.

(16) Apply a vacuum of 10 psi, so that the membrane adhere on the inner wall of the mold. Make sure that there are no wrinkles on the membrane shown in Figure 5.7.

(17) Check the top of the mold for level and measure the distance from the surface of the filter paper disc inside the mold to the top of the mold, $H_t$.

(18) The zero raining device was placed within the mold as shown in Figure 5.8 and the sand previously weighed was poured into the zero raining device. By slowly and smoothly lifting up the zero raining device, the sand fell through the screened opening on the end of the zero raining device.

(19) Level the sample top, place a piece of a filter paper disc on top of the sample followed by a porous stone shown in Figure 5.9.

(20) Apply a thin layer of grease sealant around the side of the loading cap.
(21) Place the loading cap on top of the top porous stone shown in Figure 5.10. Make sure the tube connecting to the top cap is located in such a manner that it will not hinder the chamber assembly.

(22) Check the top of the cap for level in two perpendicular directions. The maximum tilting of the sample top is limited to 0.002 times the diameter of the specimen.

(23) Check the sample height against the initial height without sample as obtained in step (6). The difference has to be very close to the desired sample height, \( H \), to ensure a good density control.

(24) Carefully pull the membrane over the top cap and use two o-rings (already placed over the cap and on the line) to secure the membrane to the loading cap. Be sure to keep the membrane smooth and free of any air bubbles.

(25) The vacuum should still be on the mold.

(26) While the mold still under vacuum, apply vacuum 10 psi to the sample from the top cap.

(27) Release the vacuum to the mold.

(28) Recheck the level shown in Figure 5.11 and the sample height to see if the applied vacuum cause excessive volume compaction.

(29) Carefully pry open and remove the mold and check the sample for any irregularity which could later cause any problem. Also check to see if sample is vertical.

(30) Use the vernier caliper to measure the sample height at four different perpendicular locations on top of the loading cap shown as in Figure 5.12. Average the four measurements as the final height with sample.
(31) Use a pi-tap shown in Figure 5.13 to measure the sample diameter with membrane. Measurements are taken at three locations: near the top, near the middle and near the bottom of the sample. The average sample diameter with membrane is calculated by using the following formula: $\frac{1}{4} \times (\text{top diameter} + 2\times \text{mid-height diameter} + \text{bottom diameter})$

The final sample diameter is determined by subtracting the double membrane thickness from the average diameter of the specimen with membrane. Knowing the final specimen diameter and height, the specimen volume is determined. The dry unit weight of the specimen is determined by dividing the specimen dry weight by the specimen volume. The specimen void ratio is then determined using the following equation: $\gamma_d = (G_s \times \gamma_w)/(1+e)$

(32) Apply a vacuum of 5 psi to the top a container half filled with deair water. Connect the water supply line from this container to the bottom of the sample. The value at the bottom of the sample is controlled in such a way that the water can be drawn from the container into the sample and out from the top of the sample at a very slow rate to prevent any significant loss of fine, when the water is observed to flow out from the top of the sample shown in Figure 5.14, procedures (29) and (30) are repeated to check and record any volume change due to flushing. Attempt is made to achieve a +/- 2% error in void ratio after flushing.

(33) Grease the large “0” ring to be placed on the triaxial cell base to provide good seal between the chamber and the base.

(34) Clean the interface area between the chamber and the cell base and apply a thin layer of grease sealant.
(35) Recheck the interface area to make sure that it is free of any sand particles.

(36) Plug an open tube to the top of the triaxial cell chamber to prevent any pressure building-up, carefully place the chamber over the sample, and lock it onto the triaxial cell base using a rim locking band.

(37) Lower the loading ram gently to check if the ram and sample loading cap are properly aligned. Then raise the ram and lock it in place with a piston lock. Before placing the triaxial cell chamber onto the base, it is absolutely critical to check if the loading ram is raised to the highest possible position and securely locked. This is to prevent the loading ram to come in contact with the top cap and severely disturbed the sample.

(38) Fill the chamber with water, until water flows out of the top of the chamber through the open tube.

(39) After the chamber is properly filled, close the valve leading to the filling chamber and remove the open tube from the top of the chamber.

(40) Close the vacuum line connecting to the top of the sample and allow the water, under 5 psi vacuum, to continued to flow into the bottom of the sample.

(41) Apply a confining pressure of 5 psi disconnect the water line under 5 psi vacuum at the bottom of the sample, and connect a water supply line to the bottom of the sample to allow desired water to flow into the sample under atmospheric pressure, until the flow of water ceases. At this moment the vacuum in the sample is completely released.

(42) Increase the confining pressure to 15 psi and allow the sample to consolidate under 15 psi.
(43) Connect the bottom of the sample to beck pressure line. Make sure both the 
confining pressure and back pressure burettes are filled with water.

(44) Connect the four-way transducer value shown in Figure 5.15 to the bottom of the 
sample.

(45) Before mounting the transducer, saturate all lines connecting to the four-way 
transducer valve with water.

(46) Saturate the four-way transducer valve with water by turning it upside down and 
gently tapping the side of the value to help getting rid of trapped air. This four-way 
transducer value is used to measure the magnitude of the confining pressure, the back 
pressure to be applied to the bottom of the sample, and the pore pressure at the top pf 
the sample using a digital multimeter during cyclic testing.

(47) Now simultaneously raise the confining and back pressures in such a manner as to 
maintain a 10psi effective stress in the sample. In order to minimize sample 
disturbance due to sudden pressure increase, raise the pressure in a 10 psi increment 
roughly every 2-3 minutes.

(48) Continue to raise the back pressure until it finally reaches 100 psi.

(49) Allow the pressure in the sample to reach an equilibrium and then check the B 
parameter. B value usually can reach better than 0.99 upon the application of the final 
increment of back pressure. This proves that the sample saturation procedure is very 
time effective in achieving a satisfactory degree of saturation.

5.4. Saturation

Water levels in the burettes connected to the cell and back pressure line were filled to 
appropriate levels.
To limit the amount of stress the specimen undergoes during the saturation phase, the back pressure and cell pressure were increased simultaneously, maintaining the cell pressure 15psi greater than the back pressure. This procedure was carried out slowly so that the pore pressure throughout the specimen was maintained the equilibrium.

In most cases, the back and cell pressures were simultaneously raised to 75 and 90 psi respectively, following which time the specimen was allowed to saturate overnight.

5.5 Determining B-Parameter

After saturating for approximately 24 hours, the B-Parameter was checked. With back pressure drainage lines closed, the cell pressure was increased by0.5mv. by measuring the corresponding rise in the specimen’s pore water pressure, Δu, the B parameter was calculated using the following equation:

\[ B = \frac{\Delta u}{\Delta \sigma_3} \]

where B= Skempton’s pore pressure parameter

\[ \Delta u = \text{rise in pore water pressure, change in backpressure} \]

\[ \Delta \sigma_3 = \text{change in cell pressure, change in confining} \]

It was desirable to obtain a B parameter of greater than or equal to 0.95 before the specimen was considered saturated. For triaxial samples in this paper, after saturating for approximately 1-2 days, the B parameters were equal to or greater than 0.95.

5.6 Test Equipments

5.6.1 Basic Principles of the MST Closed Loop System
A series 810, Material Test System furnished by the MST Systems Corporation, Minneapolis, Minnesota as shown in Fig. 5.16 was used to apply the cyclic dynamic loading to the sand specimens.

5.6.2 Hydraulic Power Supply

A fixed-volume pump supply fluid pressure to the system. The hydraulic power supply may be operated locally, through use of its own controls, or via the MTS remote control panel which was the case during tests which I performed.

Two levels of operation are provided: an output pressure of 300 psi for the low or bypass condition and an output pressure of 3000 psi for the high condition. A safety pressure control valve protects the power supply from the buildup of excessive pressure.

A fluid-to-water heat exchanger is used by the hydraulic power supply to maintain the reservoir hydraulic pressure below a maximum safe temperature. A temperature-sensitive switch mounted on the reservoir will open and turn off the hydraulic power supply if the hydraulic fluid temperature exceeds a predetermined limit.

5.6.3 Hydraulic Actuator

The hydraulic actuator is the force-generating and/or positioning device in the system. Movement of loading piston is the direct result of the application of fluid pressure to one side of the piston. A load applied to some external reaction point by the piston is equal to the effective piston area times the activating pressure.

5.6.4 Servovalve
The hydraulic actuator is controlled by the opening and closing of the servovalve in response to a control signal from the valve driver or controller. The servovalve can open in either of two positions, thereby permitting high pressure fluid to enter into either side of the piston. This alternating application of hydraulic pressure to either side of the piston makes it possible to apply smooth cyclic tension and compressive loads to a test specimen. When the servovalve is opened to allow fluid to flow into one end of the cylinder, the valve on the opposite end of the cylinder is opened to provide a path for fluid to flow back to the hydraulic power supply.

The rate of fluid flow through the servovalve is in direct proportion to the magnitude of the control signal. The polarity of the control signal determines which end of the actuator cylinder will receive additional fluid thereby determining the direction of the piston stroke.

5.6.5 Transducers

Transducers on the MTS machine sense some quantity generated by the hydraulic actuator, such as vertical load or linear displacement, and provide an output voltage directly proportional to the measured quantity.

The load-cell is a force-measuring transducer that provides an output voltage directly proportional to the applied load. Compressive and tensile forces are distinguished by the polarity of the output voltage.

The linear displacement of the loading ram is measured by a linear variable differential transformer (LVDT). The LVDT requires A-C excitation and provides an A-C output. The amplitude of the output varies in direct proportion to the amount of displacement of the LVDT core.
5.6.6 Transducer Conditioners

Transducer conditioners supply excitation voltages to their respective transducers and control the transducer output voltages to d-c levels suitable for use in the control portion of the system. Output of each transducer conditioner is 10 volts, positive or negative when the mechanical input to the transducer equals plus or minus 100% of the selected operating range. For example if the MTS machine was set on load control and 100% of the operating range, the output voltage would be ± 10 volts when a load of ±20,000 pounds was applied. Corresponding if the MTS was set on strain control and 100% of the operating range, the output voltage would be ±10 volts when a displacement of ±5 inches occurred. Fig. 5.3 shows a transducer conditioner and front panel of a typical controller chassis.

5.6.7 Triaxial Testing System

The triaxial cell was shown in Fig.5.17 for performing all cyclic triaxial tests. A pore pressure transducer attached to a four-way valve plugged into the right side of the triaxial cell base was used to measure the cell pressure, back pressure, and pore water pressure at the top of the specimen. The pore pressure transducer was attached to a four-way valve such that by changing the position of the valve the transducer could be subjected to either the cell pressure, back pressure, or pore water pressure at the top of the specimen. Fig.5.15 shows the transducer and four-way valve disconnected from the triaxial cell.

On the right side of Fig.5.17, four pressure lines are seen. The upper horizontal line entering the base of the triaxial cell applied the confining cell pressure, with the line passing behind the cell and over to the four-way valve used for transmitting the cell pressure to the pore pressure transducer. The middle line entering the base of the triaxial cell applied the back
pressure to the specimen with the lower line used for transmitting the pore pressure from the top of the specimen to the four-way valve and pressure transducer.

The pressure control panel shown in Fig. 5.18 could be used to apply cell and back pressures simultaneously to three triaxial cells. An air compressor connected to the pressure control panel furnished a maximum air pressure of 200 psi for the pressure control panel. When the pressure in the air tank dropped below a specified minimum valve a pressure switch would turn on the compressor.

The graduated burettes (vertical standpipes) shown in Fig. 5.18 were used to measure specimen volume changes within an accuracy of 0.1 cubic centimeter. Specimen volume change measurements were made after consolidation of the test specimen and after attaching the loading ram to the specimen top cap.

5.7 Operation of MTS Electro-Hydraulic Machine for Cyclic Triaxial Test

Although degree of saturation is usually satisfactory for testing upon the application of final back pressure increment, a sample is always allowed to sit overnight before it is tested. A 20-kip MTS electro hydraulic machine is used in performing all cyclic triaxial tests. Procedure for testing a sample is outlined as follows:

1. Check the pore pressure B parameter is higher than 0.95. All soil specimens were allowed to consolidate over night at the end of sample preparation, when the B value is found satisfactory.

2. Transfer the triaxial cell to the MTS machine, center and lock the triaxial cell base to the MTS loading platform using three C clamps.

3. Calculate the load required to balance the confining force acting on the triaxial cell
loading ram. This load is equal to the magnitude of the cell pressure times the cross sectional area of the loading ram.

(4) Use MTS machine to apply a slightly larger load than required to balance the confining pressure to bring down the loading ram and then connect the loading ram to the sample loading cap. Record any volume change upon the completion of the connection.

(5) Calculate the deviator load required to produce a deviator stress corresponding to a desired stress ratio. This load is equal to two times the product of initial consolidation pressure and stress ratio times the cross sectional area of the sample.

(6) Set the required deviator load level on MTS machine. An invert sine wave form with a frequency of 0.5 Hz was used in this test program.

(7) Set the excess pore pressure-time and deformation-time plotter for proper scales.

(8) Zero and set the proper sampling frequency and scales for data logger which records digitally the deviator load, the excess pore pressure, and the sample deformation.

(9) Close the value connecting to back pressure line and switch the transducer to measure pore pressure from the top of the sample.

(10) Open computer on the right side of MTS machine. See Figure 5.16.

(11) Open station manager on desk of computer, and then select “Basic Test Ware” mode.

(12) Test setup on dialog windows, select cyclic test, control mode is force and channel is “ch1”

(13) “Target setpoint” is 0.000kips, “Amplitude(±)” is deviator load, “Frequency” is 0.5 Hz, “Wave shape” is Sine, “Compensator” is None, “Start Action” is disabled, “Done Action” is disabled, Preset 50 cycles on tees counters.

(14) Save data file to my triaxial test file.
(15) Zero force and displacement on signal auto offset before run test.

(16) Test setup on computer shown in Figure 5.19.

(17) Start the test and observe the load-deformation, excess pore pressure-time, and deformation-time plotting.

(18) Terminate the test when a sample has liquefaction during cyclic loading, or when the excess pore pressure and the axial deformation are stabilized.

(19) Allow the excess pore pressure in the sample to dissipate by opening the back pressure valve, and then disconnect the sample loading cap from the loading ram.

(20) Remove the triaxial cell from the MTS machine.

(21) Remove the tested soil form the triaxial cell for recycling and clean the cell for subsequent use.
Figure 5.1 Rubber 0-Ring
Figure 5.2 Vernier caliper used to measure the thickness of the membrane and diameter of the sand specimen.
Figure 5.3 Vernier caliper used to measure the height of the test specimen
Figure 5.4 Looking down on the base of the triaxial cell
Figure 5.5 Rubber membrane placed on the base of the triaxial cell
Figure 5.6 Cylindrical mold placed on the base of the triaxial cell
Figure 5.7 the membrane adhere on the inner wall of the mold
Figure 5.8 Monterey No.0 sand placed within the zero raining device
Figure 5.9 Porous stone placed on top of the Monterey No. 0 Sand Specimen
Figure 5.10 Top cap placed on top of the specimen
Figure 5.11 Positioning of the top surface of the loading cap parallel to the base of the triaxial cell
Figure 5.12 Measuring the length of the sample using a vernier caliper
Figure 5.13 Pi tape
Figure 5.14 Specimen was saturating
Figure 5.15 Pore pressure transducer and a 4-way valve used to monitor cell pressure, back pressure, and specimen pore water pressure
Figure 5.16 Closed loop electro-hydraulic materials test system applying a sinusoidal loading to a Monterey No. 0 Sand Specimen
Figure 5.17 Test Specimen Placed Within the Triaxial Cell
Figure 5.18 Pressure control panel used to apply cell and back pressure to the triaxial cell. Graduated burettes were used to determine specimen volume change.
Figure 5.19 Set up on the computer to run MTS
6 Cyclic Triaxial Test Results

6.1 Introduction

The principle of cyclic triaxial test is shown in Figure 6.1. The soil specimen is usually encased in a rubber membrane to prevent the pressurized cell fluid (usually water) from penetrating the pores of the soils. Axial load is applied through a piston, and the induced pore water pressure during an undrained test is measured. There are the minor principal and major principal stresses on specimen show in Figure 6.3 in cyclic triaxial test. Principle stresses applied on the boundary of the specimen.

In cyclic triaxial tests, the sample is Monterey No.0/30 sand. Monterey No.0 /30 sand, provide by Lonestar company of Monterey, California, is light brown beach sand from Monterey, California. The grain size distribution of this sand is shown in Figure 6.2 and it can be seen that Monterey No.0 sand is an uniform clean sand, having a coefficient of uniformity, \( C_u \), equal to 1.6, a coefficient of curvature, \( C_v \), equal to 1.00, and a mean grain size, \( D_{50} \), equal to 0.45mm. The sand is classified as SP by the Unified Classification System. The shape of the grains vary from angular to subrounded, with a large amount of subangular grains as quartz, although some of the grains are composed of feldspar. A small percentage of the grains composed of mica can also be found. The maximum unit weight is 105.8 pcf, the minimum unit Weight is 91.7 pcf, and also its specific gravity is 2.65.

Fifteen cyclic triaxial tests were performed. Twelve samples were tested at 0.5Hz and additional three samples were prepared at 30% relative density and tested under30 psi effective confining pressure at 1Hz, 1.5Hz and 2.0Hz, respectively to check the frequency effect. For the 12 samples, six were prepared at 30% relative density and other six at 50% relative density, and
each of the six sample group, half were consolidated at 15 psi effective confining pressure and
other half at 30psi. Three different stress ratios of 0.15, 0.25 and 0.4 were used in cyclic triaxial
tests. Soil specimens were subjected to sinusoidal wave loading starting with tension phase. See
details in Table 6.1.

In presentation of test results, it shows all curves for test#12(0529). Other test results and
curves show in Appendix I.

Due to missing large membranes for Hollow Cylinder Test, no tests were performed.
There did not show any Hollow Cylinder Test results. However, Hollow Cylinder Test Device,
sample preparation procedure were put on the appendix II. All test results will be present when
large membranes get for How Cylinder Test.
Figure 6.1 Schematic Diagram of the Triaxail Apparatus
Figure 6.2 Grain Size Distribution of Monterey No.0/30 Sand
### Table 6.1 Soil Sample Data Sheet

<table>
<thead>
<tr>
<th>Test#</th>
<th>Stress Ratio</th>
<th>Deviator Stress (psi)</th>
<th>P' (psi)</th>
<th>$\sigma_3$ (psi)</th>
<th>u (psi)</th>
<th>Frequency (Hz)</th>
<th>Load (lb)</th>
<th>B (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dr=50%</td>
<td>Test#1</td>
<td>0.25</td>
<td>7.5</td>
<td>17.5</td>
<td>80</td>
<td>65</td>
<td>0.5</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>Test#2</td>
<td>0.4</td>
<td>12</td>
<td>19</td>
<td>100</td>
<td>85</td>
<td>0.5</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td>Test#3</td>
<td>0.15</td>
<td>4.5</td>
<td>16.5</td>
<td>90</td>
<td>75</td>
<td>0.5</td>
<td>14</td>
</tr>
<tr>
<td>Dr=50%</td>
<td>Test#4</td>
<td>0.15</td>
<td>9</td>
<td>33</td>
<td>100</td>
<td>70</td>
<td>0.5</td>
<td>30</td>
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<td>Test#5</td>
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<td>55</td>
<td>0.5</td>
<td>47</td>
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<td>Test#6</td>
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<td>38</td>
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<td>80</td>
<td>0.5</td>
<td>75</td>
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<tr>
<td>Dr=30%</td>
<td>Test#7</td>
<td>0.25</td>
<td>7.5</td>
<td>17.5</td>
<td>80</td>
<td>65</td>
<td>0.5</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>Test#8</td>
<td>0.4</td>
<td>12</td>
<td>19</td>
<td>95</td>
<td>80</td>
<td>0.5</td>
<td>37</td>
</tr>
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<td></td>
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<td>16.5</td>
<td>90</td>
<td>75</td>
<td>0.5</td>
<td>14</td>
</tr>
<tr>
<td>Dr=3.0%</td>
<td>Effective Stress=30psi</td>
<td>Dr=3.0%</td>
<td>Effective Stress=30psi</td>
<td>Dr=3.0%</td>
<td>Effective Stress=30psi</td>
<td>Dr=3.0%</td>
<td>Effective Stress=30psi</td>
<td></td>
</tr>
<tr>
<td>---------</td>
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</tr>
<tr>
<td>Test#10</td>
<td>0.15 9 33 100</td>
<td>Test#11</td>
<td>0.25 15 35 95</td>
<td>Test#12</td>
<td>0.4 24 38 95</td>
<td>Test#13</td>
<td>0.4 24 38 85</td>
<td>Test#14</td>
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<tr>
<td>Test#15</td>
<td>0.4 24 38</td>
<td>1.5 85</td>
<td>1.5 85</td>
<td>1.5 85</td>
<td>1.5 85</td>
<td>1.5 85</td>
<td>1.5 85</td>
<td>1.5 85</td>
</tr>
</tbody>
</table>

Table 6.1S Oil Sample Data Sheet (Con't)
Figure 6.3 Stress Conditions on the Triaxial Specimen

\[ \sigma_1 = \sigma_3 + \sigma_d \quad \sigma_1' = \sigma_1 - u \quad \sigma_3' = \sigma_3 - u \]

\[ q = \sigma_1 - \sigma_3 = \sigma_1' - \sigma_3' \]

\[ p = (\sigma_1 + 2\sigma_3)/3 \quad p' = (\sigma_1' + 2\sigma_3')/3 \]

where \( q \) = axial deviator stress (psi)
\( p' \) = mean effective stress (psi)

\( \sigma_1 \) = the major principal stress (psi)
\( \sigma_3 \) = the minor principal stress (psi)

\( \sigma_1' \) = the major mean stress (psi)

\( \sigma_d \) = single amplitude of cyclic axial stress (deviator stress)

\( \sigma_3' \) = the minor mean stress (psi)

Stress-controlled cyclic test were conducted using a frequency of 0.5 Hz sine wave. The sine wave function provides a compressive load which is followed by an extension load. Three different stress ratios of 0.15, 0.25 and 0.4 were used in cyclic triaxial tests.

These tests were conducted with stress ratio, S.R. The stress ratio is defined as the single cyclic axial stress amplitude divided by two times of the effective initial consolidation pressure the stress ratio is expressed as:

\[ S.R. = \frac{\sigma_d}{2\sigma_{3c}} \]

Where S.R. = stress ratio
\( \sigma_d = \) single amplitude of cyclic axial stress (deviator stress)  
\( \sigma_{3c} = \) initial consolidation pressure

The single amplitude of cyclic axial stress is defined as estimated cyclic axial load to be applied to the specimen divided by area of specimen.

\[ \sigma_d = \frac{F}{A} \]

where \( F = \) cyclic axial load to be applied to the specimen

\( A = \) area of specimen
6.2 Behavior of Saturated Monterey No.0 Sand under Cyclic Loads

6.2.1 Stress Behavior

6.2.1.1 Loading versus Number of Cyclic to Liquefaction

In the Test #12(0529), the height of sample is 4.00in, and its diameter is 2.00in. The mass of sample is 316.44g, and its relative density is 30.03%. The “B” parameter is 0.98, and also its confining pressure is 95psi, its back pressure is 65 psi, its S.R. is 0.4. See details on Table 6.4

In this test, a cyclic load of constant amplitude (75lb), was applied with a frequency of 0.5Hz to a sample of saturated sand [test#12(0528)]. Figure 6.4 shows loading versus number of cycles to liquefaction. In the first 15 cycles of Fig.6.4, the amplitude of cyclic loading was constant with increasing number of cycles because the sample was not noticeable deformation. It means soil still was strong. However, in 16th cycles, the amplitude of cyclic loading dropped to 65lb. The amplitude of cyclic loading begun to decreased with increasing number of cycles after 16th cycles. After 17th cycles, the amplitude of cyclic loading rapidly dropped with increasing number of cycles. That means the sample was liquefied and too soft in the last few cycles.
Figure 6.4 load versus Number of Cycles to liquefaction
6.2.1.2 Deviator Stress versus Effective Mean Stress

The relationship between deviator stress and effective mean stress is shown in Fig. 6.5. At the beginning of test, the mean effective stress (p’) started at 38 psi, and also deviator stress (24 psi) applied on the sample. During 15 cycles of stress application, the amplitude of cyclic deviator stress keep constant with decreasing the effective mean stress. The q-p’ graph reflects the gradual build up of pore pressure as the effective stress (p’) reduces until the sample approaches liquefaction condition at which time the sample starts failing and the amplitude of cyclic deviator stress begun to drop. When the soil sample had liquefied, pore-water pressure equaled to the externally applied confining pressure. In 16th cycle, the amplitude of cyclic deviator stress rapidly dropped 20 percent. It means pore-water pressure increased closed to the value of applied confining pressure. After 16th cycle, the amplitude of cyclic deviator stress continued to drop, at the same time, the effective mean stress kept decreased. It means that the sample turned softer and softer.
Figure 6.5 Deviator Stress versus Effective Mean Stress

Test#12(0529)
Dr=30% frequency=0.5hz S.R.=0.4 deviator stress =30psi

liquefaction
6.2.2 Stress-Strain Behavior

6.2.2.1 Deviator Stress versus Axial Strain

The deviator stress-strain graph is also shown on Figure 6.6. For the first 8 cycles the curves are very close together, but as the sample approaches failure the strains increase and the hysteresis loops open up quickly. In the first 8 cycles, the range of axial strain was -0.1 to +0.1. It means that the sample was no noticeable deformation. In last few cycles, the amplitude of deviator stress decreased with increasing the effective mean stress. It can be seen that the sample was softened and large flow deformation took place with increasing number of cycles. In 15th cycle, the loops begun flat shape, and also the amplitude of deviator stress rapidly dropped 20 percent. The sample developed large strains which, in the 16th cycle, exceeded 50 percent during the last two cycles. That means that the sample had liquefied. In last two cycles of Figure 6.6, the ranges of axial strain were -0.1 to +0.8. It means that the sample turned slim after liquefaction. Young’s modulus was calculated in the plot of deviator stress versus axial stress. The Young’s modulus decreased with increasing the number of cycles because the sample turned softer and softer. The maximum Young’s modulus of sample is 29131.11psi, but Young’s modulus dropped to 4899.43psi when soil was closed to liquefaction. The numbers of cycles versus Young’s modulus is shown in Table 6-2.
Figure 6.6 Deviator Stress versus Axial Strain

test#12(0529)
Dr=30% Frequency=0.5Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

$\sigma_d$ (psi) 

$\varepsilon_a$ (%)
Table 6-2 Numbers of Cycles versus Young’s modulus

<table>
<thead>
<tr>
<th>No.#</th>
<th>E(ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>29131.11</td>
</tr>
<tr>
<td>2</td>
<td>29001.28</td>
</tr>
<tr>
<td>3</td>
<td>27473.18</td>
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<td>7</td>
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<td>8</td>
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<td>14</td>
<td>8058.35</td>
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<td>15</td>
<td>7222.00</td>
</tr>
<tr>
<td>16</td>
<td>4899.43</td>
</tr>
</tbody>
</table>
6.2.3 Excess Pore Pressure Water Development

6.2.3.1 Excess Pore Pressure versus Number of Cyclic to Liquefaction

Figure 6.7 shows excess pore pressure versus number of cycles. During the first 15 cycles of stress application, the sample showed no noticeable deformation although the pore-water pressure built up gradually. However, during the 16th stress cycle, the pore pressure suddenly increased to a value equal to the externally applied confining pressure. In fact, the soil had liquefied and the effective confining pressure had been reduced to zero. Over a wide range of strains, the soil could be observed to be in a fluid condition.

Pore-water pressure continues to build up steadily as the number of stress cycles increase, until there is a sudden increase denoting the onset of initial liquefaction. The different values of pore-water pressure developed during increases and decreases in deviator stress reflect the influence of the applied stress conditions.
Figure 6.7 Excess Pore Pressure versus Number of Cycles to Liquefaction

test #12(0529)
Dr=30% Frequency=0.5Hz S.R.=0.4 Deviator stress =24psi Effective stress=30psi
6.2.3.2 Excess Pore Pressure versus Axial Strain

The excess pore pressure and axial strain relationship is shown in Fig.6.8. In the first 13 cycles of Figure6.8, the axial strain changed from -0.1 to +0.1 with increasing pore-water pressure. This means that the sample showed no noticeable deformation. However, in the 14th cycles, the positive of axial strain rapidly increased to +0.3, and also the negative of axial strain rapidly decreased to -0.2. That means the sample started to deform. After the 14th cycle, the positive of axial strain increased with increasing pore-water pressure, but the negative of axial strain decreased with increasing pore-water pressure. In the 16th cycle, the excess pore water pressure was equal to the effective confining pressure, and also the positive of axial strain increased to +0.75. That means the sample had liquefied and got more slim.
test#12(0529)
Dr=30% Frequency=0.5Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

Figure 6.8 Excess Pore Pressure versus Axial Strain
6.2.4 Liquefaction Resistance of Saturated sample in terms of Stress Ratio versus Number of Cyclic

The relationship between stress ratio and number of cyclic to liquefaction showed in Figure 6.9. There are twelve test results in Figure 6.9. Stress ratio dropped linearly with creasing number of cycles to liquefaction. The more density soil sample, the more number of cycles to liquefaction. The bigger effective stress, the more number of cycles to liquefaction.

Figure 6.9 S.R. versus Number of Cycles to liquefaction (log scale)
6.2.5 Young’s Modulus Develop

6.2.5.1 Young’s Modulus versus Number of Cyclic (log scale)

The Young’s modulus can be calculated by dividing the tensile stress by the tensile strain in the portion of the stress-strain curve.

\[ E = \frac{\text{tensile stress}}{\text{tensile strain}} = \frac{\sigma_d}{\varepsilon} = \frac{F/A}{(\Delta L/L_o)} = \frac{FL_o}{A\Delta L} \]

Where \( E \) = the Young’s modulus (modulus of elasticity)

\( F \) = cyclic axial load to be applied to the specimen

\( \sigma_d \) = single amplitude of cyclic axial stress (deviator stress)

\( \varepsilon \) = the axial strain of specimen

\( A \) = area of specimen

\( \Delta L \) = the height of specimen changes

\( L_o \) = the original height of the specimen

The Young’s modulus and number of cycles (log scale) is shown in Figure 6.10. The Young’s modulus of 1\textsuperscript{st} cycle is the highest values. The Young’s modulus decreased with increasing the number of cycle. In the 5\textsuperscript{th} cycle, the Young’s modulus rapidly dropped 20 percent of the highest values. When the soil was liquefaction, the Young’s modulus dropped to 4899.43psi.
Figure 6.10 Young’s modulus versus Number of Cyclic (log scale)
6.2.5.2 $E_n/E_1$ versus Axial Strain

The ratio of $E_n/E_1$ dropped with increasing the axial strain. The $E_n/E_1$ and axial strain is shown in Figure 6.11. There are more details about numbers of cycles versus Young’s modulus, axial strain and $E_n/E_1$ in Table 6-3. In Figure 6.11, there are more ratios of $E_n/E_1$ that dropped rapidly from 1-0.4 in axial strain (0%-0.3%) because the sample is not liquefaction and Young’s modulus changed too much. However, when axial strain reached 0.5%, the ratio of $E_n/E_1$ was not dropped too much because the soil sample reached liquefaction situation.

![Figure 6.11 En/E1 versus Axial Strain](image-url)

**Figure 6.11 En/E1 versus Axial Strain**
### Table 6-3 Numbers of Cycles versus Young’s modulus, axial strain and $E_n/E_1$

<table>
<thead>
<tr>
<th>No.#</th>
<th>$E$ (psi)</th>
<th>$\varepsilon_a$ (%)</th>
<th>$E_n/E_1$</th>
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<tbody>
<tr>
<td>1</td>
<td>29500</td>
<td>0.1094</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>29131.11</td>
<td>0.1081</td>
<td>0.9875</td>
</tr>
<tr>
<td>3</td>
<td>29001.28</td>
<td>0.112</td>
<td>0.98309</td>
</tr>
<tr>
<td>4</td>
<td>27473.18</td>
<td>0.1</td>
<td>0.93129</td>
</tr>
<tr>
<td>5</td>
<td>27190.9</td>
<td>0.13</td>
<td>0.92173</td>
</tr>
<tr>
<td>6</td>
<td>26222.86</td>
<td>0.14</td>
<td>0.88891</td>
</tr>
<tr>
<td>7</td>
<td>19615.38</td>
<td>0.16</td>
<td>0.66493</td>
</tr>
<tr>
<td>8</td>
<td>17586.21</td>
<td>0.10789</td>
<td>0.59614</td>
</tr>
<tr>
<td>9</td>
<td>16740.1</td>
<td>0.09157</td>
<td>0.56746</td>
</tr>
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<td>10</td>
<td>16709.37</td>
<td>0.1081</td>
<td>0.56642</td>
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<tr>
<td>11</td>
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<td>0.55571</td>
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<tr>
<td>12</td>
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<td>16</td>
<td>7222</td>
<td>0.68143</td>
<td>0.24481</td>
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</table>
6.2.6 Frequency versus Number of Cyclic

Frequency versus Number of cyclic is shown in Figure 6.12. The number of cyclic to liquefaction increased with frequency decreased because axial loading turned faster than lower frequency, and also the soil reached liquefaction situation faster than lower frequency.

![Figure 6.12 Frequency versus Number of cyclic](image_url)
Table 6.4 Traxial sample data sheet

<table>
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<th>Date: 05-29-2012</th>
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<td>Maximum unit Wt. (pcf)</td>
<td>105.8</td>
<td></td>
</tr>
<tr>
<td>Minimum unit Wt. (pcf)</td>
<td>91.7</td>
<td></td>
</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
<td></td>
</tr>
<tr>
<td>Required sample HT (in)</td>
<td>4.00</td>
<td></td>
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<tr>
<td>Required sample dia (in)</td>
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<tr>
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<td>Sample dia.(in)</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>Sample Cross, Area (in²)</td>
<td>3.14</td>
<td></td>
</tr>
<tr>
<td>Sample Vol. (in³)</td>
<td>12.56</td>
<td></td>
</tr>
<tr>
<td>Sample WT. (g)</td>
<td>316.44</td>
<td></td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>30.03</td>
<td></td>
</tr>
</tbody>
</table>

Remarks and Calculations

| B (%) = 98 |
| Frequency=0.5Hz |
| Confining pressure = 95psi |
| Back pressure = 65psi |
| Stress Ratio = 0.4 | Triaxial cell #1 |
6.3 Discussions of the Test Results

This chapter showed all test results and curves for one set of test. Other test data and results showed on Appendix I.

According to all test results and curves, they indicated that the lower the relative density of sample, the more easily soil liquefaction will occur, and also liquefaction resistance of a saturated sand was affected by the confining pressure acting on the sand - the lower the confining pressure the more easily liquefaction will develop.

Compare to other test results, the test#12 (0529) showed a little difference on curve of axial stress versus strain. In Figure 6.8, the axial strain ranged from 0.8 to -0.1. It means that the top of sample turned thinner than bottom when the sample reached liquefaction situation. However, other test results showed that the axial strain changed equally from positive to negative when the soil sample was liquefied. Because of bigger effective stress and deviator stress, the sample of test#12(0529) comported a little different curve on axial stress versus strain. At the same time, Young’s modulus did not drop linearly with number of cycle to liquefaction in test #12 (0529).

According to 15 test results, they showed deviator stresses were not close to zero when soil samples were liquefied in the curve deviator stress versus effective mean stress.
7 Summary, Conclusions and Recommendations for future studies

7.1 Summary

This paper talked field methods and laboratory methods for liquefaction potential evaluation. In situ-test, they included SPT, CPT, Piezometric Cone Penetrometer Test and other techniques. Laboratory tests included monotonic triaxial compression tests, cyclic triaxial tests cyclic hollow cylinder tests, cyclic simple shear test and shaking table tests, etc. This paper showed that empirical approaches, cyclic stress-based approach and cyclic strain-based approach for liquefaction resistance analysis. Detail specimen preparation procedures critical for the successes of cyclic triaxial tests are also presented. Factors affecting liquefaction resistance of soils including relative density, confining pressure, cyclic stress amplitude, number of cyclic stress cycles soil sample particles size, pre-staining, cyclic loading frequencies and stress ratio.

Fifteen isotropically consolidated undrained cyclic triaxial tests were performed under different relative densities, confining pressures, stress ratios and frequencies. Twelve samples were tested at 0.5Hz and additional three samples were prepared at 30% relative density and tested under 30 psi effective confining pressure at 1Hz, 1.5Hz and 2.0Hz, respectively to check the frequency effect. For the 12 samples, six were prepared at 30% relative density and other six at 50% relative density, and each of the six sample group, half were consolidated at 15 psi effective confining pressure and other half at 30psi. Three different stress ratios of 0.15, 0.25 and 0.4 were used in cyclic triaxial tests.

To perform fifteen cyclic triaxial tests, axial load, excess pore water pressure and axial displacement were continuously recorded. The recorded data were processed to attain stress-strain behavior, excess pore water pressure, stress path (or q-p’ plots) and Young’s modulus
degradation during the cyclic loading test. Liquefaction resistances were presented in terms of cyclic stress of stress ratio versus number of the loading cycles to liquefaction.

7.2 Conclusions

Many factors affect the liquefaction resistance of soils as discussed in the thesis. This lab cyclic triaxial test program only can investigate the effect of a few factors, like relative density, confining pressures, stress ratio, frequency. While the effect of these factors has been extensively studies by earlier investigators, it was the intent of this investigator to perform high quality cyclic triaxial tests and the results serve as the basis for the research extension into future doctoral level research on the effect of lab and field factors on the liquefaction resistance and prediction of the liquefaction resistance in an event of earthquake. The results of this study on the liquefaction of Monterey No.0/30 sand are summarized as follows:

1. At a higher stress ratio, lower density and lower confining pressure, a sample will reach liquefaction at a less number of cycles of cyclic loading.

2. Under the same specimen conditions, the cyclic loading at a lower frequency will requires more number of cycles to liquefy.

3. The cyclic triaxial test results show that the number of cycles required for liquefaction decreases nonlinearly with the increase in cyclic stress ratio.

4. During a cyclic triaxial test for liquefaction resistance, the loading-unloading stress versus strain curves can be plotted and Young modulus evaluated. It was found that Young’s modulus of elasticity decreases from its peak value in first cycle as the test progresses with increasing number of cycles.
5. Cyclic triaxial test results showed that the stress versus strain curve was the steepest in the beginning, but became flattened, i.e. the soil became softer. When the sample reached initial liquefaction, cyclic stress amplitude dropped by about 20 to 40 percent.

6. During a liquefaction test, the excess pore water pressure continued to increase as the number of stress cycles increased. The rate of increase in pore water pressure accelerated as the number of cycles increased, until the liquefaction was reached, and then the access pore water pressure ceased to increase.

7.3 Recommendations for Future Studies

In this study, fifteen isotropically consolidated undrained cyclic triaxial tests were performed to determine the effect of relative density, confining pressure, stress ratio and frequency on liquefaction resistance of Monterey No.0/30 sand and behaviors of soil specimen on the way to liquefaction. The findings serve as the basis for the future study on the effects of various laboratory factors and field factors on cyclic behaviors of saturated soils and in-situ liquefaction resistance prediction. The tasks include:

1. Different soils of varying gradations and index properties will be used in this future study to investigate their effects on the liquefaction resistance.

2. Comprehensive literature study on the laboratory and field liquefaction resistances of soils and their prediction under different earthquake intensity.

3. Cyclic hollow cylinder test will be performed to relate the triaxial liquefaction resistance to simple shear liquefaction resistance using Monterey No.0/30 sand.
4. Besides the liquefaction resistance, post liquefaction resistance of soils will also be studied to provide the residual strength needed for the post-earthquake stability of slopes.

5. Variety of mixture and natural soils will be tested to determine the effect of void ratio of sand skeleton, fines contents, liquid limit and plastic limit on liquefaction resistance of soil using cyclic triaxial tests, hollow cylinder tests and other laboratory tests.

6. A field liquefaction resistance model(s) will be formulated to prediction liquefaction based the findings from laboratory investigations, field observations and earthquake magnitudes.

7. Comparing predicted liquefaction to the field observations, the prediction model(s) will be fine-tuned practical field application.

The above constitutes the tentative outlines for my doctoral degree research.
BIBLIOGRAPHY


Geotechnical Society, 233-238 (in English), 2001.


APPENDIX I

Cyclic Triaxial Test Results

Traixial test data sheet

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>#1</th>
<th>Date: 05-15-2012</th>
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</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.65</td>
<td></td>
</tr>
<tr>
<td>Maximum unit Wt. (pcf)</td>
<td>105.8</td>
<td></td>
</tr>
<tr>
<td>Minimum unit Wt. (pcf)</td>
<td>91.7</td>
<td></td>
</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
<td></td>
</tr>
<tr>
<td>Required sample HT (in)</td>
<td>4.00</td>
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</tr>
<tr>
<td>Required sample dia (in)</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>Sample HT (in)</td>
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<td></td>
</tr>
<tr>
<td>Sample dia.(in)</td>
<td>4.001</td>
<td></td>
</tr>
<tr>
<td>Sample Cross, Area (in²)</td>
<td>3.143</td>
<td></td>
</tr>
<tr>
<td>Sample Vol. (in³)</td>
<td>12.576</td>
<td></td>
</tr>
<tr>
<td>Sample WT. (g)</td>
<td>325.74</td>
<td></td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>49.47</td>
<td></td>
</tr>
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</table>

Remarks and Calculations

B (%) = 96
Confining pressure = 80 psi
Back pressure = 65 psi
Stress Ratio = 0.25

Triaxial cell #4
# Traixial test data sheet

Sample No. | #2 | Date: 05-15-2012
--- | --- | ---
Specific gravity | 2.65 | 
Maximum unit Wt. (pcf) | 105.8 | 
Minimum unit Wt. (pcf) | 91.7 | 
Thickness of membrane (in) | 0.02525 | 
Required sample HT (in) | 4.00 | 
Required sample dia (in) | 2.00 | 
Sample HT (in) | 2.00 | 
Sample dia.(in) | 4.00 | 
Sample Cross. Area (in$^2$) | 3.14 | 
Sample Vol. (in$^3$) | 12.56 | 
Sample WT. (g) | 325.74 | 
Relative Density (%) | 50.36 | 

**Remarks and Calculations**

B (%) = 96
Confining pressure = 100 psi
Back pressure = 85 psi
Stress Ratio = 0.4

Triaxial cell #2
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<td>91.7</td>
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<tr>
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<td>Required sample dia (in)</td>
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<tr>
<td>Sample WT. (g)</td>
<td>325.74</td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>50.36</td>
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</table>

**Remarks and Calculations**

B (%) = 96
Confining pressure = 90 psi
Back pressure = 75 psi
Stress Ratio = 0.15

Triaxial cell #1
### Traixial test data sheet

**Sample No.** #4  
**Date:** 05-21-2012

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<tr>
<td>Minimum unit Wt. (pcf)</td>
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</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
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<tr>
<td>Required sample HT (in)</td>
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<tr>
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<tr>
<td>Sample Cross, Area (in²)</td>
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<tr>
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<td>Sample WT. (g)</td>
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<tr>
<td>Relative Density (%)</td>
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**Remarks and Calculations**

- \( B (\%) = 98 \)
- Confining pressure = 100 psi
- Back pressure = 70 psi
- Stress Ratio = 0.15

Triaxial cell #4
# Traxial test data sheet

**Sample No.** #5  
**Date:** 05-21-2012

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<td><strong>Minimum unit Wt. (pcf)</strong></td>
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</tr>
<tr>
<td><strong>Thickness of membrane (in)</strong></td>
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**Remarks and Calculations**

- B (%) = 96
- Confining pressure = 85 psi
- Back pressure = 55 psi
- Stress Ratio = 0.25
- Triaxial cell #2
### Traxial test data sheet

**Sample No.** #6  
**Date:** 05-21-2012

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<tr>
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<td>Relative Density (%)</td>
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**Remarks and Calculations**

B (%) = 96  
Confining pressure = 110 psi  
Back pressure = 80 psi  
Stress Ratio = 0.4  

Triaxial cell #1
### Traixial Test Data Sheet

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<tr>
<td>Thickness of membrane (in)</td>
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#### Remarks and Calculations

- B (%) = 96
- Confining pressure = 80 psi
- Back pressure = 65 psi
- Stress Ratio = 0.25

Triaxial cell #4
## Traxial test data sheet

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<td>Thickness of membrane (in)</td>
<td>0.02525</td>
</tr>
<tr>
<td>Required sample HT (in)</td>
<td>4.00</td>
</tr>
<tr>
<td>Required sample dia (in)</td>
<td>2.00</td>
</tr>
<tr>
<td>Sample HT (in)</td>
<td>4.00</td>
</tr>
<tr>
<td>Sample dia.(in)</td>
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</tr>
<tr>
<td>Sample Cross. Area (in$^2$)</td>
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</tr>
<tr>
<td>Sample Vol. (in$^3$)</td>
<td>12.56</td>
</tr>
<tr>
<td>Sample WT. (g)</td>
<td>316.44</td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>30.03</td>
</tr>
</tbody>
</table>

### Remarks and Calculations

- **B (%) = 96**
- Confining pressure = 95 psi
- Back pressure = 80 psi
- Stress Ratio = 0.4
- Triaxial cell #2
### Traxial test data sheet

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>#9</th>
<th>Date: 05-24-2012</th>
</tr>
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<tbody>
<tr>
<td>Specific gravity</td>
<td>2.65</td>
<td></td>
</tr>
<tr>
<td>Maximum unit Wt. (pcf)</td>
<td>105.8</td>
<td></td>
</tr>
<tr>
<td>Minimum unit Wt. (pcf)</td>
<td>91.7</td>
<td></td>
</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
<td></td>
</tr>
<tr>
<td>Required sample HT (in)</td>
<td>4.00</td>
<td></td>
</tr>
<tr>
<td>Required sample dia (in)</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>Sample HT (in)</td>
<td>4.00</td>
<td></td>
</tr>
<tr>
<td>Sample dia.(in)</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>Sample Cross. Area (in²)</td>
<td>3.14</td>
<td></td>
</tr>
<tr>
<td>Sample Vol. (in³)</td>
<td>12.56</td>
<td></td>
</tr>
<tr>
<td>Sample WT. (g)</td>
<td>316.44</td>
<td></td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>30.03</td>
<td></td>
</tr>
</tbody>
</table>

#### Remarks and Calculations

- B (%) = 98
- Confining pressure = 90 psi
- Back pressure = 75 psi
- Stress Ratio = 0.15

Triaxial cell #1
## Traxial test data sheet

**Sample No.** #10  
**Date:** 05-29-2012

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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<tbody>
<tr>
<td>Specific gravity</td>
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<tr>
<td>Maximum unit Wt. (pcf)</td>
<td>105.8</td>
</tr>
<tr>
<td>Minimum unit Wt. (pcf)</td>
<td>91.7</td>
</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
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<tr>
<td>Required sample HT (in)</td>
<td>4.00</td>
</tr>
<tr>
<td>Required sample dia (in)</td>
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<tr>
<td>Sample dia.(in)</td>
<td>2.00</td>
</tr>
<tr>
<td>Sample Cross, Area (in²)</td>
<td>3.14</td>
</tr>
<tr>
<td>Sample Vol. (in³)</td>
<td>12.59</td>
</tr>
<tr>
<td>Sample WT. (g)</td>
<td>316.44</td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>28.73</td>
</tr>
</tbody>
</table>

### Remarks and Calculations

- B (%) = 98
- Confining pressure = 100psi
- Back pressure = 70psi
- Stress Ratio = 0.15

Triaxial cell #4
**Traxial test data sheet**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>#11</th>
<th>Date: 05-29-2012</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
<tr>
<td>Maximum unit Wt. (pcf)</td>
<td>105.8</td>
<td></td>
</tr>
<tr>
<td>Minimum unit Wt. (pcf)</td>
<td>91.7</td>
<td></td>
</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
<td></td>
</tr>
<tr>
<td>Required sample HT (in)</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Required sample dia (in)</td>
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<td></td>
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<td>Sample HT (in)</td>
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<tr>
<td>Sample dia.(in)</td>
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<tr>
<td>Sample Cross, Area (in²)</td>
<td>3.14</td>
<td></td>
</tr>
<tr>
<td>Sample Vol. (in³)</td>
<td>12.57</td>
<td></td>
</tr>
<tr>
<td>Sample WT. (g)</td>
<td>316.44</td>
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</tr>
<tr>
<td>Relative Density (%)</td>
<td>29.81</td>
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</table>

**Remarks and Calculations**

B (%) = 96  
Confining pressure = 95psi  
Back pressure = 65psi  
Stress Ratio = 0.25  
Triaxial cell #2
## Traxial sample data sheet

Sample No. | #12 | Date: 05-29-2012
---|---|---
Specific gravity | 2.65 |  
Maximum unit Wt. (pcf) | 105.8 |  
Minimum unit Wt. (pcf) | 91.7 |  
Thickness of membrane (in) | 0.02525 |  
Required sample HT (in) | 4.00 |  
Required sample dia (in) | 2.00 |  
Sample HT (in) | 4.00 |  
Sample dia.(in) | 2.00 |  
Sample Cross. Area (in²) | 3.14 |  
Sample Vol. (in³) | 12.56 |  
Sample WT. (g) | 316.44 |  
Relative Density (%) | 30.03 |  

### Remarks and Calculations

<table>
<thead>
<tr>
<th>B (%)</th>
<th>98</th>
<th>Triaxial cell #1</th>
</tr>
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<tbody>
<tr>
<td>Frequency</td>
<td>0.5Hz</td>
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</tr>
<tr>
<td>Confining pressure</td>
<td>95psi</td>
<td></td>
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<tr>
<td>Back pressure</td>
<td>65psi</td>
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<tr>
<td>Stress Ratio</td>
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### Traixial test data sheet

<table>
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<tr>
<th>Sample No.</th>
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<th>Date: 08-04-2012</th>
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</table>

<table>
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<th>Value</th>
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<tr>
<td>Maximum unit Wt. (pcf)</td>
<td>105.8</td>
</tr>
<tr>
<td>Minimum unit Wt. (pcf)</td>
<td>91.7</td>
</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
</tr>
<tr>
<td>Required sample HT (in)</td>
<td>4.00</td>
</tr>
<tr>
<td>Required sample dia (in)</td>
<td>2.00</td>
</tr>
<tr>
<td>Sample HT (in)</td>
<td>4.00</td>
</tr>
<tr>
<td>Sample dia.(in)</td>
<td>2.00</td>
</tr>
<tr>
<td>Sample Cross, Area (in²)</td>
<td>3.14</td>
</tr>
<tr>
<td>Sample Vol. (in³)</td>
<td>12.56</td>
</tr>
<tr>
<td>Sample WT. (g)</td>
<td>316.44</td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>30.03</td>
</tr>
</tbody>
</table>

**Remarks and Calculations**

- B (%) = 96
- Confining pressure = 110psi
- Back pressure = 80psi
- Stress Ratio = 0.4
- Frequency=1.0Hz

Triaxial cell #1
## Traxial test data sheet

**Sample No.** #14  **Date:** 08-04-2012

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
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<tbody>
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<tr>
<td>Maximum unit Wt. (pcf)</td>
<td>105.8</td>
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<tr>
<td>Minimum unit Wt. (pcf)</td>
<td>91.7</td>
</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
</tr>
<tr>
<td>Required sample HT (in)</td>
<td>4.00</td>
</tr>
<tr>
<td>Required sample dia (in)</td>
<td>2.00</td>
</tr>
<tr>
<td>Sample HT (in)</td>
<td>4.00</td>
</tr>
<tr>
<td>Sample dia.(in)</td>
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</tr>
<tr>
<td>Sample Cross, Area (in^2)</td>
<td>3.14</td>
</tr>
<tr>
<td>Sample Vol. (in^3)</td>
<td>12.56</td>
</tr>
<tr>
<td>Sample WT. (g)</td>
<td>316.44</td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>30.03</td>
</tr>
</tbody>
</table>

### Remarks and Calculations

- B (%) = 98
- Confining pressure = 115psi
- Back pressure = 85psi
- Stress Ratio = 0.4
- Frequency = 2.0Hz

Triaxial cell #4
## Traxial test data sheet

<table>
<thead>
<tr>
<th>Sample No.</th>
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<th>Date: 08-13-2012</th>
</tr>
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<tbody>
<tr>
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<tr>
<td>Maximum unit Wt. (pcf)</td>
<td>105.8</td>
<td></td>
</tr>
<tr>
<td>Minimum unit Wt. (pcf)</td>
<td>91.7</td>
<td></td>
</tr>
<tr>
<td>Thickness of membrane (in)</td>
<td>0.02525</td>
<td></td>
</tr>
<tr>
<td>Required sample HT (in)</td>
<td>4.00</td>
<td></td>
</tr>
<tr>
<td>Required sample dia (in)</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>Sample HT (in)</td>
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<td></td>
</tr>
<tr>
<td>Sample dia.(in)</td>
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<td></td>
</tr>
<tr>
<td>Sample Cross, Area (in²)</td>
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<tr>
<td>Sample Vol. (in³)</td>
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</tr>
<tr>
<td>Sample WT. (g)</td>
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<tr>
<td>Relative Density (%)</td>
<td>27.76</td>
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</tr>
</tbody>
</table>

### Remarks and Calculations

- B (%) = 96
- Confining pressure = 115psi
- Back pressure = 85psi
- Stress Ratio = 0.4
- Frequency=1.5Hz

Triaxial cell #1
Monterey No. 0 Sand Test #1 (0515)
Dr = 50% Frequency = 0.5Hz S.R. = 0.25 Deviator stress = 7.5 psi Effective stress=15psi

Liquefaction

Load (lb)

Number of cycles to liquefaction
Test #1 (0515)
Dr = 50%  Frequency = 0.5 Hz  S.R. = 0.25  Deviator stress = 7.5 psi  Effective stress = 15 psi

Liquefaction
D'50% Frequency=0.5 Hz S.R.=0.25 Deviator stress=7.5 psi Effective stress=15 psi
test #1(0515)
Dr=50% Frequency=0.5hz S.R.=0.25 Deviator stress=7.5psi Effective stress=15psi

liquefaction
Dr = 50% Frequency= 0.5Hz S.R. = 0.4 Deviator stress = 12 psi Effective stress = 15 psi

Test #2 (0515)

number of cycles to liquefaction

liquefaction

(0)
test #2(0515)
Dr=50% Frequency=0.5Hz S.R.=0.4 Deviator stress=12psi Effective stress=15psi

The graph shows the change in pore pressure (psia) over the number of cycles. It indicates liquefaction at certain points.
test #2(0515)
Dr=50% Frequency=0.5Hz S.R.=0.4 Deviator stress=12psi Effective stress=15psi

liquefaction
test#2(0515)
Dr=50% Frequency=0.5 Hz S.R.=0.4 Deviator stress=12 psi Effective stress=15 psi

liquefaction
test #2(0515)
Dr=50% frequency=0.5Hz S.R.=0.4 Deviator stress = 12psi Effective stress=15psi

liquefaction
test#2(0515)

Dr=50% Frequency=0.5Hz S.R.=0.4 Deviator stress=12psi Effective stress=15psi
Dr = 50% Frequency = 0.5 Hz S.R. = 0.15 Deviator stress = 4.5 psi Effective stress = 15 psi

Test #3 (0515) liquefaction

Load (q) vs. number of cycles to liquefaction
Test #3 (0515)
Dr = 50% Frequency = 0.5 Hz S.R. = 0.15 Deviator stress = 4.5 psi Effective stress = 15 psi

Liquefaction
test #3(0515)
Dr= 50% Frequency = 0.5Hz S.R.=0.15 Deviator stress=4.5psi Effective stress=15psi

liquefaction
test #3(0515)

Df=50% Frequency=0.5Hz S.R.=0.15 Deviator stress=4.5psi Effective stress=15psi

liquefaction

(isd)b

p[psi]
test#4(0521)
Dr=50% Frequency=0.5Hz S.R.=0.15 Deviator stress=9 psi Effective stress=30psi

Load(lb)
0 200 400 600 800 1000 1200

Number of cycles

Liquefaction
test#4(0521)
Dr=50% Frequency=0.5Hz S.R.=0.15 Deviator stress=9psi Effective stress=30psi
Test #5 (0521)

Dv = 50% frequency = 0.5 Hz Deviator stress = 15 psi Effective stress = 30 psi

Liquefaction

Number of cycles to liquefaction

Pore pressure (psi)
test #5(0521)
Dr = 50% Frequency = 0.5Hz S.R. = 0.25 Deviator stress = 15psi Effective stress=30psi

liquefaction
test#5(0521)
Dr=50% Frequency=0.5Hz S.R.=0.25 Deviator stress=15psi Effective stress=30psi

liquefaction
test#5(0521)
Dr=50% Frequency=0.5Hz S.R.=0.25 Deviator stress=15psi Effective stress=30psi

liquefaction
Test #6 (0521)
Dr = 50% S.R. = 0.4 Frequency = 0.5 Hz Deviator stress = 24 psi Effective stress = 30 psi
test#6(0521)
Dr=50% Frequency=0.5Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

[Graph showing pore pressure vs. number of cycles, indicating liquefaction]
test #6(0521)
Dr=50% frequency=0.5Hz S.R.=0.4 deviator stress=24psi Effective stress=30psi

liquefaction
test#6(0521)
Dr=50% Frequency=0.5Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi
Test #70524

$D_r = 30\%$, Frequency = 0.5 Hz, S.R. = 0.2, Deviator Stress = 7.5 psi, Effective stress = 15 psi
Test #7(0524)
Dr=30% Frequency = 0.5 Hz S.R.=0.25 Deviator Stress = 7.5psi Effective stress=15psi

Liquefaction
test #7(0524)
Dr=30% Frequency=0.5Hz S.R.=0.25 Deviator stress=7.5psi Effective stress=15psi

liquefaction
Test #8(0524)
Dr = 30%  Frequency = 0.5 Hz  S.R. =0.4  Deviator stress = 12 psi  Effective stress = 15psi

Liquefaction

Number of cycles to liquefaction

Load (lb)

0 2 4 6 8 10 12 14
Dr = 30% Frequency = 0.5 Hz S.R. = 0.4 Deviator Stress = 12 psi Effective stress = 15 psi

Test #80524

Pore pressure (psi)

Number of cycles to liquefaction

liquefaction
test #8(0524)
Dr=30% Frequency=0.5Hz S.R.=0.4 Deviator stress=12psi Effective stress=15psi

liquefaction
test #8(0524)
Dr=30% frequency=0.5Hz S.R.=0.4 Deviator stress=12psi Effective stress=15psi

liquefaction
test#8(0524)
Dr=30% Frequency=0.5Hz S.R.=0.4 Deviator stress=12psi Effective stress=15psi
Test #9(0524)
Dr = 30% Frequency = 0.5 Hz S.R. = 0.15 Deviator stress = 4.5 psi Effective stress = 15 psi

Liquefaction
Test #9(0524)
Dr = 30% Frequency = 0.5Hz S.R. = 0.15 Deviator stress = 4.5psi Effective stress = 15psi

Pore pressure (psi)

Number of cycles to liquefaction
Dr=30% Frequency=0.5 Hz S.R.=0.15 Deviator stress=4.5 psi Effective stress = 15 psi

liquefaction

\[ \eta (\sigma_d, \rho_0) \]
test#9 (0524)

Dr=30% Frequency=0.5Hz S.R.=0.15 Deviator stress=4.5 psi Effective stress = 15 psi

liquefaction
test#9(0524)
Dr=30% Frequency=0.5Hz S.R.=0.15 Deviator stress=4.5psi Effective stress=15psi

liquefaction
Test#10(0529)
Dr =30% Frequency = 0.5 Hz S.R. = 0.15 Deviator stress = 9psi Effective stress =30psi

Liquefaction

Load (lb)
0 10 20 30 40 50 60 70 80 90
-40 -30 -20 -10 0

Number of cycles to liquefaction
test #10(0529)
Dr = 30\% \quad \text{Frequency} = 0.5 \text{Hz} \quad \text{S.R.} = 0.15 \quad \text{Deviator stress} = 9 \text{psi} \quad \text{Effective stress} = 30 \text{psi}

Number of cycles to liquefaction

\Delta u (\text{psi})

Liquefaction
test#10(0529)
Dr=30% Frequency=0.5Hz S.R.=0.5Hz Deviator stress=9psi Effective stress=30psi

liquefaction
test #10(0529)
Dr=30% Frequency= 0.5Hz S.R.=0.15 deviator stress=9psi effective stress =30psi
test#10(0529)
Dr=30% Frequency=0.5Hz S.R.=0.15 Deviator stress=9psi Effective stress=30psi

Liquefaction

\( \Delta u(\text{psi}) \)

\( \varepsilon a \)
Test #11 (0529)

Dr = 30% Frequency = 0.5 Hz S.R. = 0.25 Deviator stress = 15 psi Effective stress = 30 psi

liquefaction

Number of cycles to liquefaction

(qp) (lb)

60 40 20 0 20 40 60

-60 -40 -20
Test#11(0529)
Dr=30% Frequency=0.5Hz S.R.=0.25 Deviator stress=15psi Effective stress=30psi
test #11(0529)
Dr= 30% Frequency = 0.5Hz S.R.=0.25 Deviator stress=15psi Effective stress=30psi
Dr=30% Frequency=0.5 Hz S.R.=0.25 Deviator stress=15 psi Effective stress=30 psi

Test #11 (0529)

liquefaction

\( (\sigma_d - \sigma_0) \phi_{op} \)
D=30% Frequency=0.5Hz S.R.=0.25 Deviator stress=15psi Effective stress=30psi
Test #12(0529)
Dr = 30%  Frenquency = 0.5Hz  S.R. = 0.4  Deviator stress = 24psi  Effective stress = 30psi
test#12(0529)
Dr=30% Frequency=0.5Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi
test#12(0529)
Dr=30% Frequency=0.5Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

liquefaction
D=30% Frequency=2.0Hz R=0.4 Deviator stress=24psi Effective stress=30psi

Test#13(0804)
test#13(0804)
Dr=30% Frequency=2.0Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

liquefaction
test#13(0804)
Dr=30% Frequency=2.0Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

liquefaction
test#13(084)
Dr=30% Frequency=2.0Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

liquefaction
test#14(0804)
Dr=30% Frequency=1.0Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi
test#14(0804)
Dr=30% Frequency=1.0HZ S.R.=0.4 Deviator stress=24psi Effective stress=30psi
test#14(0804)
Dr=30% Frequency=1.0Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

liquefaction
$D_t=30\%$, Frequency=1.0 Hz, S.R.=0.4 Deviator stress=24 psi, Effective stress=30 psi

liquefaction

(psd)øN
test#14(0804)
Dr=30% Frequency=1.0Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi
test #15(0813)
Dr=30% Frequency=1.5Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi
test#15(0813)
Dr=30% F=1.5Hz S.R.=0.4 Deviator stress=24psi Effective stress=30psi

liquefaction
Liquefaction

DI=30% F=1.5 Hz S.R. = 0.4 Deviator stress=24psi Effective stress=30psi

(\sigma_d) n\nu

0 0.1 0.2 0.3 0.4 0.5
-0.1 -0.2 -0.3 -0.4 -0.5

35 30 25 20 15 10 5
APPENDIX II

Hollow Cylinder Test Device, Sample Preparation Procedure and Test Results

Background

A hollow cylindrical test device (HCTD) is an extremely valuable tool for studying constitutive behaviour under generalized stress conditions. The HCTD allows independent control of the magnitudes of the three principal stresses and rotation of the major-minor principal stress axes while recording the specimen deformational and pore pressure responses.

In a hollow cylinder test, a hollow cylindrical soil specimen is enclosed in between an inner membrane and an outer membrane. The confining pressure can be independently applied on both inner and outer chambers; therefore, inner and outer pressures can be controlled either equally or unequally. The axial load and torque are applied on the top of specimen and transmitted by a top cap or a pedestal to the specimen.

When each of these boundary stresses can be controlled independently, both the principal stress direction and the relative magnitude of the intermediate principal stress can be controlled, thus the hollow cylindrical test (HCT) can facilitate more generalized stress path testing than the conventional test apparatus. It is also possible to control (or measure) the pore water pressure and apply back pressure, so that drainage conditions can be controlled and both drained and undrained tests can be performed. As a result, the HCA offers an opportunity of extending the stress path approach to include simulation of both principal stress rotation and variation in intermediate principal stress, as well as conducting fundamental research into the effect of principal stress rotation under a reasonably generalized stress state.

Principles of hollow cylinder testing
Figure 2.1 illustrates idealized stress conditions in a hollow cylindrical element subjected to axial load, \( W \), torque, \( M_T \), internal pressure, \( P_i \), and external pressure, \( P_o \).

During shearing, the torque, \( M_T \), develops shear stresses, \( \tau_{\theta z} \) and \( \tau_{z\theta} \) (\( \tau_{\theta z} = \tau_{z\theta} \)) in vertical and horizontal planes, the axial load, \( W \), contributes to a vertical stress, \( \sigma_z \). \( P_i \) and \( P_o \) determine \( \sigma_r, \sigma_\theta \). The relationship between \( \sigma_r \) and \( \sigma_\theta \), is established by the differences between \( P_i \) and \( P_o \).

\[
\sigma_\theta = \sigma_r + r \left( \frac{d\sigma_r}{dr} \right) \tag{2-1}
\]

where \( r \) is the radial distance to a point in the hollow cylinder, and \( d\sigma_r \) and \( d\sigma_\theta \) are the radial and circumferential stress increments respectively. When \( P_i = P_o \), \( \sigma_r \) becomes identical to \( \sigma_\theta \).

The state of stress in a hollow cylinder test is defined with reference to cylindrical coordinates, in terms of the stress components shown in Figure 2.1.

\[
\begin{bmatrix}
\sigma_r & 0 & 0 \\ 0 & \sigma_\theta & \tau_{\theta z} \\ 0 & \tau_{z\theta} & \sigma_z
\end{bmatrix}
\]

\( (2-2) \)

Since the stresses will not be uniform across the wall of the cylinder for various loading conditions, to consider the hollow cylinder as an element, it becomes necessary to calculate average stresses, \( \bar{\sigma}_z, \bar{\sigma}_r, \bar{\sigma}_\theta, \bar{\tau}_{\theta z}. \) Hight et al. (1983) used the following expressions:

Average vertical stress \( \bar{\sigma}_z = [W/\pi (r_o^2 - r_i^2)] + [(P_o r_o^2 - P_i r_i^2) / (r_o^2 - r_i^2)] \tag{2-3} \)

Average radial stress \( \bar{\sigma}_r = (P_o r_o + P_i r_i) / (r_o + r_i) \tag{2-4} \)

Average circumferential stress \( \bar{\sigma}_\theta = (P_o r_o - P_i r_i) / (r_o - r_i) \tag{2-5} \)
Average shear stress \( \tau_{0z} = \frac{3M_T}{2\pi (R_0^3 - R_i^3)} \) \hspace{1cm} (2-6)

In hollow cylinder tests, the radial stress, \( \sigma_r \), is usually equal to the intermediate principal stress (\( \sigma_2 \)). The major and minor principal stresses, \( \sigma_1 \) and \( \sigma_3 \), are observed from the average stress components \( \sigma_r \), \( \sigma_{\theta} \), and \( \tau_{0z} \), and as following:

\[
\sigma_2 = \left( \frac{\sigma_r + \sigma_{\theta}}{2} \right) + \alpha \left( \left( \frac{\sigma_r - \sigma_{\theta}}{2} \right)^2 + \left( \frac{\tau_{0z}}{2} \right)^2 \right) \hspace{1cm} (2-7)
\]

\[
\sigma_1 = \sigma_r \hspace{1cm} (2-8)
\]

\[
\sigma_3 = \left( \frac{\sigma_r + \sigma_{\theta}}{2} \right) - \alpha \left( \left( \frac{\sigma_r - \sigma_{\theta}}{2} \right)^2 + \left( \frac{\tau_{0z}}{2} \right)^2 \right) \hspace{1cm} (2-9)
\]

By regarding the specimen as a single element, the state of strain is presented in cylindrical coordinates in terms of the following components:

\[
[\varepsilon] = \begin{bmatrix}
\varepsilon_r & 0 & 0 \\
0 & \varepsilon_{\theta} & \frac{\gamma_{z\theta}}{2} \\
0 & \frac{\gamma_{z\theta}}{2} & \varepsilon_z
\end{bmatrix}
\]

\hspace{1cm} (2-10)

Also, it is necessary to calculate the average strains. According to the paper of Hight et al. (1983), the average strains are calculated using the following equations:

Average axial strain \( \varepsilon_z = \frac{w}{H} \) \hspace{1cm} (2-11)

Average radial strain \( \varepsilon_r = - \frac{(u_o-u_i)}{(R_o-R_i)} \) \hspace{1cm} (2-12)

Average circumferential strain \( \varepsilon_{\theta} = - \frac{(u_o+u_i)}{(R_o+R_i)} \) \hspace{1cm} (2-13)

Average shear strain \( \gamma_{z\theta} = \frac{20 (R_o^3 - R_i^3)}{[3H(R_o^3 - R_i^3)]} \) \hspace{1cm} (2-14)

Where the definitions of average stresses and strains are shown in Figure 2.2.
Since the average values of $\varepsilon_z$ and $\gamma_{th}$ are based on strain compatibility only, the expressions for the average strains are valid and independent of the constitutive law of the material. The average values of $\varepsilon_r$ and $\varepsilon_\theta$ are based on a linear variation of radial displacement across the wall of the specimen. In the hollow cylinder test, the radial strain ($\varepsilon_r$) is usually the intermediate principal strain, $\varepsilon_2$. The major and minor principal strains can be observed from the average strain components:

\[
\varepsilon_1 = \left( (\varepsilon_z + \varepsilon_\theta)^{'} / 2 \right) + \alpha \left( (\varepsilon_z^{'} - \varepsilon_\theta^{'} ) / 2 \right)^2 + \left( \gamma_{th}^{'} / 2 \right)^2
\]
\[\text{(2-15)}\]

\[
\varepsilon_2 = \varepsilon_r^{'}
\]
\[\text{(2-16)}\]

\[
\varepsilon_3 = \left( (\varepsilon_z^{'} + \varepsilon_\theta^{'} ) / 2 \right) - \alpha \left( (\varepsilon_z^{'} - \varepsilon_\theta^{'} ) / 2 \right)^2 + \left( \gamma_{th}^{'} / 2 \right)^2
\]
\[\text{(2-17)}\]

Parameters $\alpha$ and $b$ are two variables of stress path to describe fundamentally different aspects in the applied state of state of stress. $\alpha$ (as shown in Figure 2.1(d)), is the inclination of major principal stress direction with respect to the vertical axis, which can be varied from 0 to 90°. It can be computed from the known average stress components

\[\tan 2 \alpha = 2 \tau_{0z}^{'} / (\sigma_{1}^{'} - \sigma_{0}^{'} )\]
\[\text{(2-18)}\]

$b$ is defined as the relative magnitude of the intermediate principal stress, which can be varied from 0 to 1:

\[b = (\sigma_{2}^{'} - \sigma_{3}^{'} ) / (\sigma_{1}^{'} - \sigma_{3}^{'} )\]
\[\text{(2-19)}\]

For the particular case of equal internal and external pressure, $P_i = P_o = P$, and are usually assumed to be equal to $P$. From Average radial stress $\sigma_r^{'} = (P_o r_o + P_i r_i) / (r_o + r_i)$, $\sigma_2$ is equal to $P$ as well. Therefore, changes in the $\alpha$ angle are accompanied by changes in magnitude of $b$. When $P_i = P_o$
\[ b = \sin^2 \alpha \quad \text{(Hight. et al., 1983)} \quad (2-20) \]

The direction of strain increment \( \alpha_{dc} \) can be calculated from the incremental strain components

\[ \tan 2\alpha_{dc} = d \gamma \theta \frac{1}{(d \varepsilon_z - d \varepsilon_0)} \quad (2-21) \]

The amount of non-coaxiality was defined as the difference between the directions of principal stress and of principal strain increments as, \( \alpha_{dc} - \alpha \).

**Stress distribution in hollow cylinder specimens**

The most critical aspect of the use of hollow cylinder specimen is the nonhomogeneity of stress and strain distributions, developed in the wall of a specimen as a result of curvature of the wall and end restraint. Stress nonuniformity due to curvature can be minimized by selecting an appropriate geometry of the specimen.

Even though hollow cylinder devices offer highly promising capabilities for the study of soil behaviour, their use has been subjected of criticism. These objections arise principally due to the non-uniform distribution of stresses and strains within the specimens. Stress non-uniformities occur across the wall of a hollow cylinder due to the specimen geometry, end restraint, the application of torque or different internal and external pressures. The tested specimen size affects significantly the stress non-uniformity level. When the wall thickness is reduced or the inner radius is increased, the stress distribution becomes more uniform (Sayao and Vaid, 1991).

Because it is not easy to measure either the stresses or the strains across the wall of the hollow cylinder directly, it becomes essential to set bounds to the differences between the calculated and real averages and the magnitude of deviations from the real averages. By using the finite element method and assuming that material behaves as either isotropic or elasto-plastic (modified Cam-clay), Hight et al. (1983) defined the non-uniformity coefficients \( \beta_1 \) and \( \beta_3 \) for
individual stress components, as shown in Figure 2.3. The magnitude of the difference between
calculated and real stress average can be characterized by normalized parameter $\beta_1$:

$$
\beta_1 = \frac{|\sigma^* - \sigma'}{\sigma_L}
$$

(2-22)

where $\sigma^*$ is the real average, $\sigma'$ is the calculated average and $\sigma_L$, which is defined as $[(\sigma_0' + \sigma_r')/2]$, is a measure of the stress level. Therefore $\beta_1$ is inversely related to accuracy. $\beta_3$ is the parameter to quantify the level of non-uniformity of stresses:

$$
\beta_3 = \frac{\int_a^b [\sigma(r) - \sigma^*] \, dr}{[(b-a) \sigma]}\quad (2-23)
$$

where $\sigma(r)$ is the distribution of the particular stress, $\sigma_\theta$, $\sigma_\rho$ or $\tau_{\rho\theta}$ under consideration across the hollow cylinder specimen. $\beta_3$ may be used to minimize the difference between the actual stress distribution and the real average.

For differences in strain averages and strain non-uniformities, similar definitions for $\beta_1$ and $\beta_3$ are used. According to Hight et al. (1983), the magnitudes of $\beta_1$ and $\beta_3$ are dependent on stress state, specimen geometry and the constitutive law of the specimen’s material. The authors recommended keeping stresses within a limit where the ratio of outer to inner cell pressures is $0.9 < P_o/P_i < 1.2$, and $\beta_3$ should be kept below 11%.

Vaid et al. (1990) analyzed non-uniformities in hollow cylinder specimens by using a linear elastic model. By comparing the results with those of a finite element method, they argued that the use of the parameter $\beta_3$ defined by Hight et al. (1983) could lead to an underestimation of the HCA non-uniformities and proposed a different stress non-uniformity parameter across the wall of the specimen in terms of the stress ratio $R$ ($R = \sigma_1'/\sigma_3'$):

$$
\beta_R = (R_{max} - R_{min})/ R'\quad (2-24)
$$
where $R_{\text{max}}$ and $R_{\text{min}}$ are the maximum and minimum stress ratios and is the average value. Wijewickreme and Vaid (1991) indicated that relatively large stress and strain non-uniformities could arise in hollow cylinder specimens, particularly in the small stress/strain (near elastic) region, for certain loading conditions. On the other hand, when large differences between $P_o$ and $P_i$ occurred, the stress non uniformity across the wall became very large. According to their study with non-linear elastic soil, the stress non-uniformity coefficient $\beta_R$ only increased continuously with the stress ratio $R$ at lower values of $R$. $\beta_R$ reached a peak point or even started to decrease when $R$ was higher.

Menkiti (1995) and Porovic (1995) found that in cases free from end restraint, the equations defined by Hight et al. (1983) to calculate average stresses and strain were sufficiently accurate for interpreting hollow cylinder tests. Furthermore, very good agreement was observed between the stress-strain and strength response of hollow cylinder simulations and a uniform single element.

Rolo (2003) used a classical elasto-plastic non-linear, modified Cam-clay soil model with a finite element method to analyze most of the features that were thought to influence the development and magnitude of non-uniformities. The non-uniformity increased as the specimen approached the failure surface, which agreed with the observations by Hight et al. (1983) on specimens with fixed ends. The specimen with free-ends resulted in more uniform conditions. The results revealed that non-uniformities could result in either over or underestimation of certain stress and strain parameters.

**Specimen geometry**
The uniformity of the stress distribution across the wall of hollow cylinder specimens is affected by the specimen geometry, both the curvature and end restraint. This result came from the detailed study of stress distributions using both isotropic linear elastic and plastic formulations to represent the soil in specimens of different geometries under different load combinations. A suitable height of the specimen can engender reasonably uniform distributions of stress (Hight et al., 1983). The differences between real and calculated averages of stress and strain were attributed to the selected specimen geometry and the stress path. As the ratio of inner to outer radii, $r_i/r_o$, approaches unity, both $\beta_1$ and $\beta_3$ reduce. Figure 2.5 was produced by Porovic (1995) by assuming a linear variation of applied shear stresses, $\tau_{\theta z}$, and a linear elastic constitutive law, to display the ratio of maximum and minimum shear stresses to average shear stress for three different specimen dimensions. As the diagram shows, the level of non-uniformity for a fixed wall thickness would reduce with the increase of specimen diameter. Therefore, the degree of the stress difference between the calculated and real average was minimized as the inner radius of specimen increased. The selection of a suitable geometry for the hollow cylinder specimen would reduce stress non-uniformities to an acceptable level. Saada (1988) also quoted that selecting particular specimen geometry played a major role in reducing non-uniformity of stress distribution.

Firstly, for sand specimens, an appropriate wall thickness should be applied to meet the following criteria:

a) A wall thickness sufficiently large enough relative to the maximum grain size of the tested specimen so the failure mechanisms would not be constrained.

b) A specimen volume sufficiently large in relation to the potential volume change resulting from membrane penetration.
c) A uniform density across the wall.

In order to determine a reasonable specimen geometry, based on elasticity theory and the assumption that the central zone, free from end effects should be the same length as the zone influenced by the platens, Saada and Townsend (1981) suggested the following criteria for the specimen geometry:

a) Height: \( H \geq 5.44\alpha \left( r_o - r_i \right) \)

b) Inner radius \( r_i \): \( n = \left( \frac{r_i}{r_o} \right) \geq 0.65 \)

where \( H \) is the height, \( r_i \) and \( r_o \) are the inner and outer radii of the specimen, and \( n \) is the ratio of inner and outer radii.

The criteria proposed by Sayao and Vaid (1991) were as follows:

a) Wall thickness: \( r_o - r_i = 20 \) to 60mm

b) Inner radius: \( 0.65 \leq \left( \frac{r_i}{r_o} \right) \leq 0.82 \)

c) Height: \( 1.8 \leq \left( \frac{H}{2r_o} \right) \leq 2.2 \)

**Membrane penetration errors**

In the hollow cylinder test, rubber membranes are used to enclose the specimens. The effect of membrane penetration on the external measurement of volumetric deformations is attributed to the flexible membrane penetrating into or withdrawing out of the external voids of the soil specimen. The membrane penetration (MP) may influence the computed specimen’s volume change in a drained test, and the magnitude of the pore water pressure measured in an undrained test. Therefore this effect should be accounted for to make a confident assessment of actual stress-strain behaviour of saturated granular materials in a test. For materials of medium sand size having mean particle size of \( D_{50} \geq 0.1 \)mm, particularly for the large diameter specimens,
correction for the membrane penetration is of great importance and should be applied
(Molenkamp and Luger, 1981).

Studies of the effect of membrane penetration have been undertaken and the particle size
of the material is identified to be the major factor to influence the membrane penetration
(Frydman et al., 1973).

Theoretical expressions for the unit membrane penetration suggested by Baldi and Nova
(1984) and Kramer and Sivaneswaran (1989) are as following:

\[ A_{MP} \nu_{MP} = \frac{1}{2} \frac{d}{D} V_{soil} \left[ \frac{(\sigma'_h d)/(E_{m}t_m)}{1/3} \right] \]

\[ \nu_{MP} = 0.395d(1- \alpha) \left[\left(\frac{1-\alpha}{(5+64\alpha^2+80\alpha^4)}\right)^{1/3}\left[\frac{(\sigma'_h d)/(E_{m}t_m)}{1/3}\right]^{2-26}\right] \]

where \( \nu_{MP} \) = unit membrane penetration (in mm); \( A_{MP} \) = surface area of membrane (in mm); \( d \) = mean particle size, \( D_{50} \) (in mm); \( D \) = Specimen diameter (in mm); \( V_{soil} \) = volume of soil specimen (in mm3); \( E_m \) = Young’s modulus of membrane (in kN/m2); \( t_m \) = thickness of membrane (in mm); \( \sigma'_h \) = effective confining pressure (in kPa).

A new approach for the assessment of MP was obtained from the differences between
measured volume strain of the specimen and the volume of the inner chamber using a single
hollow cylindrical specimen under hydrostatic loading by Sivathayalan and Vaid (1998). The
proposed expression for the unit membrane penetration is:

\[ \varepsilon_m = \frac{\Delta V_{sr} - [\Delta V_{ir} (n^2-1)]}{(A_{im}+A_{om})} \]

where \( \varepsilon_m \) is the unit membrane penetration; \( \Delta V_{sr} \) and \( \Delta V_{ir} \) are the measured volume changes of the inner chamber and the specimen, respectively; \( n \) is the ratio of the outer to inner radii of the specimen, and \( A_{im} \) and \( A_{om} \) are the surface areas of the specimen covered by the inner and outer membranes, respectively.
Kuwano (1999) evaluated the apparent volumetric strains due to MP over the vertical sides of the specimens using Ham River Sand specimens with rough and lubricated ends. By comparing the measured volume deformations with a conventional volume gauge and with local instrumentation, she obtained the following relationship for $\nu_{\text{MP}}$ based on isotropic loading/unloading/reloading tests:

$$\nu_{\text{MP}} = C_{\text{MP}} \Delta \log \sigma'_{h} = C_{\text{MP}} \log \left( \sigma'_{h} / \sigma'_{h0} \right)$$  \hspace{1cm} (2-28)

where $C_{\text{MP}}$ is a parameter that depends on specimen size and density, membrane thickness and elastic modulus, and on particle shape and size; $\sigma'_{h}$ and $\sigma'_{h0}$ are the current and initial effective confining pressures. From Kuwano’s experiments, $C_{\text{MP}}$ is 0.015 mm for 100 mm diameter specimens of Ham River Sand encased in a 0.5 mm thick latex membrane. Kuwano (1999) found that Eq. (2-27) matched the expressions suggested by Baldi and Nova (1984) and Kramer and Sivaneswaran (1989) very well.
Figure 2.1 Idealized stress and strain components within the HCA subjected to axial load, $W$, torque, $M_T$, internal pressure, $P_i$, and external pressure, $P_o$: (a) hollow cylinder coordinates; (b) element component stresses; (c) element component strains; (d) element principal stresses (after Zdravkovic and Jardine, 2001).
**Figure 2.2** Definitions of average stresses and strains (after Hight *et al.*, 1983)

**Figure 2.3** Definitions used for stress non-uniformity and accuracy (after Hight *et al.*, 1983)

\[
\beta_1 = \frac{\bar{\sigma}^* - \bar{\sigma}}{\sigma_L}
\]
\[
\beta_3 = \frac{\int_a^b (\sigma(r) - \bar{\sigma}^*) \, dr}{(b-a) \sigma_L}
\]
Figure 2.4 Effect of stress ratio level on non-uniformity coefficients (after Vaid et al., 1990)
Figure 2.5 Shear stress distribution in Hollow cylinder torsional shear test specimens

(after Porovic, 1995).
Design and Fabrication of University of Colorado Hollow Cylinder Test Device

Background

The UCD hollow cylinder test apparatus was designed with specimen dimensions adopted from a similar device in operation at Imperial College, in London (Hight, Gens and Symes 1983).

The University of Colorado at Denver Hollow Cylinder Torsional/Axial test cell, designated HCTA-88, is described in this chapter. Dr. Jing-Wen Chen was responsible for the design and fabrication of the HCTA-88 while conducting his doctoral research at UCD. Much of the chapter is adapted from Dr. Chen’s thesis (Chen, 1988), wherein the development of the apparatus and an inaugural laboratory test program conducted to evaluate its effectiveness for investigation of stress effects in Monterey No. 0/30 sand are described. Additional contemporary research has been conducted since that publication and will be reviewed with emphasis on cyclic loading response.

General Considerations

The evolution of laboratory apparatuses for the investigation of stress-strain behavior of soils in general was presented by Chen (1988). A focused literature review is presented herein on recent research with hollow cylinder soil testing devices for cyclic soil behavior investigation, with particular emphasis on undrained pore pressure response and cyclic strength.

The fundamental purpose for hollow cylinder soil tests (particularly when torsional loading is available) is the exercise of control, independently if desired, over principal and deviatoric stress direction and magnitudes that might be useful in replicating field behavior or
calibrating constitutive models. A hollow cylinder of soil is constructed by either: (1) placement, remolded, by some means between two flexible, impermeable membranes (inner and outer) temporarily restrained in the proper shape against a rigid form, or mold; or (2) trimmed from a larger, intendedly undisturbed sample of in situ material and wrapped by inner and outer impermeable membranes. The membrane-encapsulated specimen, when supported by effective stress due to external pressure or internal vacuum or both resembles a pipe made of soil.

Confining fluid pressure may be independently applied to the inner and outer membranes to vary the distribution of total static stresses within the specimen. Pore fluid (usually water) may pass into or out of the specimen through the ends of the cylinder; pressure within the specimen voids is varied and usually measured through the ends as well to regulate global effective stress. Porous, flat rings of stone or other rigid material are usually employed as filters at the ends to prevent solids from washing out with fluid flow. Axial loads or torque or both are applied to an end cap or pedestal at one end or the other of the cylindrical specimen and are transmitted to the confined soil through the porous elements.

Development of Specifications

The UCD hollow cylinder test apparatus was designed in accordance with the conditions listed earlier as established by Lade (1981) and within the constraints imposed by five aspects common to all laboratory soil testing: specimen dimensions, instrumentation placement and purposes, anticipated specimen strengths, loading machine capacity, and specimen preparation considerations. The UCD apparatus includes features to accommodate construction of soil specimens within the device, membrane encapsulation, and drainage provisions.
The ultimate size of the UCD apparatus was dictated by specimen dimensions. The platens and membrane retention components of the device were built to accommodate preparation and testing of 10-inch tall hollow cylinders of soil with an inside diameter of 8 inches and an outside diameter of 10 inches. These dimensions were shown by Hight, Gens and Symes (1983) to produce essentially uniform stress and strain distributions across specimen wall thickness and with height, particularly along the central 5 inches of length. This central section should be region within which test parameters are measured for detailed stress-strain analysis; overall specimen strengths were investigated in the present study as measured at the specimen ends, and should not be strongly affected by internal non-uniformities in an apparatus with the above dimensions.

Test measurement instrumentation versatility was considered in the design of interior space for the pressure chamber and its supporting framework. Future researchers using the UCD apparatus may desire to install deformation and stress monitoring devices within the hollow center of the soil specimen, within the specimen itself, or within the annulus between the specimen and the pressure chamber. Sufficient working space is available to these purposes. The apparatus is ported to allow independent external control of fluid flow or pressure measurement to or from the specimen and the confining fluid spaces.

The UCD apparatus was conceived to subject soil specimens to both axial and torsional loading, depending on the load frame into which the chamber is installed for test. To this end, external loading specifications were back-calculated from axial compression and shear resistances typical of an arbitrary test soil. A cohesionless soil with an effective internal friction angle of 35°, if subjected to an assumed maximum external confining pressure of 250 psi and a back pressure of 50 psi could be expected to resist up to 740 psi axial compressive stress, as
represented by the Mohr’s circle diagram in Figure 2.6. By similar reasoning (Figure 2.7), the same specimen would fail in pure shear at 115psi maximum applied shear stress.

Chen (1988) assumed a loading piston diameter of 2 inches to calculate the maximum vertical load required to fail such a specimen as follows:

\[
(740.0-200.0) \times [(10)^2 - (8.0)^2] + \pi/4 + 250.0 \times (2)^2 \times \pi/4 = 16053.0 \text{lb} \quad (2-29)
\]

The loader must apply the failure stress in addition to the uplift on loading piston caused by confining pressure, as represented by the second term in Equation 2-30.

\[
\frac{[\tau_{0z}]_{\text{max}} - (\tau_{0z})_{\text{avg}}}{(\tau_{0z})_{\text{avg}}} = n(1-n)/(1+n^2) \quad (2-30)
\]

Torque required to fail the same specimen was back-calculated by adapting Equation 2-6, substituting design dimensions and the maximum shear stress and rewriting:

\[
M_t = \frac{[2\pi \times \tau_{0z} \times (r_o^3 - r_i^3)]}{3} = \frac{[2\pi \times 115 \times (5^3 - 4^3)]}{3} = 14.692 \text{ in.-lb} \quad (2-31)
\]

The test apparatus was designed to accommodate, in consideration of the above calculations, at least 20,000 lb vertical load and 20,000 in-lb torque. Specifications developed for the purchase of the loader used in the current research were based on these computations. The loader, an Instron™ Model 1322 machine with 50,000 lb axial (compressive or tensile) and 25,000 in-lb torsional capacity, is described in the following chapter.

The apparatus is designed to permit preparation of test specimens directly on the lower pedestal, in the same fashion as conventional triaxial specimens. The relatively thick cylindrical wall of the design specimens, 1 inch, provides adequate working room for preparation of uniform reconstituted specimens.
Figure 2.6 Mohr’s circle for determining maximum vertical stress in HCTA-88 (Chen, 1988)
Figure 2.7 Mohr’s circle for determining maximum shear stress in HCTA-88 (Chen, 1988)
Figure 2.8 Schematic diagram of the HCTA-88 at the University of Colorado at Denver Geotechnical Laboratory (Chen, 1988)
Parts and their functions

The new hollow axial-torsional cylinder apparatus as shown in Figure.2.8 consists of eleven detachable components and three accessories used for sample preparation.

The bottom platen as shown in Figure.2.9 is constructed of aluminum and is used to seal the bottom of the inner chamber.

The platen is twelve inches in diameter to match the twelve inch diameter of the MTS load cell platen to avoid apparatus alignment problems. The top of the cylindrical platen has a counter bore to align and constrain the inner mold from movement during sample preparation. Instrumentation could be placed on top of the platen in the inner chamber area for future improvements. There are six holes in the outer rim of the platen for fastening the platen to the hollow cylinder base plate. Four threaded holes fasten the bottom platen to the MTS load cell. An 0-ring on top of the bottom platen seals the platen to the cylinder base plate. The bottom platen has one drainage path to supply water to the inner chamber of the sample.

The hollow cylinder base plate as shown in Fig. 2.10 is constructed of aluminum and has three tiers. The outside diameter is nineteen inches, the intermediate annulus is sixteen inches and the inner diameter is twelve inches. The center bore is nine inches in diameter. The outer annulus has four drainage paths that allow for connection to the inner chamber, outer chamber, top of sample and bottom of sample. These holes are drilled laterally into the base plate and allow for the supply of water to the inner and outer chambers, and measurement of pore pressure or volume change of a sample during testing. The exterior of the second tier has a built-in o-ring to seal the pressure chamber to the cylinder base plate. There are four holes in the top of the second tier to install support bars to connect the top plate to the base plate. In the future instrumentation could be placed in between the support bars and between the inner and outer
chambers. The inner tier has six threaded holes on the top to connect the bottom sample pedestal to the base plate, and six threaded holes on the bottom to connect the bottom platen to the cylinder base plate. An o-ring on top of the inner tier seals the sample pedestal from the cylinder base plate.

The bottom sample pedestal is constructed of aluminum and has two tiers on top and three tiers on the bottom as shown in the left hand side of Fig. 2.11. The outside diameter of the pedestal is twelve inches. The top annulus is nine inches in diameter. The bottom intermediate annulus is eight and one-half inches in diameter and the bottom inner annulus is eight and three eighth inches in diameter. The inner bore is eight inches in diameter. The one inch thickness of the top tier exactly matches the thickness of the sample tested. There are six holes bored in the top of the outer annulus to connect the bottom pedestal to the cylinder base plate. The exterior of the bottom annulus is grooved for o-rings and provides the seal for the inner chamber membrane. The exterior of the top annulus is grooved for o-rings and provides the seal for the outer chamber membrane. The top of the outer annulus has a drainage path to connect to the sample being tested. This path is drilled laterally into the base plate and connected to the sample to measure pore pressure or volume change of the sample during testing. A semicircle groove one sixteenth inch deep is circumscribed along the top of the sample pedestal and is connected to the drainage path. Twelve one inch wide, one quarter inch thick pieces of bronze porous stone are bolted to the top of the top tier, above the circumscribed groove. The porous stones provide filtration to prevent migration of the soil grains. Twelve stainless steel plates one sixteenth inch thick by one inch wide separate each of the twelve stones and protrude one quarter inch above the stones. These blades counter torque applied to the top of the sample.
The top sample pedestal is constructed of aluminum and is similar to the bottom sample pedestal in design as shown in the right hand side of Fig.2.11. The top of the sample pedestal has two tiers and the bottom of the sample pedestal has two tiers. The outside diameter is eleven and one-half inches. The top annulus has an outside diameter of eight and three-eighth inches. The bottom annulus is one inch thick and has an outside diameter of ten inches. The thickness exactly matches the sample thickness. There are six threaded holes bored in the outer annulus to connect the top sample pedestal to the top cap. The exterior of the top annulus is grooved for 0-rings to provide the seal for the inner chamber membrane. The exterior of the bottom annulus is grooved for 0-rings to provide the seal for the outer chamber membrane. The outer annulus has a drainage path connecting to the top of sample. This path is drilled laterally into the top platen and is connected to the sample to measure its pore pressure or volume change during testing. A semi-circular groove of one sixteenth inch deep is circumscribed along the bottom of the top sample pedestal and is connected to the drainage path. Twelve one inch wide one quarter inch thick pieces of bronze porous stone are bolted to the top of the top tier, above the circumscribed groove. The porous stones provide filtration to prevent migration of the sample media. Twelve stainless steel plates one sixteenth inch thick by one inch wide separate each of the twelve stones, and protrude one quarter inch above the stones. These blades transmit applied torque to the top of the sample.

The top cap is constructed of aluminum as shown in Fig.2.12. The top cap is connected to the loading piston and is used to transmit the applied loads to the top sample pedestal. The top cap also serves to separate the inner and outer chambers. The outside diameter is eleven and one-half inches, and the interior counter bore is nine inches in diameter. The outer perimeter has six vertical holes bored in it to connect the top cap to the top sample pedestal. The top of the top cap
has a three inch tall socket with a four inch outside diameter and a two inch interior counter bore. The middle of the socket has a laterally bored tapered hole drilled through both sides to provide for connection to the loading piston. The top cap has two one-half inch diameter holes that are used to equalize the inner and outer chamber pressure. If measurement of volume change in the inner chamber is required, each of the two holes in the top cap can be connected to one of the two drainage paths located in the top plate. One drainage path can be pressurized and the other path can be used to measure the volume change.

The top plate as shown in Fig. 2.12 is constructed of aluminum and is fifteen and three-quarter inches in diameter. The top plate has four holes bored to fasten the support bars connected to the cylinder base plate to the top plate. The exterior of the top plate has a built-in O-ring to seal the outer chamber cell to the top plate. A four inch diameter threaded hole is centered in the top plate and is used to attach the top plate to the piston bushing sleeve. Four locking tabs constructed of aluminum are bolted to the exterior portion of the top plate to prevent slippage of the exterior pressure chamber. Two one-half inch drainage holes are bored in the top plate. These holes function as an air bleed to allow cyclic pressurization during the cyclic test. When cyclic testing is conducted to the inner chamber and the other is connected to the outer chamber.

The piston bushing sleeve is constructed of aluminum and is nine inches tall and has a five inch outside diameter. The bottom end of the piston bushing sleeve is threaded to attach the sleeve to the top plate. Once attached, these two pieces form a permanent integral part of the cell and are not separated. Two stainless steel Thompson A-324864-SP bushings are housed in the center of the sleeve. These bushings have a two inch interior diameter to match the diameter of the loading piston. A double lip wiper ring is installed in the bottom of the sleeve to seal the pressure leakage and prevent contamination of the Thompson bushing from the loading piston.
The top of the sleeve has an annulus plate five inches in diameter used to prevent slippage of the Thompson bushing.

A piston lock mechanism as shown in Fig. 2.13 is constructed of stainless steel is connected to the top plate of the sleeve to prevent unintentional piston movement. The lock mechanism consists of two halves of a ring with lateral bolts that when tightened will prevent up and down motion of the piston. One curved slot is bored in the center portion of each half and is used to bolt the lock to the top sleeve plate. Torsional rotation of the load piston is prevented when the lock is bolted to the top sleeve plate.

The loading piston is constructed of 440 C stainless steel and is twenty inches tall and slightly less than two inches in diameter with a finely polished surface finish. A one-half inch diameter tapered bore is located at the bottom of the piston. A tapered pin that is threaded on the small end is used to connect the piston to the top cap. A nut and washer is used on the small end to produce a movement free connection.

Four support bars as shown in Fig. 2.14 are constructed of 440 C stainless steel. These bars are twenty three and three-quarter inches tall with a one inch diameter and are used to connect the cylinder bottom plate to the top plate. Each end of the support bar is counter bored and tapped to allow bolts to secure the bars to the plates. Both ends of the support bars have O-rings to prevent leakage from the outer chamber.

The pressure chamber as shown in Fig. 2.15 is constructed of aluminum and cast acrylic. The total length is twenty five and one-half inches with an outer diameter of eighteen inches and interior diameter of fifteen and one-half inches. Each end of the cast acrylic chamber is threaded to fasten aluminum end rings. Each of the rings is four inches tall. The bottom aluminum end
ring has an exterior and interior diameter equal to the cast acrylic, and has a threaded counter bore to allow the ring to attach to the cast acrylic. An o-ring is placed between the ring and acrylic to prevent any leakage. The top ring is similar to the bottom ring, but the top two inches has a smaller interior diameter. This is done to enable the pressure chamber to slip over the top plate and cylinder base plate without contacting the o-ring until the final two inches of movement.

**Sample Preparation Accessories**

Three accessories are used in the hollow axial torsional cylinder sample preparation: an inner mold, an outer mold, and an acrylic pipe stand.

The inner mold as shown in Fig. 2.16 is constructed of two aluminum caps, three maple hardwood molds and an inner expansion pipe plug. The inner mold fixes the inner specimen diameter and supports the inner membrane during the sample preparation. The bottom aluminum cap of the inner mold is two inches tall and is slightly less than eight inches in diameter. The thickness of the membrane plus the diameter of the mold exactly equals to eight inches, inner sample diameter. The bottom of the aluminum bottom cap has a dowel that fits the counter bore of the bottom platen. The top cap is similar to the bottom cap, but has a one and one-quarter inch diameter center hole to allow for pressurizing and releasing the inner expansion plug. The three maple hardwood plugs are thirteen inches tall and has an exterior diameter equal to the aluminum caps, and an inner diameter of four and one-half inches to accommodate the expansion plug. A pipe plug is used to act as the expansion plug in the interior of the hardwood molds.

The outer mold as shown in the Fig. 2.17 is constructed of aluminum and provides the outer sample shape and support of the outer membrane during sample preparation. The outer
mold is made of two longitudinal halves of an aluminum cylinder with an outside diameter of slightly more than ten inches and the inner diameter of the outer mold minus the two times membrane thickness exactly equals the ten inch, outer sample diameters. Grooves are inscribed along the interior of the mold to provide for even distribution of the vacuum applied to the outer membrane. Each half of the mold has two three-quarter inch wide flanges used to fasten the two halves of the mold together using bolts. Two ports consisting of five small holes in each port are used to apply vacuum to evacuate air from between the mold and membrane.

A stand as shown in Fig. 2.18 is constructed of acrylic pipe and is used to support the bottom pedestal during sample preparation. The acrylic stand is twenty and one-half inch tall. It is used to position the bottom pedestal with respect to the inner mold during the sample preparation. There are twelve notches in the rim of the stand. They are one-quarter inch deep to match the twelve stainless steel blades of the bottom pedestal.

**Sample Preparation Procedures**

Preparation of a hollow cylinder sand specimen consisted of the following steps.

(1) The thickness of the rubber membrane along the axial direction is nonuniform, therefore the membrane thickness is measured only along the areas where the sample is located. The measurements are obtained at the top, middle and bottom of the sample in orthogonal directions.

(2) Assemble the inner mold pieces with the balloon inside. Place the aluminum end caps on the top and bottom of the inner mold pieces. Inflate the inner balloon to around 10 psi. At the beginning of the assembly process, the inner mold is in an inverted position. After step (6), the assembly is inverted to the upright position.
(3) Wrap the inner membrane around the already assembled inner mold.

(4) Place the plastic stand around the inner mold assembly.

(5) Slip the bottom pedestal into the inner mold and partially deflate the balloon that is inside the inner mold. Once the balloon is partially deflated, the bottom pedestal will slip down to rest on top of the plastic stand.

(6) Apply a thin layer of vacuum grease around the outer edge of the bottom sample pedestal. Wrap the membrane around the edge and seal the membrane using several o-rings.

(7) Invert the inner mold and plastic stand assembly, and place the assembly on top of the bottom plate. Bolt the bottom pedestal to the bottom plate using 6 allen head bolts.

(8) Remove the plastic stand from the assembly. Place a piece of ring shaped membrane on the top of the allen head bolts. This piece of ring shaped membrane is a seal to prevent vacuum leakage when a vacuum is applied between the outer membrane and outer mold.

(9) Apply a thin layer of vacuum grease around the outer edge of the exposed top of the bottom sample pedestal. Place the outer membrane over the exposed outer edage of the bottom sample pedestal and seal the membrane using several 0-rings.

(10) Place the outer mold over the inner assembly and on top of the ring shaped membrane. Place a thick layer of vacuum grease around the bottom outer edge of the outer mold to seal the gap between the outer mold and the ring shaped membrane.

(11) Wrap the outer membrane around the upper end of the outer mold. Seal the membrane to the upper end of the outer mold using one o-ring.
(12) Apply a vacuum to the vacuum ports located on the wall of the outer mold. This will create a vacuum in the space between the inner wall of the outer mold and the outer membrane. Shape the outer membrane to be free of any wrinkles.

(13) Weigh the desired amount of sand to be used in the sample.

(14) Using the long neck funnel, uniformly deposit the sample sand in the space between the outer membrane and the inner mold in a consistent fashion. A uniform deposit is achieved by placing the mouth of the long neck funnel at the top of the sand deposit so that there is no distance for the sand to drop.

(15) Smooth the surface of the sand with a wooden plate. It is very important that the top of the sand sample is smooth to prevent necking of the sample.

(16) Supply deaired water to the bottom of the sample. Flow rate and seepage pressure must be controlled to prevent sample disturbance. A total of 2 psi pressure was applied in the preparation of all of the samples used in this study.

(17) Place the top sample pedestal on top of the wet surface of the sample sand. Press the top sample pedestal into the sand until the stainless steel torsion plates are fully embedded into the sample. Use a bubble level and a height caliper to obtain a sample of uniform height.

(18) Apply a thin layer of vacuum grease around the outer edge of the top sample pedestal. Wrap the outer membrane around the top sample pedestal and seal the membrane using several o-rings.

(19) Apply a thin layer of vacuum grease around the annular of the top sample pedestal. Wrap the inner membrane around the top sample pedestal asn seal the membrane using several o-rings.
(20) Disconnect the vacuum from the ports on the outer wall of the outer mold. Slowly apply vacuum to the top of the sample. A 10 psi vacuum was applied to the sample used in this study.

(21) Remove the outer mold from the sample assembly. Deflate the balloon from the inner mold, and remove the inner mold, piece by piece. The aluminum bottom cap of the inner mold cannot be removed from the top of the sample.

(22) To remove the aluminum bottom cap, loosen the bottom platen from the base plate. Remove the inner mold aluminum bottom cap and replace the bottom platen. Tighten the six allen head bolts connecting the bottom platen to the base plate.

(23) Measure the outer diameter of the sample at the top, middle and bottom of the sample. A “Pi-Tape” will read the sample diameter directly. Subtract the membrane thickness from the top, middle and bottom of the sample using the membrane thickness values obtained earlier. Record the top, middle and bottom sample diameters.

(24) Measure the inner dimension of the sample using an inside micrometer. Measure points at top, middle and bottom of the sample at a spread of $0^\circ$, $45^\circ$, $90^\circ$ and $135^\circ$ from the initial orientation. This will give 12 points of measurement. Add the average membrane thickness readings obtained at the top, middle and bottom of the inner membrane, to the readings obtained above. This will give the top, middle and bottom inner sample diameter.

(25) Measure the height of the sample from the bottom of the bottom plate, to the top of top sample pedestal. Obtain readings at three equally spaced points, using a height caliper. Subtract the height of the top sample pedestal portion that the top cap will rest
on, from the overall height dimension. Subtract the bottom sample pedestal portion that rests on the bottom plate. This will give the average final height of the sand sample.

(26) Install the four stainless steel support bars on the base plate and bolt them to the plate using four allen head bolts.

(27) Connect the piston and top cap with the tapered pin. Fasten the pin using the spacer and nut.

(28) Slip the top plate over the piston assembly a short distance, and tighten the locking mechanism to lock the piston.

(29) Place the top plate and piston assembly on the top of the four stainless steel support bars. Bolt the plate to the bars using the four allen head bolts.

(30) Loosen the locking mechanism on the piston and carefully lower the piston and top cap assembly to rest on the top sample pedestal. Do not disturb the sample during this procedure.

(31) Tighten and piston locking mechanism and connect the top cap and top sample pedestal using the sic allen head bolts. When torque is applied to the allen head bolts, the locking mechanism will absorb the torque reaction, not the sample.

(32) Using a forklift machine, lower the entire assembly below the hanging chamber cell. Center the assembly beneath the cell and raise the assembly into the chamber cell.

(33) Using two large bar clamps, clamps the chamber cell to the assembly.
(34) Shift the chamber cell stoppers located on the top plate, into the proper positions. Tighten the stoppers in the lock position to prevent uplift of the chamber cell during the test.

(35) Carefully mount the entire assembly in the MTS machine. Mount the assembly on top of the load cell plate and connect the assembly to the load cell plate using four allen head bolts.

(36) Supply water to the inner and outer chamber of the sample simultaneously.

(37) Supply deaired water to the bottom of the sample, flushing the air out of the sample through the top sample pedestal.

(38) After it appears the air has been flushed out of sample, gradually apply a confining pressure increment of 2 to 3 psi while lowering the vacuum by the same increment, maintaining the initial confining stress placed on the sample by the vacuum.

(39) Continue to decrease the vacuum in increments until zero vacuum. The confining stress is now due entirely to confining pressure.

(40) Adjust the confining pressure and back pressure simultaneously until the desired values are obtained.
Test Results

Due to missing large membranes for Hollow Cylinder Test, any tests were not performed.

There did not show any Hollow Cylinder Test results.
Figure 2.9 Bottom Platen
Figure 2.10 Base Plate
Figure 2.11 Bottom Sample Pedestal (left) and Top Sample Pedestal
Figure 2.12 Top Cap and Top Plate
Figure 2.13 Piston Locking Mechanism
Figure 2.14 Supporting Bars
Figure 2.15 Pressure Chamber
Figure 2.16 Inner Mold
Figure 2.17 Outer Mold
Figure 2.18 Plastic Stand