LIQUEFACTION RESISTANCE OF MONTEREY NO.0/30 SAND CONTAINING FINES UNDER CYCLIC TRIAXIAL AND CYCLIC HOLLOW CYLINDER TESTS

by

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A thesis submitted to the
Faculty of the Graduate School of the
University of Colorado Denver in partial fulfillment
of the requirements for the degree of
Doctor of Philosophy
Civil Engineering Program
2019
This thesis for the Doctor of Philosophy degree by
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Date: May, 18 2019
Liu, Jungang (Ph. D, Civil Engineering)
Liquefaction Resistance of Monterey No. 0/30 Sand Containing Fines under Cyclic Triaxial and Cyclic Hollow Cylinder Tests

Thesis directed by Professor Nien-Yin Chang

ABSTRACT

Liquefaction is the most detrimental ground failure caused by strong earthquakes. Ground liquefaction leads to associated foundation and superstructure failures due to loss of bearing capacity and excessive deformation and an appropriate assessment of liquefaction is critical to the seismic safety evaluation of foundations and super structures. This study concerns the liquefaction resistance of Monterey No. 0/30 sand contacting fines. Monterey No. 0/30 sand is a uniform clean medium sand. Fines were prepared by sieving Leyden clay through a U.S.# 200 sieve. One hundred fourteen isotopically consolidated undrain cyclic triaxial tests and thirty-seven cyclic hollow cylinder test were performed to investigate the effect of fines content on liquefaction resistance of soils. This research includes compare and relate the soil liquefaction resistance found by cyclic triaxial and cyclic hollow cylinder test results on uniform clean Monterey No.0/30 sand and soil sample with different percentage of fines content.

The cyclic triaxial and cyclic hollow cylinder liquefaction resistances of soils were expressed in terms of liquefaction potential curves. From the liquefaction potential curves, stress ratio in cyclic triaxial and cyclic hollow cylinder tests causing initial liquefaction in 10 cycles, 30 cycles, 40 cycles and 50 cycles were chosen as dependent variables in the regression model. Three independent variables of regression models, deviator stress in cyclic triaxial test, DS (cyclic shear stress in cyclic hollow cylinder test, CSS); fine content (decimal), FC; consolidation pressure, CP, were eventually selected for final statistical analysis.
In addition to the evaluation of liquefaction potential, an excess pore pressure generation was simulated using Horita constitutive model with the parameters evaluated by laboratory tests. This research includes comparison of excess pore pressure generation between simulations from Horita’s constitutive model and measured from cyclic triaxial test. The new findings of this study can be applied to better understand the field evaluation of liquefaction potential of soils containing fines and effectively assess the liquefaction resistance of soils.

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

Approved: Nien-Yin Chang
ACKNOWLEDGEMENTS

I would like to express my sincerest appreciation to my advisor, Professor Nien-Yin Chang for his constant support and guidance throughout the course of this study. Gratitude is extended to Dr. Shideh Dashti, Dr. Dobroslav Znidarcic, Dr. Brian Brady, Dr. Nghiem Hien and Dr. Shingchun Wang for serving on my final examination committee.

I would like to thank my student colleague, Mr. Brian Volmer for his friendship and help in preparing this study. Thanks are extended to Mr. Tom Thuis and his staff in Calibration shop at the University of Colorado Denver for their assistance in instrumentation.

I would like to thank my wife, Liang Feng for many years of her endurance, sacrifice, assistance and encouragement, which made this accomplishment possible. I also thank my parents for their understanding and support.
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CHAPTER I
INTRODUCTION

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. 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.”

The enormous damage due to soil liquefaction observed in both in Anchorage, Alaska and Niigata, Japan earthquake of 1964 stimulate geotechnical engineering studies of earthquake-induced liquefaction.

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. For the past 50 years, liquefaction problems have received a lot of attention and great efforts have been made to understand the liquefaction mechanism and phenomena. However, due to the complexity of liquefaction and spreading problems, such as soil properties (fine content) effect soil liquefaction resistance, limitations on experimental and constitutive model, interaction between liquefiable soil and foundation, and so
on, reliable and accurate predictive methods have yet to be developed. Therefore, nowadays, soil liquefaction is still one of the hottest research topics in the geotechnical engineering community.

**Objectives and Scope**

The major objective of this study is to investigate methods for liquefaction resistance evaluation, the factors affecting liquefaction resistance, investigate the effect of fines inclusion on liquefaction resistance under cyclic triaxial and cyclic hollow cylinder tests, excess pore water pressure generation characteristics and refine current liquefaction resistance assessment procedure and criterion to cover the effect of fines. To achieve the objectives, cyclic triaxial tests and hollow cylinder tests on samples with different percentages of fines of varying fines contents were and will be performed at the Center for Geotechnical Engineering Science at the University of Colorado Denver. Statistical analysis and modeling of test results will be conducted, including the following tasks: 1) review existing methods for liquefaction potential analysis. 2) review factors affecting liquefaction resistance of soils. 3) perform cyclic triaxial and cyclic hollow cylinder tests to evaluate the extent of effects of fines content and plasticity on liquefaction resistance of soils. 4) evaluate relationship of soil liquefaction resistance from between cyclic triaxial and hollow cylinder tests on uniform clean Monterey No.0/30 sand and soil sample with different percentage of fines content. 5) formulation of statistical relationship for predicting liquefaction resistance of soil containing fines. 6) evaluate threshold fines content effect on liquefiable soil. 7) invesitng excess pore pressure generation from laboratory test results and HORITA’s constitutive model. 8) refinement of current procedure for evaluating liquefaction resistance of soil covering fines.
Significance of This Research

In this research, by using cyclic triaxial and cyclic hollow cylinder tests for evaluating liquefaction resistance, it helps us better understand the relationship between cyclic triaxial and hollow cylinder tests on uniform clean Monterey No.0/30 sand and soil sample with different percentage of fines content. This research can accurate threshold fines content effect on soil liquefaction resistance, and develop appropriate statistical model for predicting liquefaction resistance of soil containing fines. The new findings of this study can be applied to better realize the field evaluation of liquefaction potential of soils containing fines and effectively assess the liquefaction resistance of soils.
CHAPTER II

LITERATURE REVIEW 1: LIQUEFACTION RESISTANCE

Liquefaction Resistance from Earthquake Case History Data

1964 Niigata Earthquake

In 1964 at 1:01 p.m. on June 16, 1964, a violent earthquake hit Niigata and Yamagata prefectures, inflicting considerable damage on the city of Niigata for a magnitude 7.5 earthquake. In Niigata, where sand deposits in lowland areas are widespread, the damage was primarily associated with the liquefaction of loose sand deposits. Buildings not imbedded deep in firm strata sank or tilted toward the direction of the center of gravity. Underground structures, such as septic and storage tanks, sewage conduits and manholes, floated up a meter or two above ground level. In flat fields, sand flows and mud volcanos ejected water and sand 2 to 3 minutes after the quake. Liquefied sand 20 to 30 cm thick covered the entire city, as if the whole area had been devastated by flooding. Damage to modern bridges was also extensive. Most notable was the crumbling of five girders of the Showa Bridge, which crosses the Shinano River in the downtown area. The foundation piles were bent excessively due to the loss of lateral resistance of the riverbed sand deposit, and this caused the simply-supported girders to fall. Figure 2.1 is a map of Niigata city, in which the places where liquefaction developed are indicated. It is to be noted that the whole city was built on layers of sand as deep as about 100 meters, although the origins of these deposits are somewhat different from one location to another. A typical shallow-depth soil profile is shown in Fig.2.2 and describes the soil condition at Kawagishicho, where the land was formed about 40 years ago by reclaiming an old river course.
Fig. 2.1 Niigata city destroyed by the Niigata Earthquake 1964. (National Geophysical Data Center)
1964 Alaska Earthquake USA

On Friday, March 27, 1964, a great earthquake of magnitude 9.2 struck Prince William Sound and caused severe damage in the form of landslides and liquefaction.

The major effects of liquefaction in the Alaska earthquake were massive landslides in the cities of Anchorage, Seward, Valdez and around the borders of Kenai Lake. Liquefaction in sand layers, and in sand and silt seams in the clayey soils beneath Anchorage, caused many of the destructive landslides that occurred during the earthquake. The liquefied seams and lenses disturbed the sensitive clays, and caused their strengths to drop below the levels needed for stability. It is shown for an explanation of road embankment failure in Figure 2.3. Lateral
spreading in the soil beneath the roadway embankment caused the embankment to be pulled apart, producing the large crack down the center of the road shown in Figure 2.4.

**Fig2.3** Cracked Highway in 1964 Alaska Earthquake (Timothy J. Walsh, et. al 1995)

**Figure 2.4** Road Embankment Failure Caused by Soil Liquefaction. (Timothy J. Walsh, et. al 1995)

**1995 Hyogoken-Nanbu Earthquake**

It occurred on Tuesday, January 17, 1995, at 05:46 JST (January 16 at 20:46 UTC) in the southern part of Hyōgo Prefecture, Japan. It measured 6.8 on the moment magnitude scale (USGS), and Mj7.3 (adjusted from 7.2) on JMA magnitude scale. The tremors lasted for approximately 20 seconds. The focus of the earthquake was located 16 km beneath its epicenter,
on the northern end of Awaji Island, 20 km away from the city of Kobe. Figure 2.5 shows the locations of the aftershocks.

The liquefaction and non-liquefaction data that were used in the subsequent liquefaction triggering analysis (Moss, 2003) are summarized in the accompanying Table 2.1. Figure 2.6 is a map of the greater Kobe Port region that shows the approximate locations of these CPT Field case history sites (which have a three letter designation corresponding to the trace); superimposed over SPT field case history sites (which have number designation corresponding to the log) and PGA contours, the latter two are from Cetin et al. (2000)

**Fig 2.5** Location of aftershocks (Moss, 2003)
Table 2.1: Summary of CPT Field case histories from the 1995 Hyogoken-Nambu (Kobe) earthquake. (Moss, 2003)

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<tr>
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<th>DATA CLASS</th>
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<td>2.5 to 5.0</td>
<td>3.75</td>
<td>0.42</td>
<td>2.00</td>
<td>0.38</td>
<td>0.13</td>
<td>0.13</td>
<td>7.00</td>
<td>1.15</td>
<td>0.66</td>
<td>0.22</td>
<td>0.56</td>
</tr>
<tr>
<td>Taito Kobe Factory</td>
<td>Y</td>
<td>B</td>
<td>3.2 to 4.2</td>
<td>3.70</td>
<td>0.17</td>
<td>1.60</td>
<td>0.39</td>
<td>0.13</td>
<td>0.13</td>
<td>4.95</td>
<td>0.85</td>
<td>0.39</td>
<td>0.12</td>
<td>0.75</td>
</tr>
<tr>
<td>Tokioka Concrete Factory</td>
<td>Y</td>
<td>B</td>
<td>4.0 to 4.6</td>
<td>4.40</td>
<td>0.13</td>
<td>2.00</td>
<td>0.40</td>
<td>0.13</td>
<td>0.13</td>
<td>2.50</td>
<td>0.68</td>
<td>0.40</td>
<td>0.19</td>
<td>0.80</td>
</tr>
<tr>
<td>Nisshiki Kobe Oil Tank A</td>
<td>Y</td>
<td>B</td>
<td>4.8 to 6.1</td>
<td>5.45</td>
<td>0.22</td>
<td>2.40</td>
<td>0.43</td>
<td>0.14</td>
<td>0.14</td>
<td>5.30</td>
<td>1.31</td>
<td>0.61</td>
<td>0.36</td>
<td>0.72</td>
</tr>
<tr>
<td>Nisshiki Kobe Oil Tank B</td>
<td>Y</td>
<td>B</td>
<td>5.0 to 6.0</td>
<td>5.50</td>
<td>0.17</td>
<td>2.40</td>
<td>0.43</td>
<td>0.14</td>
<td>0.14</td>
<td>6.28</td>
<td>1.34</td>
<td>0.74</td>
<td>0.27</td>
<td>0.70</td>
</tr>
<tr>
<td>New Port No. 6 Pier</td>
<td>Y</td>
<td>B</td>
<td>3.5 to 5.5</td>
<td>4.00</td>
<td>0.33</td>
<td>2.50</td>
<td>0.42</td>
<td>0.14</td>
<td>0.14</td>
<td>9.47</td>
<td>1.60</td>
<td>1.40</td>
<td>0.31</td>
<td>0.70</td>
</tr>
<tr>
<td>Minatojin Junior High</td>
<td>Y</td>
<td>B</td>
<td>4.0 to 4.5</td>
<td>4.25</td>
<td>0.08</td>
<td>2.70</td>
<td>0.32</td>
<td>0.10</td>
<td>0.10</td>
<td>4.71</td>
<td>1.35</td>
<td>0.94</td>
<td>0.42</td>
<td>0.66</td>
</tr>
<tr>
<td>New Wharf Const. Offices</td>
<td>Y</td>
<td>B</td>
<td>3.2 to 3.8</td>
<td>3.50</td>
<td>0.10</td>
<td>2.00</td>
<td>0.31</td>
<td>0.10</td>
<td>0.10</td>
<td>3.56</td>
<td>0.81</td>
<td>0.93</td>
<td>0.64</td>
<td>0.64</td>
</tr>
<tr>
<td>Fukutomi Park</td>
<td>N</td>
<td>C</td>
<td>11.0 to 12.5</td>
<td>11.75</td>
<td>0.32</td>
<td>3.10</td>
<td>0.35</td>
<td>0.16</td>
<td>0.16</td>
<td>17.09</td>
<td>3.45</td>
<td>1.42</td>
<td>0.57</td>
<td>0.40</td>
</tr>
<tr>
<td>Honyo Central Park</td>
<td>N</td>
<td>B</td>
<td>4.0 to 6.0</td>
<td>5.00</td>
<td>0.33</td>
<td>2.50</td>
<td>0.48</td>
<td>0.16</td>
<td>0.16</td>
<td>17.30</td>
<td>3.75</td>
<td>1.60</td>
<td>0.26</td>
<td>0.56</td>
</tr>
<tr>
<td>Kobe Art Institute</td>
<td>N</td>
<td>B</td>
<td>3.5 to 3.8</td>
<td>3.85</td>
<td>0.08</td>
<td>3.00</td>
<td>0.32</td>
<td>0.10</td>
<td>0.10</td>
<td>13.94</td>
<td>5.38</td>
<td>1.90</td>
<td>1.31</td>
<td>0.33</td>
</tr>
<tr>
<td>Yoshida Kobe Factory</td>
<td>N</td>
<td>B</td>
<td>3.0 to 5</td>
<td>4.00</td>
<td>0.33</td>
<td>3.00</td>
<td>0.33</td>
<td>0.11</td>
<td>0.11</td>
<td>9.43</td>
<td>7.22</td>
<td>2.71</td>
<td>2.73</td>
<td>0.34</td>
</tr>
<tr>
<td>Shimokawaji Park</td>
<td>N</td>
<td>B</td>
<td>3.0 to 4.5</td>
<td>3.75</td>
<td>0.17</td>
<td>2.00</td>
<td>0.50</td>
<td>0.16</td>
<td>0.16</td>
<td>19.49</td>
<td>0.82</td>
<td>0.73</td>
<td>0.43</td>
<td>0.55</td>
</tr>
<tr>
<td>Sunyocho Elementary</td>
<td>N</td>
<td>B</td>
<td>2.4 to 2.2</td>
<td>2.80</td>
<td>0.13</td>
<td>1.00</td>
<td>0.43</td>
<td>0.14</td>
<td>0.14</td>
<td>17.35</td>
<td>4.20</td>
<td>0.66</td>
<td>0.31</td>
<td>0.54</td>
</tr>
<tr>
<td>Nagatake Park</td>
<td>N</td>
<td>B</td>
<td>1.1 to 1.0</td>
<td>1.45</td>
<td>0.12</td>
<td>1.00</td>
<td>0.49</td>
<td>0.16</td>
<td>0.16</td>
<td>14.51</td>
<td>4.31</td>
<td>1.05</td>
<td>0.48</td>
<td>0.51</td>
</tr>
</tbody>
</table>

Fig 2.6 Map Showing CPT Field Case History Locations from the 1995 Hyogoken-Nambu Earthquake, in relation to the SPT Case History Locations and PGA Contours from Cetin et al. (2000).
1999 Chi-Chi Earthquake (Taiwan)

At 1:47am on the morning of September 21, 1999, the largest earthquake of Taiwan’s recent history hit central Taiwan at 7.3 magnitude causing thousands of deaths, building collapses, destruction to bridges, highways, dams, and railways. Schools were closed, power was cut, and people were evacuated.

The Chi-Chi Earthquake caused widespread damage to all areas of Taiwan, particularly in Yuanlin, Nantou, Wufeng, Chang-Bin, as well as the Port of Taichung. Most of the damages due to the earthquake were from ground failure mechanisms such as liquefaction and landslides.

Ku, Chih-Sheng, Der-Her Lee, and Jian-Hong Wu (2003) showed that two hundred and seventy five cone penetration test data were collected from the liquefaction-affected areas, and 46 liquefaction case histories and 88 non-liquefaction case histories were derived that can be used to evaluate the accuracy of existing liquefaction evaluation models. In Figure 2.7 shows the conventional plot of CSR$_{7.5}$ versus $q_{cIN}$ for the 134 cases. It showed that almost all liquefied cases have a $q_{cIN}$ value of less than 50, which might be result of disturbance due to liquefaction. To develop the “unbiased” boundary curve, the $q_{cIN}$ values of most if not all liquefied cases must be increased proportionally. However, no guidance is available for such correction in the liquefied cases. An alternative is to give more “weight” to the non-liquefied cases. In other words, the boundary curve is drawn not to encompass the liquefied data points, but rather to exclude all non-liquefied cases. Taking this approach, a boundary curve is obtained and shown in Figure 2.7. Symbolically,

$$CRR_{7.5} = 0.058 \exp [0.02 \ q_{cIN}]$$
2011 Tohoku Earthquake

A devastating earthquake of moment magnitude $M_w$ 9.0 struck the Tohoku and Kanto regions of Japan on 11$^{th}$ March, 2011 at 2:46 pm. The earthquake caused great economic loss, loss of lives and tremendous damage to structures and infrastructures. This was the largest earthquake ever recorded in Japan and one of the five most powerful earthquakes in the world since modern record keeping began in 1900.

Widespread liquefaction was observed in certain areas with plenty of evidence of sand boils. Fig.2.8 shows photographs of sand boils on the morning of 12th March. While Fig 2.5(left) shows the sand boils in the Takasu Park, Fig. 2.8(right) shows the liquefaction in the paved car
park of the Disneyland amusement park. The liquefied and ejected soil consisted of different types of materials ranging from pure brown sand with small fines content to grey silty sand, and also in some locations dredged recycled material.

Fig 2.8 Observed liquefaction in the park (left) and parking area of Disneyland (right). (S. Bhattacharya, et, 2011)

For Fig 2.9, a simplified stress-based liquefaction triggering analysis was carried out by considering a typical soil profile in Urayasu, where widespread liquefaction and sand boils were observed. The calculated probability of liquefaction over the depth up to 20 m is presented in the third sub-figure in Fig. 2.9. A probabilistic model of liquefaction, developed by Cetin et al., based on the database of field performance case histories. However, the probabilistic model of liquefaction has not been validated/calibrated against a mega thrust $M_w$ 9.0 subduction event. Bhattacharya, et al (2011) indicated that soil liquefaction is highly likely to trigger at shallow depths (up to 6 m) and medium depth (12-16m), and the effects of the moment magnitude are not so significant at these depths in the Figure 2.9. The main reason for the high liquefaction potential for this profile is the low blow count, $N$ (less than 5).
2011 Christchurch Earthquake New Zealand

The 2011 Christchurch (Canterbury) Earthquake was a magnitude 6.3 event that occurred at 12:15 p.m. local time on 22 February on the South Island of New Zealand. This was a shallow earthquake and its epicenter distance to the east side of Christchurch is less than 6 kilometers. There was extensive liquefaction and ground failure, which led to widespread loss of water and electricity. The earthquake produced strong ground motions within the central business district (CBD) of Christchurch.

Much of the Christchurch region is situated on unstable liquefiable sandy soil. The poor soil condition caused damage during the 2011 earthquake (see Figure 2.10). As shown in the figure, liquefaction (sand boils at the surface) was evident in many locations. The ground failure was extensive and affected residential buildings as well as a major roadway. For example, ground failure damaged roads in this area, rendering many of them inoperable. Figure 2.10 also
shows damage to a bridge abutment. The abutment cracked as the soil in the approach slab failed. Christchurch has several rivers, including the Avon and Heathcote Rivers which contributed to the underlying soil problems. Also exacerbating the problem was the fact that many water mains ruptured, pouring water onto the underlying soil and resulting in damage.

The resulting liquefaction documentation map is shown in Figure 2.11. Three areas of different liquefaction severity are indicated: A) moderate to severe liquefaction (red zone), B) low to moderate liquefaction (yellow zone), and C) liquefaction predominantly on roads with some on properties (pink zone).

Preliminary geotechnical zoning based on existing data indicates several different areas within the CBD that are dominated either by gravelly layers, thick liquefiable sands or sandy silt mixtures, and peat in the top 8–10 m of the deposits. The soil profiles and thicknesses of these layers are highly variable even within a single zone, thus imposing difficult foundation conditions and sometimes resulting in unconventional or hybrid types of foundations being adopted for buildings. The gravelly soils, even though relatively more competent foundation soils, typically show medium standard penetration test (SPT) N values of about 15 to 25 blow counts, whereas the liquefiable loose sands and silt-sand mixtures have low resistance of less than N = 12 or cone penetration test (CPT) qc values less than 3–6 MPa.

Major earthquake case histories are shown in Table 2.2. It showed that many earthquakes caused ground failure and damaged engineering structures.
Figure 2.10. Ground Failures and Resulting Water Damage. (Christchurch, New Zealand Earthquake Field Report, 2011)

Figure 2.11. Liquefaction documentation map of eastern Christchurch from drive-through Reconnaissance. (Christchurch, New Zealand Earthquake Field Report, 2011)
Table 2.2 Major earthquake case histories

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Remarks</th>
<th>Post-earthquake developments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1908 Reggio Messina earthquake (Italy)</td>
<td>120,000 fatalities. A committee of nine practicing engineers and five professors were appointed by Italian government to study the failures and to set design guidelines.</td>
<td>Base shear equation evolved i.e. the lateral force exerted on the structure is some percentage of the dead weight of the structure. (Typically 5-15%)</td>
</tr>
<tr>
<td>1923 Kanto earthquake (Japan)</td>
<td>Destruction of bridges, buildings. Foundations settled, titled and moved.</td>
<td>Seismic coefficient method (equivalent static force method using a seismic coefficient of 0.1-0.3) was first incorporated in design of highway bridges in Japan (MI 1927)</td>
</tr>
<tr>
<td>1933 Long Beach earthquake (USA)</td>
<td>Destruction of building specially school buildings</td>
<td>UBC (1927) revised. This is the first earthquake for which acceleration records were obtained from the recently developed strong motion accelerograph.</td>
</tr>
<tr>
<td>1964 Niigata earthquake (Japan)</td>
<td>Soil can also be a major contributor of damage.</td>
<td>Soil liquefaction studies started.</td>
</tr>
<tr>
<td>1971 San Fernando earthquake (USA)</td>
<td>Bridges collapsed, dams failed causing flood. More soil effects were observed.</td>
<td>Liquefaction studies intensified. Bridge retrofit studies started.</td>
</tr>
<tr>
<td>1976 Tangshan earthquake (China)</td>
<td>Most of the liquefaction occurred in the upper few meters of loose to medium-dense silty fine sand or fin-to medium-clean sand</td>
<td>More research on soil liquefaction. Using SPT to investigate soil liquefaction.</td>
</tr>
<tr>
<td>1994 Northridge earthquake (USA)</td>
<td>Steel connections failed in bridges</td>
<td>Importance of ductility in construction realized. Significant damage potential due to near-fault motions was recognized.</td>
</tr>
<tr>
<td>Year</td>
<td>Event</td>
<td>Impact and Outcomes</td>
</tr>
<tr>
<td>------</td>
<td>------------------------</td>
<td>------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>1995</td>
<td>1995 Kobe earthquake</td>
<td>Massive foundation failure. Soil effect was the main cause of failure.</td>
</tr>
<tr>
<td></td>
<td>(Japan)</td>
<td></td>
</tr>
<tr>
<td>1999</td>
<td>1999 Chi-Chi earthquake</td>
<td>Many bridges collapsed as they were located close to the faults</td>
</tr>
<tr>
<td></td>
<td>(Taiwan)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Turkey)</td>
<td>However, buildings conforming with the design codes performed well.</td>
</tr>
<tr>
<td>2001</td>
<td>2001 Bhuj earthquake</td>
<td>Large-scale destruction. Good performance of some new jetties of the Kandla port. Tilting of the Kandla Tower building without any damage.</td>
</tr>
<tr>
<td></td>
<td>(India)</td>
<td></td>
</tr>
<tr>
<td>2004</td>
<td>2004 Sumatra earthquake</td>
<td>Destruction to build environment due to earthquake and giant tsunami waves.</td>
</tr>
<tr>
<td></td>
<td>and tsunami</td>
<td></td>
</tr>
<tr>
<td>2011</td>
<td>2011 Tohoku earthquake</td>
<td>The most powerful earthquake ever recorded to have hit Japan, and the fourth most powerful earthquake in the world. The tsunami caused nuclear accidents.</td>
</tr>
</tbody>
</table>
Field Methodologies for Liquefaction Potential Evaluation

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 geologically and seismological similar 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.

**Standard Penetration Test (SPT)**

**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 sampler 1 ft (0.3m) into undisturbed soil by using the 140-lb weight falling 30 in. (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 2.12) and silty sand (Figure 2.13) 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.

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 have a higher 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 2.12 and 2.13 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)]. (Seed et al., 1985; Ishihara and Kosecki, 1996).

However Hsing-Cheng Liu (1992) showed that at a small fine content the increase in fines content leads to the decrease in liquefaction resistance until the fines content reaches a critical value where liquefaction resistance is the lowest. Beyond the critical fines content, the liquefaction resistance start to increase with increasing fines content in laboratory testing.

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”.
\[ F = \begin{cases} 
1.0 & \text{PI} \leq 10 \\
1.0 + 0.022 (\text{PI} - 10) & \text{PI} > 10 
\end{cases} \]

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 2.3.
Figure 2.12 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)
Figure 2.13 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).)
Table 2.3 Magnitude correction factors for cyclic stress approach (Seed et al, 1975)

<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 blow counts, 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. 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 shown in Table 2.4.

Table 2.4 Relationship between (N<sub>1</sub>)<sub>60</sub> and potential damage. (Seed et al, 1985)

<table>
<thead>
<tr>
<th>(N&lt;sub&gt;1&lt;/sub&gt;)&lt;sub&gt;60&lt;/sub&gt;</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 2.13 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 2.13 to determine the cyclic resistance ratio of the in situ soil, as follows: 1. Standard penetration test (N<sub>1</sub>)<sub>60</sub> value: note in Figure 2.13 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.

The full, updated database is shown with the Idriss-Boulanger triggering correlation in terms of equivalent CSR\(_{M=7.5,\sigma'=1}\) versus equivalent clean sand \((N_1)_{60cs}\) in Figure 2.14. For comparison, the database previously used by Idriss and Boulanger (2004, 2008) is shown in Figure 2.15. Comparison of Figure 2.14 with Figure 2.15 indicates that the updated database includes considerably more case histories and that the updated database continues to support the previously derived triggering correlation. Figure 2.16 showed Relationship between cyclic stress ratios causing liquefaction and \((N_1)_{60}\) values for clean sand with different plasticity index and fine content.

The revised relation for FC\(\leq 5\%\) is further compared to other published relations in Figure 2.17, including relations from early in their development (i.e., Seed 1979) to a very recent relation by Cetin et al (2000) that is summarized in Seed et al (2001).
Figure 2.14 Updated SPT case history database of liquefaction in cohesionless soils with various fines contents in terms of equivalent CSR for $M = 7.5$ and $\sigma'_v = 1$ atm and equivalent clean sand $(N_1)_{60cs}$ (Idriss-Boulanger, 2008).
Figure 2.15. SPT case history database used previously by Idriss and Boulanger (2004, 2008)
Figure 2.16 Parts (a) and (b) of Figure 2.6 in Cetin et al. (2004); note that the points representing case histories are identical in parts (a) and (b) of this figure and as representing conditions with $\sigma'_v = 1$ atm.
Fig 2.17 Curves relating CSR to \((N_1)_{60}\) published over the past 24 years for clean sands and the recommended curve for \(M = 7\frac{1}{2}\) and \(\sigma'_{vo} = 1\) atm (Seed et al 2001).
Cone Penetration Test (CPT)

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 lithological 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 lithological 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 2.18 and Figure 2.19), 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 (2000) 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 2.20 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, 2000). These normalized SPT data may then be
compared to laboratory cyclic strength test data or the field performance data base for various soils.

Figure 2.18 Manufacturing and operating tolerances of cones, taken from ASTM D5778.
Figure 2.19 Schematic section through a piezocone head, showing the piezo-element and friction sleeve (from ASTM D5778).
Figure 2.20 CPT prediction of overburden pressure-corrected SPT blow-count (Olsen and Malone, 2000)
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

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

where \( \sigma_v' \) 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 2.3. Kayen et al. (1992) found that liquefaction observations in the 1989 Loma 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 \((q_{c1N})_{cs} < 50\), \( \text{CRR} = 0.833 \left[ \frac{(q_{c1N})_{cs}}{1000} \right] + 0.05 \)

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

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):

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

\[ \text{CRR}_a = \text{CRR} \times K_{a2} \]

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).

I.M. Idriss and R.W. Boulanger (2004) showed that the CRR – q_{C1N} relation is shown in Figure 2.21 with the case history points for cohesionless soils having FC≤5%. The derived relation can be conveniently expressed as:
This CRR – $q_{c1N}$ relation is compared in Figure 2.22 to those by Shibata and Teparaksa (1988), Robertson and Wride (1997), Suzuki et al (1997), and the 5% probability curve by Moss (2003) as summarized in Seed et al 2003.

In liquefaction charts such as Fig. 2.22, every dot represents a saturated sand site which was subjected to a specific earthquake event, and the chart is essentially a way to organize the information available for those sites and their responses during actual earthquakes, which maximizes the chances of predicting if a similar site will or will not liquefy during a future earthquake of the same magnitude (M=7.5). That is, while the way of defining the cyclic stress ratio in Fig. 2.22 and in similar charts was initially suggested by laboratory cyclic test results, both the development and use of a chart like this have their own rules which are unrelated to any cyclic laboratory test. The method relies exclusively on the field measurement of $q_{c1N}$ to characterize the liquefaction resistance of the sand.

<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>
Fig. 2.21 CPT-based case histories and recommended relation for clean sands for $M = 7\frac{1}{2}$ and $\sigma'_v = 1$ atm (I.M. Idriss and R.W. Boulanger, 2004)
Fig. 2.22 CPT-based case histories and recommended relation for clean sands with relations proposed by others.
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.

Seismic Cone Penetration Test

Seismic Cone Penetration Testing (SCPT) sounding provides a rapid and cost-effective method for directly measuring shear wave velocity of soils in situ. Shear wave velocity is used as an index of liquefaction resistance since both are influenced by many of the same factors.
The seismic cone (Figure 2.23) measures the shear wave velocity of the soil being investigated. Together with a knowledge of the soil saturated unit weight, the shear wave velocity allows an assessment of the small strain shear modulus ($G_0$) and the constrained modulus ($M_0$) to be made. The small strain shear modulus is an essential input for prediction of ground surface motions from earthquake excitation, evaluation of foundations for vibrating equipment, offshore structures behavior during wave loading, and for prediction of deformations around excavations. The results of the SCPT carried out on Nunavik, Canada are given in Figure 2.25.

The seismic cone is available in 10 and 15 cm$^2$ areas. The cone usually consists of a piezocone unit – measuring the geotechnical parameters $q_c$, $f_s$ and $U_2$ – with a receiver for the seismic measurements above it. A schematic diagram, with the layout of the standard technique using a seismic cone, is shown in Figure 2.24. The extra equipment needed, in addition to the built in seismometer, is a memory oscilloscope and an impulse source with a trigger for the oscilloscope. The source can consist of a steel beam for shear (S) wave generation or a flat plate for compression (P) wave generation.

The moduli $G_0$ and $M_0$ can be determined from the following:

$G_0 = \rho(V_s)^2$ (kN/m$^2$)

$M_0 = \rho(V_p)^2$ (kN/m$^2$)

where: $\rho$ = the soil mass density (kg/m$^3$)

$V_s$ = shear wave velocity (m/sec)

$V_p$ = compression wave velocity (m/sec)
SCPT shear wave velocity measurements are used in these evaluations: 1) Liquefaction Risk; 2) Earthquake generated ground-surface movements; 3) Foundations for vibrating equipment; 4) Behavior of offshore structures due to wave loading.

Figure 2.23. Seismic Cone (from A.P. van den berg)

Figure 2.24. Seismic cone configuration (from A.P. van den berg)
Figure 2.25. Results of the SCPT carried out on June 16th 2001 in a permafrost mound near Umiujaq, Nunavik, Canada.

Other Techniques (Appendix A)
Laboratory Methods for Liquefaction Resistance Evaluation

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 (2004) 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

**Laboratory Test**

**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 2.26 and 2.27 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 2.28. Instructions for performance of cyclic triaxial tests may be found in Engineer Manual 1110-2-1906 (Department of the Army, 1990).

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.

Figure 2.26 Schematic of cyclic triaxial test equipment (Marcuson and Krinitzsky, 1976)
Figure 2.27 Typical analog recordings of load, deformation, and pore pressures during a cyclic triaxial test (Department of the Army, 1990)
Figure 2.28 Cyclic triaxial strength curves for Monterey No.0 sand (Department of the Army, 1990)

Cyclic Hollow Cylinder Tests

A hollow cylindrical test device (HCTD) is an extremely valuable tool for studying constitutive behavior 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.

The University of Colorado at Denver Hollow Cylinder Torsional /Axial test cell was designed and fabricated by Dr. Jing-Wen Chen while conducting his doctoral research at UC Denver in 1988. In the hollow cylinder test at UC Denver, 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 (Jing-Wen Chen, 1988).

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 apparatus (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, τ_θz and τ_zθ (τ_θz = τ_zθ) in vertical and horizontal planes, the axial load, W, contributes to a vertical stress, σ_z. P_i and P_o establish a gradient of radial stress, σ_r, across the cylinder wall. The relationship between radial stress, σ_r, and the circumferential stress, σ_θ, is expressed by the equilibrium equation:

\[ σ_θ = σ_r + r \left( \frac{dσ_r}{dr} \right) \]

where r is the radial distance to a point in the hollow cylinder, and dσ_r and dσ_θ are the radial and circumferential stress increments respectively. The stress condition in an element of a hollow cylinder specimen is shown in Fig. 2-29. Both inner and outer pressure are applied on the
membrane so that there is no shear stress on the vertical boundaries, \( \sigma_r \) is always a principal stress because there are no shear stresses on circumferential surface throughout the wall.

**Figure 2.29** 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)

The state of stress in a hollow cylinder test is defined with reference to cylindrical coordinates, in terms of the stress components.
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, $\sigma_z^\prime$, $\sigma_r^\prime$, $\sigma_\theta^\prime$, $\tau_{\theta z}^\prime$. Hight et al. (1983) used the following expressions:

Average vertical stress $\sigma_z^\prime = \left[ \frac{W}{\pi} \left( r_o^2 - r_i^2 \right) \right] + \left[ \frac{(P_o r_o^2 - P_i r_i^2)}{(r_o^2 - r_i^2)} \right]$ 

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

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

Average shear stress $\tau_{\theta z}^\prime = \frac{3M_T}{2\pi (r_o^3 - r_i^3)}$ 

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

$\sigma_1^\prime = \left[ \frac{(\sigma_z^\prime + \sigma_\theta^\prime)}{2} \right] + \sqrt{ \left[ \frac{(\sigma_r^\prime - \sigma_\theta^\prime)}{2} \right]^2 + (\tau_{\theta z}^\prime)^2}$

$\sigma_2^\prime = \sigma_r^\prime$ 

$\sigma_3^\prime = \left[ \frac{(\sigma_z^\prime + \sigma_\theta^\prime)}{2} \right] - \sqrt{ \left[ \frac{(\sigma_r^\prime - \sigma_\theta^\prime)}{2} \right]^2 + (\tau_{\theta z}^\prime)^2}$

By regarding the specimen as a single element, the state of strain is presented in cylindrical coordinates in terms of the following components:
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 \( \varepsilon_z' = \Delta H/H \)

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 \( \gamma_{\theta z}' = \frac{2 \theta (r_o^3-r_i^3)}{3H (r_o^2-r_i^2)} \)

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:

\[
\varepsilon_1 = \left[ (\varepsilon_z' + \varepsilon_\theta')/2 \right] + \sqrt{\left[ (\varepsilon_z' - \varepsilon_\theta')/2 \right]^2 + \left[ \gamma_{\theta z}'/2 \right]^2}
\]

\[
\varepsilon_2 = \varepsilon_r'
\]

\[
\varepsilon_3 = \left[ (\varepsilon_z' + \varepsilon_\theta')/2 \right] - \sqrt{\left[ (\varepsilon_z' - \varepsilon_\theta')/2 \right]^2 + \left[ \gamma_{\theta z}'/2 \right]^2}
\]

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

\[ \tan 2 \alpha = 2 \frac{\tau_{0z}}{(\sigma_r' - \sigma_0')} \]

\( b \) 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 \( \sigma_r' = \frac{P_o r_o + P_i r_i}{r_o + r_i} \), \( \sigma_2 \) 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

\[ \tan 2 \alpha_{dc} = \frac{d \gamma_{0z}'}{(d \varepsilon_z - d \varepsilon_0)} \]

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

**Cyclic Simple Shear Test**

Cyclic simple shear tests were conducted on Drammen clay by Andersen et al. (1980) to investigate the effects of shear stress on clay through the overconsolidation ratio, one-way cyclic loading, shear stress amplitude which varied during the test, strain-controlled cyclic loading, horizontal shear stress during consolidation, drainage during and after undrained cyclic loading, and artificial cementation. In case of cyclic loading, the test can be used in evaluating the liquefaction potential of sands.

The cyclic simple shear test is capable of reproducing earthquake stress conditions.
much more accurately than the cyclic triaxial test (Figure 2.30). It is more commonly used for liquefaction testing. In this test, a short cylindrical specimen is restrained against lateral expansion by rigid boundary platens, a wire reinforced membrane or with a series of stacked rings. By applying cyclic horizontal shear stresses to the top or bottom of the specimen, the test sample is deformed in the same way as an element of soil subjected to vertically propagating S waves.

Fig 2.30 Cyclic Simple Shear Test Device

Peacock and Seed (1968) used cyclic simple shear tests to study liquefaction problems. Subsequently, Seed and Peacock (1971), Finn (1971, 1985), Pickering (1973), Martin et al. (1975) performed considerable cyclic simple shear tests to study liquefaction problem. In recent years, simple shear devices that allow independent control of vertical and horizontal stresses have been developed.
**Shaking Table Test**

The other type of laboratory test is model test. Model test usually attempt to reproduce the boundary condition and material property in the field by a small-scale physical model. It may be used to evaluate the performance of a prototype or verify predictive theories. Model test is a useful method for studying dynamic behavior of earth structure and foundation.

Shake table tests of many sizes are being used for liquefaction studies on saturated soil samples prepared in a container, fixed to a shaking platform and vibrated at the desired frequency for a prescribed time. A surcharge is placed on the sample to provide the confining pressure. The measurements of acceleration pore water pressure and settlements are made during the test. Shaking tables utilize a single horizontal translation degree of freedom, but shake table with multiple degrees of freedom have also been developed. Kokusho (1987) developed a numerical model based on shake table test.

**Centrifuge Test**

A geotechnical centrifuge is used to accurately conduct model tests in studying geotechnical problems such as strength, stiffness and capacity of foundations for bridges and buildings. It makes use of centrifugal acceleration to match soil stresses in a 1/50 scale model. So, for a model container 1 m deep filled with soil, subjected to a centrifugal acceleration of 50 g, the pressures and stresses will be increased by that factor of 50. The purpose of the centrifuge machine is to shake the receptor in a controlled manner to simulate a dynamic event similar to an earthquake. But, most importantly, it is useful to study ground-shaking effects without risking public safety.

The basic principle of centrifuge modeling is to recreate the stress conditions which would exist in a full scale construction (prototype), using a model on a greatly reduced scale.
This is done by subjecting the model components to an enhanced body force, which is provided by a centripetal acceleration of magnitude “ng”, where g is the acceleration due to the Earth Gravity (i.e. 9.81 m/s\(^2\)). Stress replication in an nth scale model is achieved when the imposed “gravitational” acceleration is equal to “ng”. Thus, a centrifuge is suitable for modeling stress-dependent problems. Moreover, reduction of time for model tests such as consolidation time can be achieved by using a reduced size model.

**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:

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

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.

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 \(a_{\max}\) 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:

\[
F = ma = \left(\frac{W}{g}\right) a = \left(\gamma_t z\right) a_{\text{max}} = \sigma_{vo} \left(\frac{a_{\text{max}}}{g}\right)
\]

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 \( W/g \).

\( W = \) 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 \( \gamma_t z \)

\( \gamma_t = \) total unit weight of soil, lb/ft\(^3\) or kN/m\(^3\)

\( z = \) depth below ground surface of soil column

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

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

\( \sigma_{vo} = \) total vertical stress at bottom of soil column, lb/ft\(^2\) 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 \( \tau_{\text{max}}, \)

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

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

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

Seed and Idriss (1971) incorporated a depth reduction factor \( \gamma_d \), showed in Figure 2-31, into the above equation: \( \left(\frac{\tau_{\text{max}}}{\sigma_{vo}'}\right) = \gamma_d \left(\frac{\sigma_{vo}}{\sigma_{vo}'}\right)\left(\frac{a_{\text{max}}}{g}\right) \)
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:

\[ \tau_{\text{cyc}} = 0.65 \tau_{\text{max}} \]

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

\[ \text{CSR} = \left( \frac{\tau_{\text{max}}}{\sigma_{\text{vo}'}} \right) = 0.65 \gamma_d \left( \frac{\sigma_{\text{vo}}}{\sigma_{\text{vo}'}} \right) \left( \frac{a_{\text{max}}}{g} \right) \]

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

**Figure 2.31** Reduction factor to estimate the variation of cyclic shear stress with depth below level or gently sloping ground surfaces. (After Seed and Idriss, 1971)
CHAPTER III

LITERATURE REVIEW II: FACTOR LIQUEFACTION RESISTANCE OF SOILS

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, $K_o$, over consolidation ratio, depth to water table, and effective confining pressure; and (3) earthquake characteristics, specifically ground shaking intensity and duration.

Effects of Soil Factors

Soil Properties

In term of the soil types most susceptible to liquefaction, Ishihara (1985) stated: “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 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:

1. 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).
2. The soil must have a liquid limit (LL) that is less than 35 (that is, LL < 35).
3. The water content ω of the soil must be greater than 0.9 of the liquid limit [that is, ω > 0.9 (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.

Polito (2001) found that soils with LL<25 and PI<7 are liquefiable, soils with 25<LL<35 and 7<PI<10 are potentially liquefiable, and soils with 35<LL<50 and 10<PI<15 are susceptible to cyclic mobility. Figure 3.1 showed Wang’s data indicated plasticity index and liquid limit for soil liquefaction in Chinese Criteria. Figure 3.2 showed J.D. Bray & R.B. Sancio (2006)’s study in liquid limit and plasticity index.
I.M. Idriss and R.W. Boulanger (2004) showed that fine-grained soils transition from sand-like to clay-like behavior at plasticity indices (PI) between about 3 and 8, with the transition point appearing to be slightly lower for ML-CL soils than for ML soils. For practical purposes, it is recommended that fine grained soils be categorized as sand-like (i.e., susceptible to liquefaction) if they have a PI < 7 and clay-like (i.e., susceptible to cyclic failure, not
liquefaction) if they have a PI ≥ 7. This criterion may be adjusted on a site-specific basis if justified by the results of detailed in situ and laboratory testing.

**Fine Content and Plasticity Index**

In Dr. Hsing-Cheng Liu’s dissertation (1992), it found that inclusion of fines in a clean sand at a constant overall void ratio does not necessarily cause increase in liquefaction resistance, and also at a constant overall void ratio, the increase in fines content at a constant plasticity index in the clean sand cause a decrease in liquefaction resistance, beyond 30%, the further increase in fines content results increase in liquefaction resistance.

Nien-Yin Chang (1990) run triaxial tests with samples tested under 15 psi confining pressure in the medium-sand with 5% fines and plasticity indices ranging from 0 to 40. The triaxial tests results showed that the trend of increasing liquefaction resistance with increasing plasticity index is more obvious, and also the effect of fine contents is very much more significant than the effect of plasticity indices.

I.M.Idriss and R.W.Boulanger (2004) showed that cyclic stress ratio (CSR) increased with increased fine content percent under the same modified standard penetration blows count. The cases for cohesionless soils with FC ≥ 35% are plotted in Figure 3.3. Figure 3.4 shows the case history points for cohesionless soils with 5 %< FC <15%, while Figure 3.5 shows the cases for 15% ≤ FC < 35%.

However, Chang (1990) showed the clean medium sand has the strongest liquefaction resistance and as the fine content increases, the liquefaction resistance decreases until the fine content reaches approximately 26%, as indicated by the liquefaction potential curves shifting toward the left in Figure 3.6. Then the trend reverses itself and soils begin to gain strength as the
fine content further increases, as indicated by the curves shifting toward the right. In Figure 3.6 shows, for the medium-sand test series, exactly the similar trend of decreasing resistance and then increasing resistance with the increasing fine content. Each curve in Figure 3.6 gives the stress ratios required to achieve initial liquefaction in ten cycles of loading for soils with the same plasticity index for each curve. The collection of curves also indicates the existence of a critical fine content, below which, the resistance decreases, and, above which, the resistance increases with increasing fine content, and the critical fine content decreases with increasing plasticity of soils. Figure 3.6, 3.7, and 3.8 seem to indicate that the critical fine content increases as the number of cycles required reaching initial liquefaction increases.

Tzuo-Shin Ueng, Chia-Wen Sun and Chieh-When Chen (2013) showed that as the fine content increases, cyclic resistance ratio decreases until the fine content approximately 20% in the Figure 3.9. And also indicated that the effect of fines on the liquefaction resistance of a soil is more prominent using (FC)_{400} (passing the No.400 sieve) than (FC)_{200} (passing the No. 200 sieve), probably due partly to the different in plasticity of the fines.

Yong Wang and Yanli Wang (2010) showed that with a constant dry density, the liquefaction resistance first decrease and then increase with the increase of the fines content, when the relative density reaches the minimum value at the fines content of 30% and the liquefaction resistance also reaches the minimum value at the same fines content in Figure 3.10.

Mehmet Murat Monkul and Jerry A. Yamamuro (2011) showed that relative density alone cannot be a consistent comparison basis for the influence of fines content on liquefaction potential of sand in Figure 3.11, and also the D_{50-sand}/d_{50-silt} ratio becomes larger (for SiCoSil and Potsdam fines), the void ratio decreases consistently with increasing fines content, show as Figure 3.12.
Chang (1990) showed that the medium sand with a small fine content of 5% reflects in the irregular relationship between the stress ratio required to reach initial liquefaction versus plasticity index, and also comparison between the effect of fine contents and the effect of plasticity indices of fines on the liquefaction resistance of soils indicated that the effect of fine contents is very much more significant than the effect of plasticity index.

Fig. 3.3 SPT case histories of cohesionless soils with FC ≥ 35% and the NCEER Workshop (1997) curve and the recommended curves for both clean sand and for FC = 35% for M = 7½ and σ'vo = 1 atm (I.M.Idriss and R.W.Boulanger, 2004)
Fig. 3.4 SPT case histories of cohesionless soils with 5% <FC< 15% and the recommended curves for both clean sand and for FC = 15% for M = 7½ and $\sigma'$ = 1 atm (I.M.Idriss and R.W.Boulanger, 2004)
Fig. 3.5 SPT case histories of cohesionless soils with 15% < FC < 35% and the NCEER Workshop (1997) curve and the recommended curves for both clean sand and for FC = 15% for M = 7½ and $\sigma'_{vo} = 1$ atm (I.M. Idriss and R.W. Boulanger, 2004)
Figure 3.6 Stress Ratio Required to reach Liquefaction in 10 Cycles versus Fine Content Under Confining pressure 15 psi (N.Y. Chang, 1990)

Figure 3.7 Stress Ratio Required to reach Liquefaction in 30 Cycles versus Fine Content Under Confining pressure 15 psi (N.Y. Chang, 1990)
**Figure 3.8** Stress Ratio Required to reach Liquefaction in 100 Cycles versus Fine Content Under Confining pressure 15 psi (N.Y.Chang, 1990)

**Figure 3.9** Cyclic resistance ratio versus equivalent (FC)$_{400}$ (Tzuo-Shin Ueng, Chia-Wen Sun and Chieh-When Chen, 2013)
Fig. 3.10 Variation in Liquefaction Resistance with Fines Content for $N_l=20$ (Yong Wang and Yanli Wang, 2010)

Fig. 3.11 Change of relative density and liquefaction potential with different fines contents and Silts for tested specimens. (Mehmet Murat Monkul and Jerry A. Yamamuro, 2011)
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.
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) stated that field evidence indicates that most liquefaction failures have involved uniformly graded granular soils.

Chang (1990) showed that the soil fines controlled the cyclic triaxial strength when fines comprised more than about 40% of each gap-graded mixture, and also well-graded mixtures exhibited higher cyclic triaxial strength at high fines contents than were measured in their clean parent sand in cyclic triaxial test.

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.

Geological 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.

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 alluvial river fans (Chang, 1990). 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. Youd and Hoose (1977) reported that liquefaction most commonly occurs in Holocene deposits far less often in Pleistocene soils, and very rarely in Pre-Pleistocene deposits.

Based on a series of minicone cyclic triaxial tests were performed on “aged” and “unaged” samples, to simulate geologic aging effects, a small amount of cement was mixed in with the soil during sample preparation. From “unaged samples” without cement, Takaji Kokusho, Fumiki Ito, Yohta Nagao and A. Russell Green found that although both liquefaction resistance ($R_L$) and penetration resistance ($q_t$) decrease as fine content ($F_c$) increases, a unique relationship exists between $R_L$ and $q_t$ that is independent of $F_c$. However, from samples having the same cement content $C_c/F_c$ (i.e., simulating the same geologic age), it was found that $R_L$ increases as $F_c$ increases for the same $q_t$. This trend is consistent with field-based $R_L$-$q_t$ correlations for natural soil deposits to which aging effects are intrinsic. Thus, it has been clarified that it is not the $F_c$-value by itself but rather the cementation effect associated with higher $F_c$ that results in a higher liquefaction resistance for a given $q_t$. 
Effects of Laboratory Factors

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.

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.

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 3.13. In Figure 3.13, 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 waveforms; (1) a sinusoidal 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. 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 changed 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.
**Figure 3.13** Effect of loading wave form on cycles to initial liquefaction for moist-tamped Specimen. (Silver, et al, 2000)

**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).

**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.

**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).

**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.

When pressure during consolidation is applied to a sample through a rubber membrane, the membrane deforms and is pushed into the pore spaces between the grains. This results in expulsion of some pore water from the sample, without a change in void ratio of the sample. Thus the measured volume change during consolidation must be corrected for membrane penetration when void ratio is calculated.
There are a number of theoretical studies (Molenkamp and Luger, 1981; Baldi and Nova, 1984; Kramer et al., 1990) summarized by Ali et al., (1995) suggesting the form of the equation for membrane penetration. For practical purposes, membrane penetration can be quantified in terms of a normalized membrane penetration:

$$\varepsilon_m = \frac{\Delta V_m}{A_s} \log \left( \frac{p_1'}{p_2'} \right)$$

where $\varepsilon_m$ = normalized membrane penetration

$\Delta V_m$ = volume change due to membrane penetration

$A_s$ = sample area covered by the membrane (2$\pi$rh for a cylindrical sample)

$p_1'$, $p_2'$ = net pressure acting across the membrane before and after the volume change

For sands, $\varepsilon_m$ is primarily dependent on grain size, assuming other factors such as membrane thickness and modulus are content.

**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 $D_r = 70\%$ (Muilis, 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.

**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.

Y.P Vaid, J.D. Stedman and S.Sivathayalan (2001) showed that in cyclic loading the effect of increasing confining stress at a given static shear generally decreased the resistance to liquefaction. However, at the loosest states the increase in confining stress had little effect.

Cyclic Stress Amplitude and Number of Cyclic Stress Cycles

In their laboratory study, Seed and Lee (1966) concluded that the larger the stress or stain, 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, Lee, 1997).

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 0.1 mm 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.

**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.

**Lateral Earth Pressure ($K_0$) and Over consolidation Ratio**

A Study on 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 $K_0$ 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.

**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.
Effects of Field Factors

There are many factors that 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:

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 ground water 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.
**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.

**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.

**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.

**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 over consolidation radio (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 over consolidation 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.

**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.

**Effects of Earthquake Factors**

**Magnitude Scaling**

The magnitude scaling factor (MSF) is used to account for duration effects (i.e., number and relative amplitudes of loading cycles) on the triggering of liquefaction. The MSF relationships used by Idriss and Boulanger (2008) and revised herein were derived by combining (1) laboratory-based relationships between the CRR and the number of equivalent uniform loading cycles, and (2) correlations of the number of equivalent uniform loading cycles with earthquake magnitude. The MSF for sands used by Idriss and Boulanger (2008) was developed by Idriss (1999), who derived the following relationship:

$$\text{MSF} = 6.9 \exp \left( -\frac{M}{4} \right) - 0.058 \leq 1.8$$
This relationship is plotted in Figure 3.14. An upper limit for the MSF is assigned to very-small-magnitude earthquakes for which a single peak stress can dominate the entire time series. The value of 1.8 is obtained by considering the time series of stress induced by a small magnitude earthquake to be dominated by single pulse of stress (i.e., $\frac{1}{2}$ to 1 full cycle, depending on its symmetry), with all other stress cycles being sufficiently small to neglect.

![Figure 3.14](image)

**Figure 3.14.** Magnitude scaling factor (MSF) relationship by some researchers.

The MSF relationships used by Idriss and Boulanger (2008) for sands or clays, as shown in Figure 3.15, can be rewritten in a more general form as,

$$MSF = 1 + (\text{MSF}_\text{max} - 1) \left( \frac{\exp\left(\frac{-M}{4}\right)}{\exp\left(\frac{-7.5}{4}\right)} - \frac{\exp\left(\frac{-5.25}{4}\right)}{\exp\left(\frac{-7.5}{4}\right)} \right)$$

where $\text{MSF}_\text{max} = 1.8$ for sand and $\text{MSF}_\text{max} = 1.09$ for clay and plastic silt. With the fixed terms expressed numerically, the above equation becomes,

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The proposed relationships relate \( \text{MSF}_{\text{max}} \) to \( q_{c1Ncs} \) and \( (N_1)_{60cs} \) values as,

\[
\text{MSF}_{\text{max}} = 1.09 + \left( \frac{q_{c1Nc}}{180} \right)^3 \leq 2.2
\]

\[
\text{MSF}_{\text{max}} = 1.09 + \left( \frac{(N_1)_{60cs}}{180} \right)^3 \leq 2.2
\]

The resulting MSF relationships for different values of \( q_{c1Ncs} \) and \( (N_1)_{60cs} \) are shown in Figure 3.16. This relationship produces \( \text{MSF}_{\text{max}} = 1.8 \) at \( q_{c1Ncs} \approx 160 \) or \( (N_1)_{60cs} \approx 27 \), which matches the MSF relationship for sand by Idriss (1999), and \( \text{MSF}_{\text{max}} \approx 1.10 \) for \( q_{c1Ncs} < 60 \) \( (N_1)_{60cs} < 6 \), which is consistent with the expected results for very loose sands or soft low-plasticity silts.

![Figure 3.15](image-url)
Figure 3.16. Variation in the MSF relationship with $q_{c1Ncs}$ and with $(N1)_{60cs}$ for cohesionless soils (Boulanger and Idriss 2007)

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.
Ground Motions

Earthquake magnitude, distance from the hypocenter and local subsurface conditions are the three major factors that affect the seismic intensity at the site. The larger the magnitude or shorter the distance from the earthquake focus, the stronger is the seismic intensity at a given site. In addition, the level of shaking intensity in rock is generally different from that in a soil deposit at ground surface or at any depth below the ground surface. Other factors being equal, local subsurface conditions alone can both amplify and attenuate earthquake forces. During small earthquakes and microtremors, the ground surface accelerations on soil deposits, especially on soft compressible clay layers and alluvial deposits, are usually higher than those occurring on bedrock. However, as earthquake magnitudes become greater, the horizontal accelerations on soil sites may be equal to or lower than those on rock sites.

As noted by Ishihara (1985), when a soil layer is liquefied, the energy of the vertically propagating shear waves will, to a certain degree, be absorbed by the liquefied soil. As a result, the transmission of incoming acceleration to ground surface is depressed and the shaking intensity attenuated. He suggests using 0.2g as the maximum ground surface acceleration on top of a liquefied layer.

The maximum ground acceleration caused by an earthquake may be obtained from field measurements or numerical methods using the wave propagation theory (Idriss and Seed, 1968; Schnabel et al., 1972; Finn et al., 1976; Liou et al., 1977; Martin and Seed, 1978; Singh et al., 1981; Prevost, 1989). The most widely used numerical method to estimate the ground site response has been a quasi-linear total stress analysis using the computer program SHAKE developed by Schnabel et al. (1972), in which nonlinear stress-strain behavior of soil is accounted for by adjusting shear modulus and damping, iteratively, until the computed dynamic
shear strains converge. For design simplicity, Seed et al. (1976) established approximate relationships between maximum accelerations on rock and maximum ground accelerations for various subsurface conditions, as shown in Figure 3.17. It shows that maximum horizontal accelerations on rock and on stiff soil sites are almost identical over a wide range of accelerations. The curve that represents soft to medium clay and sand sites is believed to be unconservative for soft and medium clays in estimating the maximum ground accelerations and, therefore, should not be used for such soil deposits for any design purposes. Idriss (1990) modified this curve to Figure 3.18 based on the ground motion data obtained at soft clay sites in the 1985 Mexico City earthquake and the 1989 Loma Prieta earthquake and a number of site response analyses of soft clay deposits subjected to the 1964 Alaska acceleration time history. This figure indicates that bedrock accelerations beneath soft soil sites are generally amplified by the soft sediment below 0.4g and attenuated above 0.4g.
Figure 3.17 Approximate relationship between maximum accelerations on rock and maximum ground accelerations (Seed et al., 1976)
Figure 3.18 Amplification – attenuation relationship for modifying bedrock acceleration at soft soil sites (Idriss, 1990)
Table 3.1 Estimated susceptibility deposits to liquefaction during strong seismic shaking based on geologic age and depositional environment. (Some researchers)

<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></td>
<td>&lt;500 years</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>

(a) Continental deposits
Table 3.1 (con’t)

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

(b) Coastal zone

| Beach-large waves | Widespread | Moderate | Low       | Very low   | Very low |
| Beach-small waves | Widespread | High     | Moderate  | Low        | Very low |
| Delta            | Widespread | Very high| High      | Low        | Very low |
| Estuarine        | Locally variable | High     | Moderate  | Low        | Very low |
| Foreshore        | Locally variable | High     | Moderate  | Low        | Very low |
| Lagoonal         | Locally variable | High     | Moderate  | Low        | Very low |

(c) artificial

| Compacted fill   | Variable | Low       | Unknown   | Unknown   | Unknown |
| UnCompacted fill | Variable | Very high | Unknown   | Unknown   | Unknown |
CHAPTER IV

CYCLIC TRIAXIAL TEST PROGRAM and RESULTS ANALYSIS

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 that 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 that 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 effective stress 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 tests. To simulate field conditions, a test can be performed by keeping the axial stress constant, while decreasing the effective stress. 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.

Test Program

Triaxial Testing System

The triaxial cell was shown in Fig.4.1 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.4.2 shows the transducer and four-way valve disconnected from the triaxial cell.

On the right side of Fig.4.1, four pressure lines are seen. The upper horizontal line entering the base of the triaxial cell applied the consolidation 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.4.3 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 stand pips) shown in Fig.4.3 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.

Figure 4.1 Test Specimen Placed Within the Triaxial Cell. (Jungang Liu, 2019)
Figure 4.2 Pore pressure transducer and a 4-way valve used to monitor cell pressure, back pressure, and specimen pore water pressure. (Jungang Liu, 2019)

Figure 4.3 Pressure control panel used to apply cell and back pressure to the triaxial cell. Graduated burettes were used to determine specimen volume change. (Jungang Liu, 2019)
Test Equipment

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.4.4 was used to apply the cyclic dynamic loading to the sand specimens.

Figure 4.4 Closed loop electro-hydraulic materials test system applying a sinusoidal loading to Soil specimen. (Jungang Liu, 2019)

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 that was the case during tests that 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.

**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.

**Servo valve**

The hydraulic actuator is controlled by the opening and closing of the servo valve in response to a control signal from the valve driver or controller. The servo valve 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 servo valve 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 servo valve 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.

**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.

**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.
Soil Sample Preparation

Soil Samples

The soil sample is the mixture of a uniform medium Monterey No. 0/30 sand and Leyden clay (M-L) from Golden Colorado. Monterey No. 0/30 sand sample is a uniform clean sand from Monterey, California with a coefficient of uniformity, $C_u$, equals 1.6, a coefficient of curvature, $C_v$, 1.00, and a mean grain size, $D_{50}$, 0.45mm. The sand is classified as SP via the Unified Soil Classification System. The sand has maximum unit weight of 105.8 pcf (Ib/ft$^3$), minimum unit weight 91.7 pcf (Ib/ft$^3$), its friction angle 37° and specific gravity 2.65.

Leyden clay was sieved through a #200 sieve to remove any impurities. The specific gravity of clay is 2.67, Liquid limit of 42%, plastic limit of 22% and plastic index of 20%. The maximum dry unit weight of clay is 109 pcf (Ib/ft$^3$) with the optimum of 17% water content in the standard proctor compaction test.

Mixing Soil Samples

It is extremely time consuming to achieve a uniform sand-fine mixture, especially when a large amount of high plastic fine is involved. Silt have to pass #200 sieve. The following strict procedure is followed in mixing soils containing fines:

1) Determine the weight percentage of dry fines and clean sand.

2) Obtain the water content of each constituent, including silt and sand of required sizes, which were previously sieved and stored in covered containers.

3) Determine the dry weight of the soil required for preparing ten triaxial samples, usually around 20 pounds.

4) Calculate the dry weight of all constituents required to make 20 pounds of a dry soil mix.
5) Use the water contents obtained in (6) to calculate the required moist weight of all constituents.

6) Weight an exact amount of each soil constituent.

7) Mix these soil constituents in a large pan, until it reaches a satisfactory degree of uniformity. The time required to achieve a condition of a uniform mix increases with the amount of its plastic fines. Mixing is continued until no visual color variation can be detected. The effort could take as long as a couple of hours, when a large percent of plastic fine is involved.

8) Determine the amount of water to be added to result in a water content of 6-9%. A soil mix of a higher fine content tends to require a higher water content to reduce the tamping effort needed to achieve a required compacted density.

9) Weight the required amount of water and spray it intermittently onto the soil while mixing.

10) Sieve the soil mix through #8 sieve, whenever necessary, to screen out large moist soil crumbs, which are likely to form, when mixing and wetting a soil containing a large amount of fines. These large crumbs are broken down and remix with the rest of soil to achieve a uniform moisture distribution.

11) Continue (10) until the soil is uniformly mixed with water and no large soil crumbs larger than #8 sieve opening are present.

12) Transfer the moist soil from the mixing pan to a covered container.

13) Store the container with the moist soil in a moisture control room at least overnight to achieve a uniform moisture distribution before it is used to prepare a triaxial test sample.
Specific gravities were determined for the silt and uniform Monterey No. 0/30 sand. The specific gravity of silt is 2.67 and specific gravity of sand is 2.65. The specific gravity of soils containing fines was obtained by mixing calculated amounts of different percentage of fines and uniform Monterey No. 0/30 sand. As an example, the specific gravity of soils containing fines, twenty-five percentage of fines and seventy-five percentage of uniform Monterey No. 0/30 sand, was calculated as follows:

$$G_{SF} = \frac{1}{\left( \frac{25}{2.67 \times 100} + \frac{75}{2.65 \times 100} \right)} = 2.655$$

The weighted specific gravity for soil containing fines was also determined using the same procedure. The dry unit weight at 50% relative density for each parent sand was determined using following equation.

$$Dr=50\% = \frac{1}{\gamma_{dmin}} - \frac{1}{\gamma_{dmax}}$$

Void ratios corresponding to dry unit weights at 50% relative density, were determined using the equation:

$$\gamma_{d50\%} = \frac{Gs \times \gamma_w}{1+e50\%}$$

Sample Preparation

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 o-rings.
The rubber o-ring shown in Figure 4.5. 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 4.6. 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 4.7. 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 mix can be determined using a sample procedure outlined 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:

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

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:

\[
r_d = \frac{G_s \times w}{1 + e_d}
\]

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 moist tamping method is adopted for compacting moist soil, the required weight of moist soil, \( W_{wet} \), is calculated as: \( W_{wet} = W_d \times (1 + W) \) 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 4.8.

(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 4.9.

(11) Carefully place two o-rings in the 0-ging 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 4.10.

(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 4.11.

(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 4.12 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 4.13.

(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 4.14. 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 4.15 and the sample height to see if the applied vacuum causes 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 4.16. Average the four measurements as the final height with sample.

(31) Use a pi-tap shown in Figure 4.17 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: \(\frac{1}{4} \times \text{(top diameter + 2\times mid-height diameter + bottom diameter)}\)

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: 

\[
\gamma_d = \frac{(Gs \times \gamma_w)}{(1+e)}
\]
(32) Apply a vacuum of 5 psi to the top a container half filled with de-air 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 4.18, 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 continue to flow into the bottom of the sample.

(41) Apply an effective stress 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 effective stress to 15 psi and allow the sample to consolidate under 15 psi.

(43) Connect the bottom of the sample to back pressure line. Make sure both the effective stress and back pressure burettes are filled with water.

(44) Connect the four-way transducer valve 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 effective stress, the back pressure to be applied to the bottom of the sample, and the pore pressure at the top of the sample using a digital multimeter during cyclic testing.

(47) Now simultaneously raise the consolidation 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.

Figure 4.5 Rubber o-ring (Jungang Liu, 2019)

Figure 4.6 Vernier caliper used to measure the thickness of the membrane and diameter of the sand specimen (Jungang Liu, 2019)
Figure 4.7 Vernier caliper used to measure the height of the test specimen. (Jungang Liu, 2019)
Figure 4.8 Looking down on the base of the triaxial cell. (Jungang Liu, 2019)

Figure 4.9 Rubber membrane placed on the base of the triaxial cell. (Jungang Liu, 2019)
Figure 4.10 Cylindrical mold placed on the base of the triaxial cell. (Jungang Liu, 2019)

Figure 4.11 The membrane adheres on the inner wall of the mold. (Jungang Liu, 2019)
Figure 4.12 Soil sample placed within the zero raining device. (Jungang Liu, 2019)

Figure 4.13 Porous stone placed on top of the Monterey No. 0 Sand Specimen. (Jungang Liu, 2019)
Figure 4.14 Top cap placed on top of the specimen (Jungang Liu, 2019)

Figure 4.15 Positioning of the top surface of the loading cap parallel to the base of the triaxial cell (Jungang Liu, 2019)
Figure 4.16 Measuring the length of the sample using a vernier caliper (Jungang Liu, 2019)
Figure 4.17 Pi tape (Jungang Liu, 2019)

Figure 4.18 Specimen was saturating (Jungang Liu, 2019)
Samples Quality Control

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: 
\[ D_r = \frac{\gamma_{\text{max}}}{\gamma_d} \times \frac{(\gamma_d - \gamma_{\text{min}})}{(\gamma_{\text{max}} - \gamma_{\text{min}})} \times 100\% \]

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
- \( \gamma_d \) = 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:

\[ \gamma_d = \frac{w_s}{\pi(D_s^2)(H_s)/4} \]

Where \( \gamma_d \) = 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.

Soil Sample 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 15 psi 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 psi and 90 psi respectively, following which time the specimen was allowed to saturate overnight.

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 by 0.5 mv. By measuring the corresponding rise in the specimen’s pore water pressure, \( \Delta u \), the B parameter was calculated using the following equation:

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

where B = Skempton’s pore pressure parameter

\( \Delta u = \) rise in pore water pressure, change in backpressure

\( \Delta \sigma_3 = \) change in cell pressure, change in consolidation

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.

Test Procedure

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 consolidation 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 consolidation pressure to bring down the loading ram and then connect the loading ram to the sample loading cap. Record any volume change open 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 effective stress 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.

(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 0.000 kips, “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 4.19.

(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 from the triaxial cell for recycling and clean the cell for subsequent use.
Figure 4.19 Set up on the computer to run MTS. (Jungang Liu, 2019)
Tests Results

A series of isotropically consolidated undrain cyclic triaxial tests were conducted to investigate the effect of fine contents on the liquefaction resistance of soils. All triaxial specimens were prepared to attain 2 inches in diameter and 4 inches in length under three different relative densities (30%, 45% and 60%). Two-effective stresses of 15psi and 30 psi, three cyclic stress ratios (0.2, 0.3 and 0.4) were used for series of tests on the uniform medium Monterey sand containing fines at half-hertz frequency.

A total of 96 cyclic triaxial test was performed on the uniform medium Monterey No. 0/30 sand with six different percentages of fine content (5%, 10%, 15%, 25%, 35% and 45%) and plasticity index 20 showed in Table 1.

In this chapter, one set of cyclic triaxial test result is shown, and it includes soil liquefaction potential curves, cyclic axial load versus number of cycles to liquefaction and excess pore water pressure versus number of cycles to liquefaction. The rest of test results put in the Appendix D.

Table 4.1 All soil samples in cyclic triaxial test (Jungang Liu, 2019)

<table>
<thead>
<tr>
<th>Cyclic Triaxial Test</th>
<th>Relative Density (%)</th>
<th>PI</th>
<th>Fine Content (%)</th>
<th>Cyclic Stress Ratio</th>
<th>Effective Stress (psi)</th>
<th>Frequency (Hz)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>30</td>
<td>20</td>
<td>5</td>
<td>0.2</td>
<td>15</td>
<td>0.5</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td></td>
<td>10</td>
<td>0.3</td>
<td>30</td>
<td></td>
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<tr>
<td></td>
<td>60</td>
<td></td>
<td>15</td>
<td>0.4</td>
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<tr>
<td></td>
<td>45^*</td>
<td></td>
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</tr>
</tbody>
</table>

45*: Fine content 45% was only prepared at a relative density of 30 percent.
Soil Liquefaction Potential Curves

In the Figure 4.20, 4.21, 4.22, 4.23, 4.24 and 4.25, it showed that 96 cyclic triaxial tests conducted by Jungang Liu. In 96 cyclic triaxial tests, another uniform medium sand (Monterey sand) with PI 20% and fine contents were 5%, 10%, 15%, 25%, 35%, 45%, were prepared under 30%, 45% and 60% of relative densities, and run in 15 psi and 30 psi effective stress.

In the Figure 4.20 and 4.21, it showed that cyclic stress ratio versus number of cycles to liquefaction for soil samples with PI 20% and six different fines contents 5%, 10%, 15%, 25%, 35%, 45%, prepared at relative density of 30% and under 15 psi, 30 psi effective stress. In the Figure 4.20 and 4.21, it indicated that number of cycles to liquefaction did not increase with increasing percent of fine content under the cyclic stress ratio, but number of cycles to liquefaction increase with increasing effective stress. In Figure 4.20 and 4.21, it showed that soil with fine content 15%, prepared at 30% relative density and run under 15 psi, 30 psi effective stress, had the smallest number of cycles to liquefaction under the same cyclic stress ratio. Soil sample with fine content 45% had the largest number of cycles to liquefaction under the same cyclic stress ratio.

In the Figure 4.22 and 4.23, it showed that cyclic stress ratio versus number of cycles to liquefaction for soil samples with the same PI and five different fines contents 5%, 10%, 15%, 25%, 35%, prepared at relative density of 45% and under 15 psi, 30 psi effective stress.

In the Figure 4.22 and 4.23, it also indicated that number of cycles to liquefaction did not increase with increasing percent of fine content under the cyclic stress ratio, however number of cycles to liquefaction increase with increasing effective stress. Compare to Figure 4.20 and 4.21, the bigger relative density caused the more number of cycles to liquefaction. In Figure 4.22 and 4.23, it also showed that soil had the smallest number of cycles to liquefaction under the same
cyclic stress ratio when sample contained fine content 15. Soil sample with fine content 35% had the largest number of cycles to liquefaction under the same cyclic stress ratio.

In the Figure 4.24 and 4.25, it presented that cyclic stress ratio versus number of cycles to liquefaction for soil samples with PI 20% and different fines contents 5%, 10%, 15%, 25%, 35%, prepared at relative density of 60% and under 15psi, 30psi effective stress.

In the Figure 4.24 and 4.25, it also indicated that the similar tests results with different relative densities 30% and 45%. It is number of cycles to liquefaction did not increase with increasing percent of fine content under the cyclic stress ratio and also number of cycles to liquefaction increase with increasing effective stress and relative density. In Figure 4.24 and 4.25, it also showed that soil with fine content 15%, prepared at 60% relative density and run under 15psi, 30psi effective stress, had the smallest number of cycles to liquefaction under the same cyclic stress ratio. Soil sample with fine content 35% had the largest number of cycles to liquefaction under the same cyclic stress ratio.

**Figure 4.20** Cyclic stress ratio versus number of cycles to liquefaction for Monterey No.0/30 sand with different percent of fines contents at relative density of 30% and effective stress 15psi. (Jungang Liu, 2019).
Figure 4.21 Cyclic stress ratio versus number of cycles to liquefaction for Monterey No.0/30 sand with different percent of fines contents at relative density of 30% and effective stress 30psi. (Jungang Liu, 2019)

Figure 4.22 Cyclic stress ratio versus number of cycles to liquefaction for Monterey No.0/30 sand with different percent of fines contents at relative density of 45% and effective stress 15psi. (Jungang Liu, 2019)
Figure 4.23 Cyclic stress ratio versus number of cycles to liquefaction for Monterey No.0/30 sand with different percent of fines contents at relative density of 45% and effective stress 30psi. (Jungang Liu, 2019)

Figure 4.24 Cyclic stress ratio versus number of cycles to liquefaction for Monterey No.0/30 sand with different percent of fines contents at relative density of 60% and effective stress 15psi. (Jungang Liu, 2019)
**Figure 4.25** Cyclic stress ratio versus number of cycles to liquefaction for Monterey No.0/30 sand with different percent of fines contents at relative density of 60% and effective stress 30psi. (Jungang Liu, 2019)

**Cyclic Axial Load versus Number of Cycles to Liquefaction**

In Cyclic Triaxial test, a cyclic axial load of constant amplitude (24 lb) was applied on top of soil specimen with a frequency of 0.5 Hz. Figure 4.26 shows cyclic load versus number of cycles to liquefaction in Cyclic Triaxial Test.

In the first 8 cycles of Fig.4.26, the amplitude of cyclic loading was held constant without noticeable sample deformation with increasing number of cycles. It means soil still was strong. The amplitude of cyclic loading begun to decreased with increasing number of cycles after 4th cycles. After 9th cycles of Figure 4.26, the amplitude of cyclic loading rapidly dropped with increasing number of cycles. That means the both sample were liquefied and too soft in the last few cycles.
Figure 4.26 Cyclic Axial Load versus Number of cycles to liquefaction in Cyclic Triaxial Test. (Jungang Liu, 2019)

**Excess Pore Water Pressure versus Number of Cycles to Liquefaction**

Figure 4.27 shows excess pore pressure versus number of cycles in Cyclic Triaxial Test. During the first 8 cycles of cyclic axial load application, the sample showed no noticeable deformation although the pore-water pressure built up gradually. However, during the 9th stress cycle, the pore pressure suddenly increased to a value equal to the externally applied effective stress. In fact, the soil had liquefied and the effective stress had been reduced to zero. Over a wide range of strains, the soil could be observed to be in a fluid condition.

Excess 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.
Cyclic Deviator Stress versus Axial Strain

The cyclic deviator stress-axial strain graph was shown on Figure 4.28. The cyclic deviator stress of constant amplitude (7.5 psi) was applied on the top of soil specimen. In the CTT, the range of axial strain was -0.15 to +0.15 in the first three cycles. It means that the soil sample was no obvious deformation. In the last few cycles of figure 4.28, the samples turn larger axial strain which was -0.3 to +0.3. The cyclic deviator stress dropped 80 percent. It means that soil sample had liquefied.
Figure 4.28 Cyclic deviator stress versus axial strain in Cyclic Triaxial Test. (Jungang Liu, 2019)

Stress Path

Q versus P’ Curve

In the figure 4.29, it showed that the relationship between mean effective stress (p’) and q. At the beginning of cyclic triaxial test, the mean effective stress (p’) started at 17 psi, and also deviator stress (7.5 psi) applied on the sample, and also during first 8 cycles of stress application, the amplitude of cyclic deviator stress kept constant with decreasing the effective mean stress. The amplitude of cyclic deviator stress dropped 10 percent after 8th cycle in the Figure 4.29. Excess pore-water pressure increased closed to the value of applied effective stress. It means that the sample turned softer and softer.
Figure 4.29 $p'$ versus $q$ stress path in Cyclic Traixial Test. (Jungang Liu, 2019)

Effect of Fines Content on Liquefaction Resistance

It shows in the chapter 7.
CHAPTER V
HOLLOW CYLINDER TEST APPARATUS, PROGRAMS AND TEST RESULTS

ANALYSIS

Background

A hollow cylindrical apparatus (HCA) is an extremely valuable tool for studying constitutive behavior under generalized stress conditions. The HCA allows independent control of the magnitudes of the three principal stresses and rotation of the major-minor principal stress axes under simple shear conditions prevailing in earthquake shaking while recording the specimen deformational and pore pressure responses.

The University of Colorado at Denver Hollow Cylinder Torsional/Axial test cell was designed and fabricated by Dr. Jing-Wen Chen while conducting his doctoral research at UC Denver in 1988. In the hollow cylinder test at UC Denver, a hollow cylindrical soil specimen is enclosed in between an inner membrane and an outer membrane. The effective stress 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 HCT 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.

Hollow Cylinder Test Apparatus

Principles of Hollow Cylinder Testing

Figure 5.1 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 \( P_o \) establish a gradient of radial stress, \( \sigma_r \), across the cylinder wall. The relationship between radial stress, \( \sigma_r \), and the circumferential stress, \( \sigma_\theta \), is expressed by the equilibrium equation:

\[
\sigma_\theta = \sigma_r + r \left( \frac{d\sigma_r}{dr} \right) \quad (5-1)
\]

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. The stress condition in an element of a hollow cylinder specimen is shown in Fig. 5.1. Both inner and outer pressure are applied on the membrane so that there is no shear stress on the vertical boundaries, \( \sigma_r \) is always a principal stress because there are no shear stresses on circumferential surface throughout the wall.

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Figure 5.1 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).


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, $\sigma_z^{'}, \sigma_r^{'}, \sigma_\theta^{'}, \tau_{\theta z}^{'}, \tau_{z \theta}$. Hight et al. (1983) used the following expressions:

Average vertical stress $\sigma_z^{'_\text{av}} = \left[\frac{W}{\pi (b^2 - a^2)} + \frac{(P_0 b^2 - P_i a^2)}{(b^2 - a^2)}\right]$  

Average radial stress $\sigma_r^{'_\text{av}} = \frac{(P_0 b + P_i a)}{(b + a)}$  

Average circumferential stress $\sigma_\theta^{'_\text{av}} = \frac{(P_0 b - P_i a)}{(b - a)}$  

Average shear stress $\tau_{\theta z}^{'_\text{av}} = \frac{3M_T}{2\pi (b^3 - a^3)}$  

In hollow cylinder tests, the radial stress, $\sigma_r^{'_\text{av}}$, 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 $\sigma_r^{'_\text{av}}, \sigma_\theta^{'_\text{av}},$ and $\tau_{\theta z}^{'_\text{av}},$ and as following:

$$\sigma_1 = \left[\frac{(\sigma_z^{'_\text{av}} + \sigma_\theta^{'_\text{av}})}{2}\right] + \sqrt{\left[\frac{(\sigma_z^{'_\text{av}} - \sigma_\theta^{'_\text{av}})}{2}\right]^2 + (\tau_{\theta z}^{'_\text{av}})^2}$$  

$$\sigma_2 = \sigma_r^{'_\text{av}}$$  

$$\sigma_3 = \left[\frac{(\sigma_z^{'_\text{av}} + \sigma_\theta^{'_\text{av}})}{2}\right] - \sqrt{\left[\frac{(\sigma_z^{'_\text{av}} - \sigma_\theta^{'_\text{av}})}{2}\right]^2 + (\tau_{\theta z}^{'_\text{av}})^2}$$  

By regarding the specimen as a single element, the state of strain is presented in cylindrical coordinates in terms of the following components:
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 $\varepsilon_z' = \Delta H / H$

Average radial strain $\varepsilon_r' = - [(u_o - u_i) / (b-a)]$

Average circumferential strain $\varepsilon_\theta' = - [(u_o + u_i) / (b+a)]$

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

Where the definitions of average stresses and strains are shown in Figure 5.2.

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:

$\varepsilon_1 = \left[ (\varepsilon_z' + \varepsilon_\theta') / 2 \right] + \sqrt{ \left[ (\varepsilon_z' - \varepsilon_\theta') / 2 \right]^2 + \left[ \gamma_{\theta z}' / 2 \right]^2 }$

$\varepsilon_2 = \varepsilon_r'$

$\varepsilon_3 = \left[ (\varepsilon_z' + \varepsilon_\theta') / 2 \right] - \sqrt{ \left[ (\varepsilon_z' - \varepsilon_\theta') / 2 \right]^2 + \left[ \gamma_{\theta z}' / 2 \right]^2 }$
Parameters $\alpha$ and $b$ are two variables of stress path to describe fundamentally different aspects in the applied state of stress. $\alpha$ (as shown in Figure 5.1(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

$$\tan 2\alpha = 2 \tau_{0z}/ (\sigma_r - \sigma_0)$$  \hspace{1cm} (5-18)

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

$$b = (\sigma_2 - \sigma_3) / (\sigma_1 - \sigma_3)$$  \hspace{1cm} (5-19)

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 $P_i=P_o$

$$b = \sin^2 \alpha \quad \text{(Hight. et al., 1983)}$$  \hspace{1cm} (5-20)

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

$$\tan 2\alpha_{dc} = d \gamma_{0z} / (d \varepsilon_z - d \varepsilon_0)$$  \hspace{1cm} (5-21)

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 behavior, 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 5.3. The magnitude of the difference between calculated and real stress average can be characterized by normalized parameter $\beta_1$:

$$\beta_1 = \frac{|\overline{\sigma^*} - \overline{\sigma}|}{\sigma_L}$$  \hspace{1cm} (5-22)

where $\overline{\sigma^*}$ is the real average, $\overline{\sigma}$ 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:

$$\beta_3 = \frac{\int_a^b |\sigma(r) - \overline{\sigma}| dr}{(b-a)\sigma_L}$$  \hspace{1cm} (5-23)

where $\sigma(r)$ is the distribution of the particular stress, $\sigma_0$, $\sigma_\theta$ or $\tau_{0z}$ 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 $R$ ($R = \sigma_1'/\sigma_3'$):

$$\beta_R = (R_{\text{max}} - R_{\text{min}})/R'$$  \hspace{1cm} (5-24)

where $R_{\text{max}}$ and $R_{\text{min}}$ are the maximum and minimum stress ratios and $R'$ 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.

![Diagram](image)

**Figure 5.2** Definitions of average stresses and strains (after Hight *et al.*, 1983)
Figure 5.3 Definitions used for stress non-uniformity and accuracy (after Hight et al., 1983).

**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 5.5 was produced by Porovic (1995) by assuming a linear variation of applied shear stresses, \( \tau_{th} \), 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: \( H \geq 5.44 \sqrt{r_o - r_i} \)

b) Inner radius \( r_i \): \( n = (r_i / r_o) \geq 0.65 \)

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: \( 0.65 \leq (r_i / r_o) \leq 0.82 \)

c) Height: \( 1.8 \leq (H/2r_o) \leq 2.2 \)

**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 \( D_{50} \geq 0.1 \text{mm} \), 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:

\[
A_{MP} v_{MP} = \frac{1}{2} \frac{d}{D} V_{soil} \left[ \frac{(\sigma_h \cdot d)/(E_{mtm})} \right]^{1/3}
\]

\[
v_{MP} = 0.395d(1 - \alpha) \left[ \frac{(1-\alpha)/(5+64\alpha^2+80\alpha^4)} \right]^{1/3} \left[ \frac{(\sigma_h \cdot d)/(E_{mtm})} \right]^{1/3}
\]

where \( v_{MP} \) = 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 mm$^3$); $E_m$ = Young's modulus of membrane (in kN/m$^2$); $t_m$ = thickness of membrane (in mm); $\sigma'_h$ = effective stress (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:

$$\varepsilon_m = \frac{[\Delta V_{sr} - \Delta V_{ir} (n^2 - 1)]}{(A_{im} + A_{om})}$$  \hspace{1cm} (5-27)

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 $\nu_{MP}$ based on isotropic loading/unloading/reloading tests:

$$\nu_{MP} = C_{MP} \Delta \log \sigma'_h = C_{MP} \log \left(\frac{\sigma'_h}{\sigma'_{h0}}\right)$$  \hspace{1cm} (5-28)

where $C_{MP}$ 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 stresses. From Kuwano’s experiments, $C_{MP}$ is 0.015mm for 100mm diameter specimens of Ham River Sand encased in a 0.5mm thick latex membrane. Kuwano (1999) found that Eq. (5-27) matched the expressions suggested by Baldi and Nova (1984) and Kramer and Sivaneswaran (1989) very well.
Figure 5.4 Effect of stress ratio level on non-uniformity coefficients (after Vaid et al., 1990)
Figure 5.5 Shear stress distribution in hollow cylinder torsional shear test specimens (after Porovic, 1995).
Soil Samples Preparation

Soil Samples

The same soil samples were used in Chapter 4.4.1.

Mixing Soil Samples

The mixing soils procedure is the same as in Chapter 4.4.2.

Samples Preparation

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 edge of the bottom sample pedestal and seal the membrane using several O-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 soil 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) 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.

(17) 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.

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

(19) 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.

(20) 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.

(21) 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.

(22) 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.
(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°, 45°, 90° and 135° 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 bead 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 three 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) Gradually apply a effective stress increment of 2 to 3 psi while lowering the vacuum by the same increment, maintaining the initial effective stress placed on the sample by the vacuum.

(38) Continue to decrease the vacuum in increments until zero vacuum. The effective stress is now due entirely effective stress.

(39) Adjust the effective stress and back pressure simultaneously until the desired values are obtained.
Samples Quality Control

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:

\[
D_r = \frac{\gamma_{max}}{\gamma_{min}} \times \frac{(\gamma_d - \gamma_{min})}{(\gamma_{max} - \gamma_{min})} \times 100\%
\]

(5.29)

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

\[
\gamma_d = \frac{w_s}{\pi(D_{outer}^2 - D_{inner}^2) + H_s}
\]

(5.30)

Where \(\gamma_d\) = unit dry weight of the sample at desired relative density
\(w_s\) = dry weight of specimen
\(D_{outer}\) = outer diameter of specimen
\(D_{inner}\) = inner 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.
Soil Sample Saturation

The procedure of soil sample saturation is the same as in Chapter 4.3.2.2.

Determining B-Parameter

The method of determining B-Parameter is same as in Chapter 4.3.2.3.

Tests Program

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, intentionally 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.

Consolidation 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
platen and membrane retention components of the device were built to accommodate
preparation and testing of 10inch 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 consolidation 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 effective stress 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 5.6. By similar reasoning (Figure 5.7.), 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:

$$(740.0-200.0) \times [(10)^2 - (8.0)^2] + \pi/4 + 250.0 \times (2)^2 \times \pi/4 = 16053.0 \text{lb} \quad (5-31)$$

The loader must apply the failure stress in addition to the uplift on loading piston caused by effective stress, as represented by the second term in Equation 6-30.

$$\left[ (\tau_{0z})_{\text{max}} - (\tau_{0z})_{\text{avg}} \right] / (\tau_{0z})_{\text{avg}} = \frac{n \times (1-n)}{(1+n^2)} \quad (5-32)$$

Torque required to fail the same specimen was back-calculated by adapting Equation 6-6, substituting design dimensions and the maximum shear stress and rewriting:
\[
M_t = \left[ 2\pi \times \tau_{oz} \times (r_o^3 - r_i^3) \right] / 3 = \left[ 2\pi \times 115 \times (5^3 - 4^3) \right] / 3 = 14.692 \text{ in.-Ib} \quad (5-33)
\]

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

The apparatus was 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, provided adequate working room for preparation of uniform reconstituted specimens.

**Parts and their Functions**

The new hollow axial-torsional cylinder apparatus as shown in Figure.5.8 consists of eleven detachable components and three accessories used for sample preparation. The bottom platen as shown in Figure.5.9 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 o-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. 5.10 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.5.11. 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.5.11. 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-eight 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 O-rings to provide the seal for the inner chamber membrane. The exterior of the bottom annulus is grooved for O-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 semicircular 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.5.12. 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.5.12 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. 5.13 is constructed of stainless steel and 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. 5.14 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. 5.15 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.
Figure 5.6 Mohr’s circle for determining maximum vertical stress in HCTA-88 (Chen, 1988)

Figure 5.7 Mohr’s circle for determining maximum shear stress in HCTA-88 (Chen, 1988)
Figure 5.8 Schematic diagram of the HCTA-88 at the University of Colorado at Denver Geotechnical Laboratory (Chen, 1988)
Figure 5.9 Bottom Platen (Jungang Liu, 2019)

Figure 5.10 Base Plate (Jungang Liu, 2019)
Figure 5.11 Bottom Sample Pedestal (left) and Top Sample Pedestal (Jungang Liu, 2019)

Figure 5.12 Top Cap and Top Plate (Jungang Liu, 2019)
Figure 5.13 Piston Locking Mechanism (Jungang Liu, 2019)

Figure 5.14 Supporting Bars (Jungang Liu, 2019)
Figure 5.15 Pressure Chamber (Jungang Liu, 2019)
Test Procedure

After the application of final back pressure increment, soil sample is fully saturated. An Instron™ Model 1322 machine with 50,000 lb axial (compressive or tensile) and 25,000 in-Lb torsional capacity is used in performing all cyclic hollow cylinder 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 hollow cylinder cell to the Instron™ machine, center and lock the triaxial cell base to the Instron™ loading platform using three bolts.

3. Calculate the cyclic torsional torque required to produce a cyclic shear stress corresponding to a cyclic stress ratio. This load is equal to the product of initial effective stress and stress ratio times the cross sectional area of the sample.

4. Set the required cyclic torsional torque level on Instron™ machine. An invert sine wave form with a frequency of 0.5 Hz was used in this test program.

5. Set the transducer of excess pore pressure to the data logger.

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

7. Open computer on the right side of Instron™ machine.

8. Open station manager on desk of computer, and then select “Multi-Purpose Test Ware” software in MTS Systems.

9. Procedure file named “large triaxial test”, control model is force and cyclic command In cyclic command of procedure file, “Segment Shape” is Sine, “Frequency” is 0.5 Hz,
“Channel” is Torisonal, “Control Mode” is Torque, “Relative End Level 1” is 128 ft lbf, “Relative End Level 2” is -128 ft lbf.

(10) Save data file to hollow cylinder test file.

(11) Zero axial force, axial displacement, torsional torque and torsional angle on signal auto offset before run test.

(12) Test setup on computer shown in Figure 5.16.

(13) Start the test and observe the torsional torque-deformation, excess pore pressure-time, and deformation-time plotting.

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

(15) 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.

(16) Remove the hollow cylinder cell from the Instron™ machine.

(17) Remove the tested soil from the cell for recycling and clean the cell for subsequent use.

Figure 5.16: Hollow cylinder test setup in software of MTS. (Jungang Liu, 2019)
Analysis of Results

A series of cyclic torsional hollow cylinder test were performed to compare of results of liquefaction resistance of soil from cyclic triaxial and cyclic hollow cylinder tests and also investigate effect of fine contents on the liquefaction resistance of soils.

Twenty cyclic torsional hollow cylinder tests performed on the uniform medium Monterey sand with six different percentages of fine content (5%, 10%, 15%, 25%, 35% and 45%) and plasticity index 20.

Table 5.1 detailed test program with information including specimen relative densities, effective stress, cyclic stress ratio and frequency. All hollow cylinder samples are inner diameter of 8 inches, outer diameter of 10 inches, and height of 10 inches. The properties of all soil sample are the same as in cyclic triaxial test in Chapter 4.

In this chapter, one set of cyclic hollow cylinder test result is shown, and it includes cyclic torque versus number of cycles to liquefaction and excess pore water pressure versus number of cycles to liquefaction. The rest of test results put in the Appendix D.

Table 5.1 Test program details in hollow cylinder test (Jungang Liu, 2019)

<table>
<thead>
<tr>
<th>Hollow Cylinder Test</th>
<th>Relative Density (%)</th>
<th>PI</th>
<th>Fine Content (%)</th>
<th>Cyclic Stress Ratio</th>
<th>Effective Stress (psi)</th>
<th>Frequency (Hz)</th>
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*: each percentage of fine content run under corresponding cyclic stress ratio.
Cyclic Torque versus Number of Cycles to liquefaction

In HCT test, a cyclic torsion load of constant amplitude (40lbf-ft) was applied with a frequency of 0.5Hz to a sample of saturated sand. Figure 5.17 shows cyclic torsion loading versus number of cycles to liquefaction in HCT test.

In the first 4 cycles of Fig.5.17, the amplitude of cyclic loading was held constant without noticeable sample deformation with increasing number of cycles. It means soil still was strong. However, in 5th cycles of Figure 5.17, the amplitude of cyclic loading dropped to 25lbf-ft. The amplitude of cyclic loading begun to decreased with increasing number of cycles after 4th cycles. After 4th cycles of Figure 5.17, the amplitude of cyclic loading rapidly dropped with increasing number of cycles. That means the both sample were liquefied and too soft in the last few cycles.

Figure 5.17 Cyclic Torque versus Number of cycles to liquefaction in Hollow Cylinder Test. (Jungang Liu, 2019)
Excess Pore Water Pressure versus Number of Cycles to liquefaction

Figure 5.18 shows excess pore pressure versus number of cycles in Hollow Cylinder test. Excess pore–water pressure starts to build up with the number of torsion loading cycles increase in the first 4 cycles. During the first 4 cycles of cyclic torsion application, the sample was no noticeable deformation although the pore-water pressure built up gradually. Nevertheless, the pore water pressure rapidly increased to the equal to the externally applied effective stress in the 5th cycle. It showed that the soil had liquefied after the 5th cycle.

Figure 5.18 Pore water pressure change versus Number of cycles to liquefaction in CHCT. (Jungang Liu, 2019)
Cyclic Shear Stress versus Shear Strain

The cyclic shear stress-shear strain graph is also shown on Figure 5.19. For the first 3 cycles the curves are close together, but as the sample approaches failure the strains increase and the hysteresis loops open up quickly. In the first 3 cycles, the range of shear strain was -5.0 to +5.0. 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 5th cycle, the loops begun flat shape, and also the amplitude of deviator stress rapidly dropped 30 percent. The sample developed large strains which, in the 5th cycle, exceeded 50 percent during the last three cycles. That means that the sample had liquefied. In last two cycles of Figure 5.18, the ranges of shear strain were -10% to +10%.

**Figure 5.19** Cyclic shear stress versus shear strain in Hollow Cylinder Test. (Jungang Liu, 2019)
**Stress Path**

Q Versus P’ Curve

The q-p’ graph reflects the gradual buildup of pore pressure as the mean effective stress (p’) reduces until the sample approaches liquefaction condition at which time the sample starts failing and the amplitude of cyclic shear stress begun to drop. When the soil sample had liquefied, pore-water pressure equaled to the externally applied effective stress. In the figure 5.20, it showed that the mean effective stress p’ ($p' = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$) versus q($q = \frac{1}{2}(\sigma_1 - \sigma_3)$). In the beginning of hollow cylinder test, mean effective stress p’ started at 15psi and cyclic shear stress q at 4.5 psi applied on the soil sample. In 5th cycle of figure 5.19, the amplitude of cyclic shear stress rapidly dropped 33 percent. At the same time, the effective mean stress kept decrease. It means that the soil sample turned softer and softer.

**Figure 5.20** $p'$ versus q stress path in Hollow Cylinder Test. (Jungang Liu, 2019)
Effect of Fines Content on Liquefaction Resistance

It shows in the Chapter 7.
CHAPTER VI
COMPARISON OF CYCLIC TRIAXIAL AND HOLLOW CYLINDER TEST RESULTS

Introduction

A series of isotropically consolidated undrained cyclic triaxial (CTT) and cyclic hollow cylinder tests (CHCT) were conducted to determine and compare the behavior and liquefaction resistance of soil specimens. In this research, two soil specimens were included for comparing the liquefaction resistance of soil in CTT and CHCT. One of soil specimens was uniform medium clean Monterey No. 0/30 sand, another is the mixture of a uniform medium Monterey No. 0/30 sand and Leyden clay from Golden Colorado.

Eighteen cyclic triaxial and seventeen hollow cylinder tests were performed on the uniform medium clean Monterey No. 0/30 sand. All thirty five specimens were prepared at four different relative densities of 30, 45, 50, 60 percent and tested at frequency 0.5 Hertz. Table 6.1 detailed this comparative test program with information including specimen densities, effective stresses, and cyclic stress ratios, number of cycles to liquefaction for both CTT and CHCT.

The mixture of a uniform medium Monterey No. 0/30 sand and Leyden clay with five different percentages of fine content (5%, 10%, 15%, 25% and 35%) and plasticity index 20. Twenty cyclic triaxial and twenty cyclic hollow cylinder tests were performed on the mixture samples. All forty samples were prepared at two different relative densities of 30 and 60 percent and tested at frequency 0.5 Hertz. Table 6.2 detailed comparative test program.

All triaxial specimens were prepared to attain 2 inches in diameter and 4 inches in length and hollow cylinder samples to inner diameter of 8 inches, outer diameter of 10 inches, and height of 10 inches.
Table 6.1: Result of comparative study on clean sand (Liu, et al, 2017)

<table>
<thead>
<tr>
<th>Relative Density (%)</th>
<th>(psi)</th>
<th>Cyclic Stress Ratio*</th>
<th>Frequency (Hz)</th>
<th>Number of Cycles to liquefaction</th>
<th>Relative Density (%)</th>
<th>Effective Stress (psi)</th>
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Stress Ratio*: Cyclic stress ratio (cyclic triaxial test) = $\frac{\sigma_{cc}}{\sigma_a}$ Cyclic stress ratio (cyclic hollow cylinder test) = $\frac{\tau_h}{\sigma'_v}$
Table 6.2: Result of Comparative Study on mixture samples (Jungang Liu, 2019)

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<th>Relative Density (%)</th>
<th>Effective Stress (psi)</th>
<th>Cyclic Stress Ratio*</th>
<th>Fine Content (%)</th>
<th>Number of Cycles to liquefaction</th>
<th>Relative Density (%)</th>
<th>Effective Stress (psi)</th>
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<td>60.59</td>
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<td>88</td>
<td>59.71</td>
<td>30</td>
<td>0.4</td>
<td>35</td>
<td>39</td>
</tr>
</tbody>
</table>

Cyclic stress ratio*: Cyclic stress ratio (cyclic triaxial test) = \( \frac{\sigma_c}{2\sigma_a} \) Cyclic stress ratio (cyclic hollow cylinder test) = \( \frac{\tau_h}{\sigma_v} \)
Cyclic Strength

Correction Factor between Cyclic Stress Ratio Causing Liquefaction in the Field and Cyclic Stress Ratio Causing Liquefaction of Triaxial Test Sample in the Laboratory

Seed (1979) found that the cyclic stress ratio causing liquefaction under multidimensional shaking conditions in the field is related to the cyclic stress ratio causing liquefaction of a triaxial test sample in the laboratory by expression:

\[
\frac{\tau_{\text{field}}}{\sigma'} \approx C_r \left(\frac{\sigma_{dc}}{2\sigma_a}\right)_{\text{triaxial}}
\]

Where \( C_r = 0.57 \) for \( K_0 = 0.4 \)

\( C_r = 0.9 \) to 1 for \( K_0 = 1 \)

Table 6.3 Values of \( C_r \) (some researchers)

<table>
<thead>
<tr>
<th>Source</th>
<th>Equations</th>
<th>( C_r ) for ( K_0 = 0.4 )</th>
<th>( C_r ) for ( K_0 = 1.0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finn, et al</td>
<td>( C_r = (1+ K_0)/2 )</td>
<td>0.7</td>
<td>1.0</td>
</tr>
<tr>
<td>Seed and Peacock</td>
<td>Varies</td>
<td>0.55-0.72</td>
<td>1.0</td>
</tr>
<tr>
<td>Castro</td>
<td>( C_r = (1+ K_0)/(3\sqrt{3}) )</td>
<td>0.69</td>
<td>1.15</td>
</tr>
</tbody>
</table>

In this research, a series of cyclic hollow cylinder and cyclic triaxial tests were performed to establish the following relation between the cyclic liquefaction resistance stresses ratios of these two types of tests, the former closely simulates the field shear wave propagation during earthquake shaking. The main objective is to evaluate the correction factor, \( C'_{r} \), in the following equation:

\[
\frac{\tau_{h}}{\sigma'} \approx C'_{r} \left(\frac{\sigma_{dc}}{2\sigma_a}\right)_{\text{triaxial}}
\]

The results of this comparison on the clean sand are summarized Table 6.4. As may be seen, the \( C'_r \) values range from 0.46 to 0.63 at the same relative densities of 30 percent and 60 percent, which falls within the range of values that Seed assessed in 1979. For the samples with different fines content, Table 6.5 showed results of the comparison. The \( C'_r \) values range from
0.32 to 0.93 at soil samples with different fines content. For 30 percentage of relative density in table 6.5, the $C_r'$ values range from 0.5 to 0.93, but $C_r'$ values at 60 percent of relative density range from 0.32-0.44.

Table 6.6 showed the correction coefficient matrix of dependent variable $C_r'$ and independent variables. After run the correction coefficient analysis, the dependent variable is correction factor $C_r'$. Three independent variables are relative density in cyclic triaxial test, $D_r$ (CTT), number of cycles to liquefaction in cyclic triaxial test No. cycles (CTT) and relative density in cyclic hollow cylinder test, $D_r$ (CHCT). A linear regression model was obtained as follows: $C_r'=1.136+0.015*D_r$(CTT)-0.002*No.Cycles (CTT)-0.030*D_r (CHCT). The $R^2$ value of the regression equation is 0.84.
Table 6.4: $C_r$ values on the clean sand produced in Laboratory by CTT and CHCT (Liu, et al, 2017)

<table>
<thead>
<tr>
<th>Number of Cycles to liquefaction</th>
<th>Effective Stress (psi)</th>
<th>Cyclic Stress Ratio</th>
<th>Target Relative Density (%)</th>
<th>$C_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cyclic Triaxial Test</td>
<td>Cyclic Hollow Cylinder Test</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cyclic Triaxial Test</td>
<td>Cyclic Hollow Cylinder Test</td>
<td>Cyclic Triaxial Test</td>
<td>Cyclic Hollow Cylinder Test</td>
<td></td>
</tr>
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<td>15</td>
<td>15</td>
<td>0.3</td>
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Table 6.5: $C_r'$ values on the mixture samples produced in laboratory by CTT and CHCT (Jungang Liu, 2019)

<table>
<thead>
<tr>
<th>Number of Cycles to liquefaction</th>
<th>Effective Stress (psi)</th>
<th>Cyclic Stress Ratio</th>
<th>Fine Content (%)</th>
<th>Target Relative Density (%)</th>
<th>$C_r'$</th>
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</thead>
<tbody>
<tr>
<td>Cyclic Triaxial Test</td>
<td>Cyclic Hollow Cylinder Test</td>
<td>Cyclic Triaxial Test</td>
<td>Cyclic Hollow Cylinder Test</td>
<td>Cyclic Triaxial Test</td>
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<td>30</td>
<td>0.4</td>
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</table>
Table 6.6 Correlation coefficient matrix of dependent variable Cr’ and independent variables (Jungang Liu, 2019)

<table>
<thead>
<tr>
<th></th>
<th>Relative Density (%) in CTT</th>
<th>Number of Cycles to liquefaction in CTT</th>
<th>Relative Density (%) in CHCT</th>
<th>Effective Stress (psi)</th>
<th>Cyclic Stress Ratio</th>
<th>Fine Content (%)</th>
<th>Number of Cycles to liquefaction for CHCT</th>
<th>Cr’</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative Density (%) in CTT</td>
<td>1</td>
<td></td>
<td>0.998</td>
<td>0.841</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of Cycles to liquefaction for CTT</td>
<td>0.854</td>
<td>1</td>
<td>0.490</td>
<td>-0.019</td>
<td>1</td>
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<td></td>
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<tr>
<td>Relative Density (%) in CHCT</td>
<td>0.998</td>
<td>0.841</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Effective stress (psi)</td>
<td>0.003</td>
<td>0.490</td>
<td>-0.019</td>
<td>1</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Cyclic Stress Ratio</td>
<td>0.069</td>
<td>-0.017</td>
<td>0.061</td>
<td>-0.069</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fine Content (%)</td>
<td>-0.002</td>
<td>0.002</td>
<td>-0.007</td>
<td>0</td>
<td>0.070</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of Cycles to liquefaction for CHCT</td>
<td>0.507</td>
<td>0.854</td>
<td>0.487</td>
<td>0.861</td>
<td>0.016</td>
<td>-0.011</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Cr’</td>
<td>-0.882</td>
<td>-0.647</td>
<td>-0.894</td>
<td>0.301</td>
<td>0.028</td>
<td>-0.141</td>
<td>-0.183</td>
<td>1</td>
</tr>
</tbody>
</table>
Comparison on Both Test Results

Cyclic Torque, Cyclic Axial Load versus Number of Cycles to Liquefaction

In the Hollow Cylinder Test, the height of sample is 10.00 in, outside diameter is 10.00 in and inside diameter is 8.00 in. The mass of sample is 6900.5g, and its relative density is 30.0%. The “B” parameter is 0.96, and also its effective stress is 80psi, its back pressure is 65 psi, its S.R. is 0.25. However, In Cyclic Triaxial Test, the height of sample is 4.0 inch; its diameter is 2.00 inch. The weight of sample is 316.44g, and its relative density is 30.0%. The “B” parameter is 0.96 when its effective stress is 80psi, and its back pressure is 65psi, its S.R. is 0.25.

In HCT test, a cyclic torsion load of constant amplitude (40lbf-ft) was applied with a frequency of 0.5Hz to a sample of saturated sand. Figure 6.1 shows cyclic torsion loading versus number of cycles to liquefaction in HCT test. In Cyclic Triaxial test, a cyclic axial load of constant amplitude (24 lb) was applied on top of soil specimen with a frequency of 0.5 Hz. Figure 6.2 shows cyclic load versus number of cycles to liquefaction in Cyclic Triaxial Test.

In the first 8 cycles of Fig.6.2, 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 5th cycles of Figure 6.1, the amplitude of cyclic loading dropped to 25lbf-ft. The amplitude of cyclic loading begun to decreased with increasing number of cycles after 4th cycles. In cyclic triaxial test, more of numbers cycles to liquefaction than in HCT test. In cyclic triaxial test, soil specimen stronger than in HCT test under the same cyclic stress ratio and relative density.

After 4th cycles of Figure 6.1 and 9th cycles of Figure 6.2, the amplitude of cyclic loading rapidly dropped with increasing number of cycles. That means the both sample were liquefied and too soft in the last few cycles.
Figure 6.1 Cyclic Torque versus Number of cycles to liquefaction in Hollow Cylinder Test (Liu, et al, 2017)
Figure 6.2 Cyclic Axial Load versus Number of cycles to liquefaction in Cyclic Triaxial Test. (Liu, et al, 2017)
Pore Water Pressure versus Number of Cycles to Liquefaction

Figure 6.3 shows excess pore pressure versus number of cycles in Hollow Cylinder test. During the first 4 cycles of cyclic torsion application, the sample showed no noticeable deformation although the pore-water pressure built up gradually. However, during the 5th stress cycle, the pore pressure suddenly increased to a value equal to the externally applied effective stress. In fact, the soil had liquefied and the effective stress 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 6.4 shows pore water pressure change versus Number of cycles to liquefaction in Cyclic Triaxial Test. Compare to hollow cylinder test, it need more number of stress cycles to make pore pressure increased to a value equal to the externally applied effective stress. In HCT test, pore water pressure of soil specimen more faster build up to be equal to effective stress 15 psi than in CTT test.
Figure 6.3 Pore water pressure change versus Number of cycles to liquefaction in HC Test. (Liu, et al, 2017)
Figure 6.4 Pore water pressure change versus Number of cycles to liquefaction in Cyclic Triaxial Test. (Liu, et al, 2017)
Cyclic Shear Stress, Cyclic Deviator Stress versus Shear Strain, Axial Strain

The shear stress-shear strain graph is also shown on Figure 6.5. For the first 3 cycles the curves are close together, but as the sample approaches failure the strains increase and the hysteresis loops open up quickly. In the first 3 cycles, the range of shear strain was -5.0 to +5.0. 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 4th cycle, the loops begun flat shape, and also the amplitude of deviator stress rapidly dropped 30 percent. The sample developed large strains which, in the 5th cycle, exceeded 50 percent during the last three cycles. That means that the sample had liquefied. In last two cycles of Figure 6.5, the ranges of shear strain were -10% to +10%.

In HCT test, a cyclic shear stress of constant amplitude (4.5 psi) was applied with a frequency of 0.5Hz to a sample of saturated sand. However, a cyclic deviator stress (7.5 psi) was applied on the top of soil specimen with the same frequency of 0.5 Hz. In CTT, the range of axial strain was -0.15 to +0.15 in the first three cycles. It is not noticeable deformation like in hollow cylinder test. In last few cycles of Figure 6.6, the amplitude of deviator stress decreased with increasing the effective stress, and also soil sample was softened. In last cycles of Figure 6.6, the sample got larger strain, which was -0.3 to 0.3. It means that the sample had liquefied.
Figure 6.5 Cyclic shear stress versus shear strain in Hollow Cylinder Test (Liu, et al, 2017)
Figure 6.6 Cyclic deviator stress versus axial strain in Cyclic Triaxial Test. (Liu, et al, 2017)
Stress Path

Q versus P’ Curve

Figure 6.7 showed the mean effective stress (p’) started at 15psi, and also cyclic shear stress (q) started at 4.5psi applied on the sample in the beginning of Hollow Cylinder test. In the first 4 cycles of shear stress application, the amplitude of cyclic shear stress kept constant with decreasing the effective mean stress. However, at the beginning of cyclic triaxial test, the mean effective stress (p’) started at 17psi, also deviator stress (7.5psi) applied on the sample, and during first 8 cycles of stress application, the amplitude of cyclic deviator stress kept constant with decreasing the effective mean stress.

The q-p’ graph reflects the gradual buildup of pore pressure as the mean effective stress (p’) reduces until the sample approaches liquefaction condition at which time the sample starts failing and the amplitude of cyclic shear stress begun to drop. When the soil sample had liquefied, pore-water pressure equaled to the externally applied effective stress. In 5th cycle of hollow cylinder test, the amplitude of cyclic shear stress rapidly dropped 33 percent. However, in the cyclic triaxial test of figure 6.8, the amplitude of cyclic deviator stress dropped 10 percent after 8th cycle. It means pore-water pressure increased closed to the value of applied effective stress. After 5th cycle of hollow cylinder test (8th cycle of cyclic triaxial test), the amplitude of cyclic shear stress continued to drop, at the same time, the effective mean stress kept decreased. It means that the sample turned softer and softer.
Figure 6.7 $p'$ versus $q$ stress path in Hollow Cylinder Test. (Liu, et al, 2017)
Figure 6.8 $p'$ versus $q$ stress path in Cyclic Triaxial Test. (Liu, et al, 2017)
CHAPTER VII
THRESHOLD FINES CONTENT

Previous Studies

In Dr. Hsing-Cheng Liu’s dissertation (1992), it found that inclusion of fines in a clean Denver sand at a constant overall void ratio does not necessarily cause increase in liquefaction resistance, and also at a constant overall void ratio, the increase in fines content at a constant plasticity index up to 30% in the clean sand cause a decrease in liquefaction resistance, beyond 30%, the further increase in fines content results increase in liquefaction resistance.

Based on run triaxial test with samples tested under 15 psi confining pressure in the medium-sand with 5% fines and plasticity indices ranging from 0 to 40, Nien-Yin Chang (1990) showed that the trend of increasing liquefaction resistance with increasing plasticity index is more obvious, and also comparison between the effect of fine contents and the effect of plasticity indices of fines on the liquefaction resistance of soil indicated that the effect of fine contents is more significant than the effect of plasticity indices.

I.M.Idriss and R.W.Boulanger (2004) showed that cyclic stress ratio (CSR) increased with increased fine content percent under the same modified standard penetration blows count. The cases for cohesionless soils with FC ≥ 35% are plotted in Figure 7.1. Figure 7.2 showed the case history points for cohesionless soils with 5%< FC <15%, while Figure 7.3 showed the cases for 15% ≤ FC < 35%.

However, Chang (1990) showed the clean medium sand has the strongest liquefaction resistance and as the fine content increases, the liquefaction resistance decreases until the fine content reaches approximately 26%, as indicated by the liquefaction potential curves shifting
toward the left. Then the trend reverses itself and soils begin to gain strength as the fine content further increases, as indicated by the curves shifting toward the right. In Figure 7.4, it showed for the medium-sand test series, exactly the similar trend of decreasing resistance and then increasing resistance with the increasing fine content. Each curve in Figure 7.4 gave the stress ratios required to achieve initial liquefaction in ten cycles of loading for soils with the same plasticity index for each curve. The collection of curves also indicates the existence of a threshold fine content, below which, the resistance decrease, and, above which, the resistance increases with increasing fine content, and the threshold fine content decrease with increasing plasticity of soils. Figure 7.4, 7.5, and 7.6 seem to indicate that the threshold fine content increases as the number of cycles required reaching initial liquefaction increases.

Tzuo-Shin Ueng, Chia-Wen Sun and Chieh-When Chen (2013) showed that as the fine content increased, cyclic resistance ratio decreased until the fine content approximately 20% in the Figure 7.7. And also indicated that the effect of fines on the liquefaction resistance of a soil was more prominent using (FC)\textsuperscript{400} (passing the No.400 sieve) than (FC)\textsuperscript{200} (passing the No. 200 sieve), probably due partly to the different in plasticity of the fines.

Yong Wang and Yanli Wang (2010) showed that with a constant dry density, the liquefaction resistance first increased and then decreased with the increase of the fines content, when the relative density reached the minimum value at the fines content of 30% and the liquefaction resistance also reached the minimum value at the same fines content in Figure 7.8.

Mehmet Murat Monkul and Jerry A. Yamamuro (2011) showed that relative density alone cannot be a consistent comparison basis for the influence of fines content on liquefaction potential of sand in Figure 7.9, and also the D_{50-sand}/d_{50-silt} ratio becomes larger (for SilCoSil and
Potsdam fines), the void ratio decreases consistently with increasing fines content, show as Figure 7.10.

Polito and Martin (2001) used Monterey and Yatesville sand with Yatesville silt to study the effects of non-plastic fines on the liquefaction resistance of sands. Their study showed the transitional fines content around 35 %, based on the relationship between cyclic stress ratio and void ratio. Thevanayagam et al. (2002) used foundry sand and silica fines to study the undrained fragility of clean sands, silty sands, and sandy silts. The transitional fines content based on the steady state line was around 40 %.

Shaoli Yang, Suzanne Lacasse, and Rolf Sandven (2005) showed the curves, based on the cyclic test results, appear change direction at fines content of 30% and the curves for threshold fine content of 30% always give low the void ratio and low cyclic stress ratio.

Chang (1990) showed that the medium sand with a small fine content of 5% reflects in the irregular relationship between the stress ratio required to reach initial liquefaction versus plasticity index, and also comparison between the effect of fine contents and the effect of plasticity indices of fines on the liquefaction resistance of soils indicated that the effect of fine contents is very much more significant than the effect of plasticity indices.
Fig. 7.1 SPT case histories of cohesionless soils with FC ≥ 35% and the NCEER Workshop (1997) curve and the recommended curves for both clean sand and for FC = 35% for M = 7½ and $\sigma'_{vo} = 1$ atm (I.M. Idriss and R.W. Boulanger, 2004).
Fig. 7.2 SPT case histories of cohesionless soils with 5% < FC < 15% and the recommended curves for both clean sand and for FC = 15% for M = 7½ and σ'₀ = 1 atm (I.M. Idriss and R.W. Boulanger, 2004)
Fig. 7.3: SPT case histories of cohesionless soils with $15\% < FC < 35\%$ and the NCEER Workshop (1997) curve and the recommended curves for both clean sand and for $FC = 15\%$ for $M = 7\frac{1}{2}$ and $\sigma'_o = 1$ atm (I.M. Idriss and R.W. Boulanger, 2004)
Figure 7.4 Stress Ratio Required to reach Liquefaction in 10 Cycles versus Fine Content Under Confining pressure 15 psi (N.Y. Chang, 1990)

Figure 7.5 Stress Ratio Required to reach Liquefaction in 30 Cycles versus Fine Content Under Confining pressure 15 psi (N.Y. Chang, 1990)
**Figure 7.6** Stress Ratio Required to reach Liquefaction in 100 Cycles versus Fine Content Under Confining pressure 15 psi (N.Y. Chang, 1990)

**Figure 7.7** Cyclic resistance ratio versus equivalent (FC) subscripts (Tzuo-Shin Ueng, Chia-Wen Sun and Chieh-When Chen, 2013)
Fig. 7.8 Variation in Liquefaction Resistance with Fines Content for $N_l=20$ (Yong Wang and Yanli Wang, 2010)

Fig. 7.9 Change of relative density and liquefaction potential with different fines contents and Silts for tested specimens. (Mehmet Murat Monkul and Jerry A. Yamamuro, 2011)
Fig. 7.10 Change of void ratio and liquefaction potential with different fines contents and silts for tested specimens. (Mehmet Murat Monkul and Jerry A. Yamamuro, 2011)
Threshold Fines Content

Definition

Research has shown that the behavior of the mixtures can be characterized into two groups, one is sand-dominated, and the other is fines-dominated.

The intergrain state concept was proposed by Thevanayagam (1998). The intergrain void ratio (referring to the intergranular and interfine void ratios) is used to indicate the state of the sand and silt mixtures instead of the global void ratio.

For sand with low fines content, the intergranular void ratio \( e_c \) is defined as:

\[
e_c = \frac{e + fc}{1 - fc}
\]

where \( fc \) is fines content (in decimal) and \( e \) is the void ratio of the silty sand.

For sand with high fines content, the interfine void ratio \( e_f \) is defined as:

\[
e_f = \frac{e}{fc}
\]

Threshold fines content is a transitional fines content, sand-dominated behavior passes to fines-dominated behavior when the fines content is beyond the threshold fines content.

Threshold fines content \((FC_{th})\) is expected to occur when the shear response of the mixtures is expected to weaken, fine grains plays a primary role.

\[
FC_{th} \leq \frac{100e_c}{1 + e_c + e_{max, HF}} \% = \frac{100e}{e_{max, HF}} \%
\]

where \( e_{max, HF} \) = the maximum void ratio of the pure silt above which it has no appreciable strength.

Factors Effects Threshold Fines Content

Factors from Soil Properties

The threshold fines content depend on many factors from soil properties such as the fine particles, volumes of solids and void, the values of \( e_{max} \) and \( e_{min} \), global void radio \((e)\) and the characteristics of fines and coarse grains

Fines particles

In the intergrain state concept, the fine particles are regarded as void when the fines content is low (Fig. 7.11), and the fines are assumed to not participate in the resistance of shearing. When the fines content is high, the sand grains are regarded as void, and the sand particles are assumed to not contribute to the shearing resistance (Fig. 7.11).
Fig. 7.11 (a) Sand with low fines content (b) Sand with high fines content
(Shaoli Yang, Suzanne Lacasse and Rolf Sandven, 2005)

Volume solids and void

When mixing spherical particles of two different sizes, the packing will be affected by the proportion of large-size and small-size spheres in the total volume of solids as well as by the relative size of the large and small spheres.

Misko Cubrinovski and Kenji Ishihara (2002) showed how the volumes of solids and void vary with a change in the percentage of the small-size particles in Figure 7.12. In Figure 7.12, point L means the densest possible packing of the larger spheres. In the beginning, adding smaller size particles into the densest packing of large spheres cause to a decrease in the volume of voids because the small spheres fill in the voids among the larger particles. This filling-of-voids phase is showed in the diagram with the path L-T. Upon adding small particles beyond a certain percentage corresponding to point T, a reverse trend is observed in which the volume of voids increase with the percentage of the small-size fraction. In this so-called replacement-of-solids phase, the large-size particles are pushed apart and gradually replaced by the small-size spheres until the entire volume of solids are comprised of smaller particles at point S. It is indicated that $e_{\text{min}}$ decreases in the course of the filling-of-voids process and reaches its minimum value of $e_{\text{min}(T)}$ at the threshold percentage corresponding to T, and then $e_{\text{min}}$ steadily increases during the replacement-of-solids process at the path T-S in the Figure7.12(b).

Misko Cubrinovski and Kenji Ishihara (2002) exposed that fines-containing sands are that the threshold percentage of fines at which the filling process is reversed into the replacement process is significantly smaller than 50%.
Figure 7.12 Effects of fines on Binary Packing of Spherical Particles: (a) variation in the volume of voids and solids (b) variation on $e_{\text{min}}$. (Misko Cubrinovski and Kenji Ishihara, 2002)

$e_{\text{max}}$ and $e_{\text{min}}$

Misko Cubrinovski and Kenji Ishihara (2002) showed that the values of $e_{\text{max}}$ and $e_{\text{min}}$, have been determined by the adopted laboratory procedures, as a function of the fines contents for each of the composite soils, as shown in Figure 7.13. In Figure 7.13, it is observed that $e_{\text{max}}$ and $e_{\text{min}}$ decreased as the fines content increased from 0% to 20%. However, $e_{\text{max}}$ and $e_{\text{min}}$ reached the lowest void ratio within the range of 20 to 40% because soil had a transition from the filling-of-voids to the replacement-of-solids process. After 40% fines, $e_{\text{max}}$ and $e_{\text{min}}$ are seen to steadily increase until they reach the highest values at 100% fines content.
Figure 7.13 Variation in $e_{\text{max}}$ and $e_{\text{min}}$ with fine content of mixtures of Cambria Sand and Nevada fines. (Misko Cubrinovski and Kenji Ishihara, 2002)

Global void ratio, characteristics of fines and coarse grains

S. Thevanayagam; T. Shenthan; S. Mohan and J. Liang (2002) found that the value of threshold fines content ($F_{Cth}$) depends on global void ratio ($e$) and the characteristics of fines and coarse grains. It was found that at the same $e$ and confining stress, the collapse potential (fragility) of silty sand increases with an increase in fines content (FC) due to a reduction in intergranular contact between the coarse grains. At $FC<F_{Cth}$, intergranular contact friction plays the primary role. At large fines contents ($FC>F_{Cth}$), fine grain friction plays a primary role and dispersed coarse grains provide a beneficial, secondary reinforcement effect.

They showed that the microstructure of a granular mix, which can be constituted in many different ways with different types of intergrain contacts, leads to different undrained shear responses in the Figure 7.14. There are three extreme limiting categories of microstructure:

(a) primarily the coarse grains are in contact [cases (i)–(iii) in Fig. 7.14 (a)],
(b) primarily the fine grains are in contact with each other [case (iv) in Fig. 7.14 (b)],
(c) a layered system [Fig. 7.14 (c)].
Factors from Laboratory Tests

There are many factors affecting threshold fines content in the laboratory test. They include relative density, cyclic stress ratio, consolidation pressure and number of cycles to liquefaction. Based on all laboratory test results from cyclic triaxial tests and cyclic hollow cylinder tests, they indicated that threshold fines content was happened and the value was 15\%. Table 7.1 showed threshold fines content, test parameters and soil properties in cyclic triaxial test and hollow cylinder test.

Table 7.2 showed the correction coefficient matrix of dependent variable threshold fines content and independent variables. After run the correction coefficient analysis, the dependent variable is threshold fines content, TFC. Three independent variables are relative density, \(D_r\), number of cycles to liquefaction, \(No\). and consolidation pressure, \(CP\). A linear regression model was obtained as follows: \(TFC = 14.88 - 0.022*D_r + 0.027*No. - 0.009*CP\). The \(R^2\) value of the regression equation is 0.44.
Table 7.1 Threshold fines content in cyclic triaxial test and cyclic hollow cylinder test (Jungang Liu, 2019)

<table>
<thead>
<tr>
<th>Cyclic Traxial Test</th>
<th>Threshold Fines Content (%)</th>
<th>Relative Density (%)</th>
<th>No. Cycles to Liquefaction</th>
<th>Consolidation Pressure (psi)</th>
<th>Cyclic Stress Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>14</td>
<td>30</td>
<td>10</td>
<td>15</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>30</td>
<td>35</td>
<td>30</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>45</td>
<td>35</td>
<td>15</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>45</td>
<td>50</td>
<td>30</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>14.5</td>
<td>60</td>
<td>50</td>
<td>15</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>15.5</td>
<td>60</td>
<td>90</td>
<td>30</td>
<td>0.2</td>
</tr>
<tr>
<td>Cyclic Hollow Cylinder Test</td>
<td>14</td>
<td>30</td>
<td>8</td>
<td>15</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>30</td>
<td>27</td>
<td>30</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>14.5</td>
<td>60</td>
<td>20</td>
<td>15</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>60</td>
<td>40</td>
<td>30</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 7.2 Correlation coefficient matrix of threshold fines content and laboratory factors (Jungang Liu, 2019)

<table>
<thead>
<tr>
<th>Threshold Fines Content (%)</th>
<th>Relative Density (%)</th>
<th>No. Cycles to Liquefaction</th>
<th>Consolidation Pressure (psi)</th>
<th>Cyclic Stress Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Threshold Fines Content (%)</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>0.0831</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. Cycles to Liquefaction</td>
<td>0.5765</td>
<td>0.5929</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Consolidation Pressure (psi)</td>
<td>0.3716</td>
<td>0</td>
<td>0.5259</td>
<td>1</td>
</tr>
<tr>
<td>Cyclic Stress Ratio</td>
<td>0.0000</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Results of Laboratory Test

All Soil Samples Results in Cyclic Triaxial Test

A series of isotropically consolidated undrain cyclic triaxial tests were conducted to investigate the effect of fine contents on the liquefaction resistance of soils. Ninety-six cyclic
triaxial tests were performed on the uniform Monterey No. 0/30 sand with six different percentages of fine content (5%, 10%, 15%, 25%, 35% and 45%) and plasticity index 20.

**Stress Ratio for 10, 35, 50 and 90 Cycles to Liquefaction vs Different Percent of Fine Content**

The figures of stress ratio vs number of cycles to liquefaction for soil with different percent of fine contents to determine the stress ratio required to reach liquefaction in 10, 35, 50 and 90 cycles for all sample tested. In the Figure 7.15, it shows the relationship between the stress ratio for 10 and 35 cycles to liquefaction and soil with different percent of fine contents (5%, 10%, 15%, 25%, 35% and 45%) and the same PI. In the Figure 7.15, soil samples prepared at relative density of 30% and run under 15 psi, 30psi of effective consolidation stress. The Curve of Figure 7.15 indicated that the liquefaction resistance decreases as the fine content increase until the fine content reaches 15%. The soil liquefaction resistance has the lowest value at 15 % of fine content. After then the trend reverse itself and soil begin to gain strength with fine content increase.

Figure 7.16 present that the relationship between the stress ratios required reaching liquefaction in 35 and 50 cycles and soil with various percent of fines content (5%, 10%, 15%, 25% and 35%). In the Figure 7.16, all samples prepared at relative density of 45% and run under 15 psi, 30psi of effective consolidation stress. The curves in figure 7.15 and 7.16 are very similar sharp.

Figure 7.17 present that the relationship between the stress ratios required reaching liquefaction in 50 and 90 cycles and soil with various percent of fines content (5%, 10%, 15%, 25% and 35%). In the Figure 7.17, all samples prepared at relative density of 60% and run under 15 psi, 30psi of effective consolidation stress. Figure 18 gave the same curve sharp as in the Figure 7.15 and 7.16.
Figure 7.15 Stress ratio require for liquefaction in 10 and 35 cycles versus variable fine content (M-L mixing samples at relative density 30% and confining pressure 15,30 psi) in cyclic triaxial tests. (Jungang Liu, 2019)

Figure 7.16 Stress ratio require for liquefaction in 35 and 50 cycles versus variable fine content (M-L mixing samples at relative density 45% and confining pressure 15,30 psi) in cyclic triaxial tests. (Jungang Liu, 2019)
Figure 7.17 Stress ratio require for liquefaction in 50 and 90 cycles versus variable fine content (M-L mixing samples at relative density 60% and confining pressure 15,30 psi) in cyclic triaxial tests. (Jungang Liu, 2019)

All Soil Samples Results in Cyclic Hollow Cylinder Test

Twenty hollow cylinder tests were performed to investigate the effect of fine contents on the liquefaction resistance of soil.

All samples are the mixture of uniform Monterey No. 0/30 sand with five different percentages of fine content (5%, 10%, 15%, 25% and 35%) and plasticity index 20. All hollow cylinder specimens were prepared at two different relative densities of 30, 60 percent and run under 15 psi, 30 psi of effective consolidation stress.

Figure 7.18 present that the relationship between the stress ratios required reaching liquefaction in 8 cycles and soil with various percent of fines content (5%, 10%, 15%, 25% and 35%). In the Figure 7.18, all samples prepared at relative density of 30% and run under 15 psi, 30 psi of effective consolidation stress. In the cyclic hollow cylinder test, the curve of figure 19 showed the same relationship between stress ratio and fines content as in the cyclic triaxial test. The liquefaction resistance began to decrease with fine content increase until fine content reaches 15%, after that soil got stronger as the fine content increases.
In the Figure 7.19, 7.19, 7.20 and 7.21, they showed that the same results as in the cyclic triaxial test, and proved that the threshold fines content is expected to occur. In this research, it showed that the threshold fines content is 15% on cyclic triaxial and cyclic hollow cylinder tests.

**Figure 7.18** Stress ratio require for liquefaction in 8 cycles versus variable fine content (mixing samples at relative density 30% and confining pressure 15psi) in cyclic hollow cylinder tests. (Jungang Liu, 2019)
Figure 7.19 Stress ratio require for liquefaction in 27 cycles versus variable fine content (mixing samples at relative density 30% and confining pressure 30 psi) in cyclic hollow cylinder tests. (Jungang Liu, 2019)
Figure 7.20 Stress ratio require for liquefaction in 20 cycles versus variable fine content (mixing samples at relative density 60% and confining pressure 15 psi) in cyclic hollow cylinder tests. (Jungang Liu, 2019)
**Figure 7.21** Stress ratio require for liquefaction in 40 cycles versus variable fine content (mixing samples at relative density 60% and confining pressure 30 psi) in cyclic hollow cylinder tests. (Jungang Liu, 2019)
CHAPTER VIII
EXCESS PORE PRESSURE GENERATION

Excess Pore Pressure Generation from Laboratory Test Results

Previous Studies

In the 1970s, Seed et al. (1975b) developed an empirical model for predicting $r_u$ using data from tests performed on clean sands. In their model, $r_u$ is a function of the cycle ratio, which is the ratio of the number of applied uniform cycles of loading ($N$) to the number of cycles required to cause liquefaction in the soil ($N_l$), and an empirically determined parameter $\alpha$

$$r_u = \frac{1}{2} + \frac{1}{\pi} \arcsin \left( 2 \times \left( \frac{N}{N_l} \right)^{\frac{1}{\alpha}} - 1 \right)$$

According to Polito et al. (2008), the value of $\alpha$ is a function of fine content (FC), relative density ($D_r$), and CSR, and cannot be assumed equal to 0.7 for all cases. Booker et al. (1976) proposed an alternative, somewhat simplified version of this equation

$$r_u = \frac{2}{\pi} \arcsin \left( \frac{N}{N_l} \right)^{\frac{1}{2\alpha}}$$

Each of the above equations makes use of two calibration parameters (i.e., $N_l$ and $\alpha$) that can be determined from stress controlled cyclic triaxial tests, as well as other types of undrained cyclic tests.

Nonlinear mixed effect (NLME) models were used in regression analyses to develop correlations for estimating $\alpha$. Others who have used NLME models in regression analyses of geotechnical/earthquake data include Abrahamson and Silva (1996), Liu et al. (2001), and Rathje et al. (2004). Several forms of equations were used in the regression analyses, with the following giving the lowest total standard deviation ($\sigma_{tot}$):

$$\alpha = c_1 \times FC + c_2 \times D_r + c_3 \times CSR + c_4$$

where $D_r$=relative density in percent; $CSR$=cyclic stress ratio; $FC$=fines content in percent; and $c_1$, $c_2$, $c_3$, $c_4$, are regression coefficients (for FC$<35\%$: $c_1=0.01166$; $c_2=0.007397$; $c_3$ =0.01034; and $c_4=0.5058$; and for FC$\geq35\%$: $c_1=0.002149$; $c_2 =-0.0009398$; $c_3=1.667$; and $c_4=0.4285$)

Green et al. (2000) developed the Green-Mitchell-Polito model (GMP) model, which is an empirical expression that relates $r_u$ to the energy dissipated per unit volume of soil (i.e., unit energy). The GMP model is a special case of the more general energy-based model proposed by
Berrill and Davis (1985). The GMP model was developed using data from tests performed on nonplastic silt-sand mixtures that ranged in fines contents from clean sands to pure silts.

The GMP model is

\[ r_u = \sqrt{\frac{W_s}{PEC}} \leq 1 \]

where \( W_s \) = energy dissipated per unit volume of soil divided by the initial effective confining pressure (i.e., normalized unit energy); and PEC = “pseudoenergy capacity,” a calibration parameter.

For general loadings, increments in \( W_s \) can be related to stress conditions and increments in strain equation is

\[ dW_s = \left( \sigma'_v d\varepsilon_v + 2\sigma'_h d\varepsilon_h + \tau_{vh} d\gamma_{vh} + \tau_{hv} d\gamma_{hv} \right) \frac{1}{\sigma'_o} \]

where \( dW_s \) = incremental dissipated energy normalized by the initial effective mean stress; \( \sigma'_v \) = effective vertical stress; \( d\varepsilon_v \) = incremental vertical strain; \( \sigma'_h \) = effective horizontal stress; \( d\varepsilon_h \) = incremental radial strain; \( \tau_{vh} \) = horizontal shear stress acting on a plane having a vertical normal vector; \( d\gamma_{vh} \) = incremental shear strain resulting from \( \tau_{vh} \); \( \tau_{hv} \) = vertical shear stress acting on a plane having a horizontal normal vector; \( d\gamma_{hv} \) = incremental shear strain resulting from \( \tau_{hv} \); and \( \sigma'_o \) = initial effective stress.

For undrained cyclic triaxial test loadings, \( W_s \) can be computed numerically

\[ W_s = \frac{1}{2\sigma'_o} \sum_{i=1}^{n} (\sigma_{d,i+1} + \sigma_{d,i})(\varepsilon_{a,i+1} - \varepsilon_{a,i}) \]

where \( n \) = number of load increments to liquefaction; \( \sigma_{d,i} \) and \( \sigma_{d,i+1} \) = applied deviator stress at load increment \( i \) and \( i+1 \), respectively; and \( \varepsilon_{a,i} \) and \( \varepsilon_{a,i+1} \) = axial strain at load increment \( i \) and \( i+1 \), respectively.

The pseudoenergy capacity (PEC) is determined from cyclic test data by plotting \( r_u \) versus the square root of \( W_s \). The square root of PEC is the value on the horizontal axis corresponding to the intersection of a straight line drawn through the origin and the point of \( ru=0.65 \) and a horizontal line drawn at \( ru=1.0 \).

\[ \ln(\text{PEC}) = \begin{cases} 
\exp(c_3 \cdot D_r) + c_4 & \text{if } FC < 35\% \\
(c_1 \cdot FC)^{c_2} + \exp(c_3 \cdot D_r) + c_4 & \text{if } FC \geq 35\% 
\end{cases} \]

where \( D_r \) = relative density in percent; and \( c_1, c_2, c_3, \) and \( c_4 \) are regression coefficients \((c_1=-0.597; c_2=0.312; c_3=0.0139; \text{ and } c_4=-1.021)\). For this equation and regression coefficients, \( \sigma_{\text{tot}} \ln(\text{PEC})=0.6591 \).
For FC< 35%, PEC increases as Dr increases, and decreases as FC increases. In contrast, for FC≥ 35%, PEC is relatively independent of Dr and FC, increasing slightly as Dr increases and decreasing slightly as FC increases.

Vucetic and Dobry (1986) developed a unique relationship among PWP ratio (ru), cyclic shear strain (γc), and number of loading cycles (Nc) based on the results of undrained, strain-controlled cyclic triaxial compression tests, as:

\[ ru,N = \frac{p f N_c F (\gamma_c - \gamma_{tvp})^s}{1 + f N_c F (\gamma_c - \gamma_{tvp})^s} \]

where \( ru,N \) = residual excess PWP ratio at cycle N; \( f = 1 \) or 2 depending on whether cyclic loading is generated by one- or two-directional loading; \( p, F, \) and \( s \) = curve-fitting constants; and \( \gamma_{tvp} \) = volumetric threshold shear strain, defined as the shear strain threshold below which no significant PWP is generated during cyclic loading. This shear strain falls between 0.01 and 0.02% for most sands (Dobry et al. 1982). Mei et al. (2015) recommended values of \( p = s = 1 \) for the curve-fitting parameters for clean sands. Figure 8.1 presents their proposed correlation between the parameter \( F \) and soil index properties Dr (relative density) and CU (coefficient of uniformity).

Figure 8.1. Correlation to estimate parameter \( F \) in Vucetic-Dobry PWP generation model. (Vucetic and Dobry, 1986)

Martin, Finn and Seed, (1975) based on the compatibility of volume change of soil skeleton and the pore water developed a relation for the pore water pressure increment for each
cycle of shear stress. For saturated sand under an undrained cyclic shearing loading, the total volumetric strain is the sum of the volumetric strains of solid particle and pore water.

\[
\frac{\Delta u}{k_w} = \Delta \varepsilon_{vd} - \frac{\Delta u}{E_r} \quad \text{or} \quad \Delta u = \frac{\Delta \varepsilon_{vd}}{E_r + \frac{1}{k_w}}
\]

where \( \Delta u \) = the increase in residual pore pressure for the cycle; \( k_w \) = bulk modulus of water; \( n_e \) = porosity of sample; and \( E_r \) = tangent modulus of the one-dimensional unloading curve at a point corresponding to the initial vertical effective stress, then considering the unit volume of sand; Change of volume of voids \( = \frac{\Delta u}{n_e} \); reduction in volume of sand structure due to slip deformation \( = \Delta \varepsilon_{vd} \); and increase in volume of sand structure due to recoverable volumetric strain \( = \Delta \varepsilon_{vr} \).

For saturated sample, \( k_w = 4 \times 10^7 \) psf, where \( E_r \) is generally of the order of \( 10^6 \) psf. Considering the relative orders of magnitude of the moduli, the water may be assumed to be effectively incompressible, and thus, under conditions of zero volume change:

\[
\Delta \varepsilon_{vd} = \Delta \varepsilon_{vr} = \frac{\Delta u}{E_r}
\]

Or \( \Delta u = E_r \Delta \varepsilon_{vr} \)

The key to the practical application of the theory rests in the fact that values of \( \Delta \varepsilon_{vd} \) have been found to be independent of vertical stress. Therefore, the theory in its simplest form implies that if a sample of saturated sand loaded to an initial vertical effective stress of \( \sigma_{vo} \) has a recoverable volumetric strain of \( \varepsilon_{vro} \), then liquefaction will occur under an applied cyclic strain history that produces a volumetric strain, \( \varepsilon_{vd} = \varepsilon_{vt} \) under drained conditions.

Ueng, Wu, Lin and Yu (2000) showed that the change of effective stress, or pore water pressure of a saturated sand under the undrained condition, for a given shear strain increment becomes,

\[
\Delta\sigma_n' = \int d\sigma_n' = -\Delta u = \frac{1}{\left(\frac{n}{k_w} + \frac{1}{k_f}\right)} \frac{1}{b} \left[ \frac{\tau}{\sigma_n'} - (\pm \tan \phi) \right] d\gamma
\]

where \( \tau \) = shear stress on the shearing plane
\( \sigma_n' \) = effective normal stress on the shearing plane
\( \phi \) = basic friction angle between sand grains
\( b \) = a positive value related to shearing plan orientation, friction angle, and sand fabric
\( d\gamma \) = shear strain increment, \( n = \) porosity of the sand
\( k_w = \) bulk modulus of pore water
\( k_r = \) rebound modulus of volume change of the sand structure

Chung, Yokel and Wechsler (1984) conducted three torsional resonant column tests on hollow cylindrical specimens. The specimens were prepared to 60% relative density and tested under a 96 kpa confining pressure. The correlation between the volumetric strain, as measured by pore water displacement, and the excess pore water pressure developed is shown in Figure 8.2. Strain controlled cyclic triaxial tests reported by National Research Council (NRC,1985) in Figure 8.3. In the figure 8.3, it indicated that little pore pressure is generated for 10 cycles of strain if the cyclic strain is less than 0.01%. \( \gamma_t = 0.005\% \) gives an upper bound to this data. Based upon the data of these two figures a compromise \( \gamma_t = 0.005\% \) was selected for calibration with the results of load controlled tests.

**Figure 8.2.** Excess Pore Water Pressure ratio vs. Volumetric Strain (Chung, R.M., Yokel, F.Y. and Wechsler, H. 1984)
Figure 8.3 Excess pore water pressure build up Test data from NRC (1985)

Excess Pore Pressure Generation from Laboratory Test Results

A series of isotropically consolidated undrained cyclic triaxial (CTT) and cyclic hollow cylinder tests (CHCT) were conducted to determine excess pore water generation. All samples were prepared at three relative densities of 30%, 45% and 60%, mixed six different percentages of fines contents (5%, 10%, 15%, 25%, 35% and 45%), and tested at different stress ratios and effective consolidation stress.

Excess pore pressure ratio, $R_u$, defined as the ratio of excess pore water pressure, $u$, to initial effective confining stress, $\sigma_3'$. The normalized excess pore water pressure ratio was expressed as $\frac{u}{\sigma_3'}$ and the normalized number of cyclic stress cycle was designated as $\frac{N}{N_l}$. In the figure 8.4 and 8.5, they showed that the relationship between the normalized excess pore pressure ratio and the normalized number of cyclic stress cycle in cyclic triaxial tests and cyclic hollow cylinder tests. In the figure 8.6, it indicated that average excess pore pressure versus normalized number of cyclic stress cycles in all samples of cyclic triaxial tests and cyclic hollow cylinder tests.
Figure 8.4. The normalized excess pore water pressure ratio versus the normalized number of cyclic stress cycle with different percent of fines content and relative densities in cyclic triaxial test. a) relative density at 30% and different of fines content (0%, 5%, 10%, 15%, 25%, 35% and 45%). (Jungang Liu, 2019)
b) Relative density at 45% and different fines content (0%, 5%, 10%, 15%, 25%, and 35%). (Jungang Liu, 2019)

c) Relative density at 60% and different fines content (0%, 5%, 10%, 15%, 25%, and 35%). (Jungang Liu, 2019)
Figure 8.5. The normalized excess pore water pressure ratio versus the normalized number of cyclic stress cycle with different percent of fines content and relative densities in cyclic hollow cylinder test. a) relative density at $30\%$ and different of fines content (0%, 5%, 10%, 15%, 25% and 35%). (Jungang Liu, 2019)
b) relative density at 60% and different fines content (0%, 5%, 10%, 15%, 25% and 35%). (Jungang Liu, 2019)
Figure 8.6. Average excess pore pressure versus normalized number of cyclic stress cycles in all samples a) in cyclic triaxial test with different of fines content (0%, 5%, 10%, 15%, 25%, 35% and 45%). (Jungang Liu, 2019)
b) in cyclic hollow cylinder test with different of fines content (0%, 5%, 10%, 15%, 25% and 35%). (Jungang Liu, 2019)

8.2 Constitutive models for simulating pore pressure generation

**Constitutive Models for Simulating Pore Pressure Generation**

**Constitutive Model UBC3D-PLM**

UBC2D is used for the UBCSAND model, has defined at first by Puebla et al. and then has used in FLAC software by Beaty and Byrne (Puebla et al., 1997, Beaty & Byrne, 1998).

The UBCSAND model is a simple 2D model developed specially for estimate of liquefaction behavior of sand. Also the model has been verified in various applications related to liquefaction. The original 2D model uses a Mohr-Coulomb yield function and a corresponding non associated plastic potential function. The flow rule is based on the well-known Rowe’s stress dilatancy formulation with a modification (Rowe, 1962).

UBCPLM model is based on UBCSAND model which has presented by Anteneh Biru Tsegaye (A.B. Tsegaye, 2010). UBCPLM model, primarily based on the elastoplastic functions mentioned accordingly far a generalized 3D formulation has been considered. The new model
uses the Mohr-Coulomb yield condition in a generalized stress space. The use of non-associated plastic potential based on the same function as the yield function (with mobilized friction angle replaced by mobilized dilatancy angle) has been found to introduce non-coaxially between the stress and the strain in the deviatoric plane.

In 2013, Alexanderos Petalas and Vahid Galavi have introduced UBC3D-PLM code for using in Plaxis (A, Petalas & V, Galavi, 2013). The UBC3D-PLM combination has three aspects: a) UBC model; b) 3- Dimension; c) Past Liquefaction Model.

The undrained behaviour of the soil is treated implicitly by the UBC3D- PLM constitutive model. Therefore, the increment of the pore water pressure is computed at each step of the analysis. Considering a saturated soil specimen, the increments in total stress during loading is given by the following equation:

\[ dp = Ku \, d \varepsilon_v \]

where \( Ku \) is the bulk modulus of the undrained soil and \( d \varepsilon_v \) the volumetric strain of the soil as a whole.

The effective stress increment can be computed as follows:

\[ dp' = K' \, d \varepsilon_v \]

where \( K' \) is the bulk modulus of the soil skeleton and \( d \varepsilon_v \) its volumetric strain.

The increments of the pore water pressure is computed with the following equation:

\[ dp_w = \frac{K_w}{n} \, d \varepsilon_v \]

where \( K_w \) is the bulk modulus of the water, \( n \) is the soil porosity and \( d \varepsilon_v \) is the volumetric strain of the fluid.

The relationship between the total stresses, the effective stresses and the pore pressure is assumed according to Terzaghi’s theory. Moreover, the volumetric compatibility under undrained conditions requires that the equivalent fluid volumetric strain must be equal to the volumetric strain of the soil skeleton.

\[ dp = dp' + dp_w \]

\[ \frac{K_w}{n} = K_u - K' \]

The Poisson’s ratio for undrained condition is set as \( \nu = 0.495 \) implicitly by the model. This value is close to the upper limit (of 0.5) as water is almost incompressible. Using a value of 0.5 is to be avoided as this is known to cause numerical instabilities. Based on this Poisson’s ratio the bulk modulus of the undrained soil is computed as follows:
\[ K_d = \frac{2G^e (1 + \nu_u)}{3(1 - 2\nu_u)} \]

Where \( G^e \) is the elastic shear modulus.

The drained bulk modulus of the soil skeleton \( K' \) is computed in the same way using the drained Poisson’s ratio which is based on the stress dependent stress moduli.

\[ \nu = \frac{3K^e - 2G^e}{6K^e + 2G^e} \]

In the latest version of the UBC3D the bulk modulus of water is dependent with the degree of saturation of the soil.

**Finn Constitutive model**

FLAC software (Fast Lagrangian Analysis of Continua) is a Finite Difference Method-based program (FDM). According to FLAC guidance manual, there are several constitutive models that facilitate soil behavior under static and dynamic loadings (Itasca FLAC manual, 2008). Calculation of excess pore water pressure in the soil mass due to dynamic loading is the main factor in the modeling process of liquefaction phenomenon.

FLAC has a constitutive model named Finn model which equations represented by Martin et al. (1975) and Byrne (1991) into the standard Mohr-Coulomb plasticity model. Using this model, it is possible to calculate pore water pressure generation by calculating irrecoverable volumetric strains during dynamic analysis. The void ratio in this model is supposed to be constant, also it can be calculated as a function of volumetric strain and other parameters can be defined by void ratio (B.R. Khatibi et al, 2012).

Martin et al. (1975) described initially the effect of cyclic loading on increase of pore water pressure as a result of irrecoverable volume contraction in the soil mass. In these situations, because the matrix of grains and voids is filled by water, the pressure of pore water increases (Itasca FLAC manual, 2008).

It supply the following empirical equation that relates the increment of volume decrease “\( \Delta \varepsilon_{vd} \)”, to the cyclic shear-strain amplitude “\( \gamma \)”, where “\( \gamma \)” is presumed to be the “engineering” shear strain (Itasca FLAC manual, 2008):

\[ \Delta \varepsilon_{vd} = C_1 (\gamma - C_2 \varepsilon_{vd}) + (C_3 \varepsilon_{vd}^2)/ (\gamma + C_4 \varepsilon_{vd}) \]

Where \( C_1, C_2, C_3 \) and \( C_4 \) are constants.

Note that the equation involves the accumulated irrecoverable volume strain “\( \varepsilon_{vd} \)”, in such a way that the increment in volume strain decreases as volume strain is accumulated.
Presumably “Δεvd” should be zero if “γ” is zero; this implies that the constants are related as follows: C₁ C₂ C₄ = C₃.

Martin et al. (1975) then go on to compute the change in pore pressure by assuming certain module and boundary conditions ((Itasca FLAC manual, 2008)).

Later, Byrne (1991) presented a simpler equation which correspond irrecoverable volume change and engineering shear strain with two constants. In this model, a soil mass with liquefaction potential was modeled using (N₁)₆₀ parameter as a main factor to the Finn model, so all of the soil properties needed for the model were defined for the program by (N₁)₆₀.

\[
\frac{\Delta \varepsilon_{vd}}{\gamma} = C_1' \exp \left(-C_2' \left(\frac{\varepsilon_{vd}}{\gamma}\right)\right)
\]

Where C₁' and C₂' are constants with different interpretations from above equation. In many cases, C₂' = 0.4C₁', so the above equation involves only one independent constant; however, both C₁' and C₂' have been retained for generality (Itasca FLAC manual, 2008).

As mentioned before, to the usual parameters (friction, module, etc.), the model needs the four constants for C₁, C₂, C₃ and C₄, or two constants for C₁' and C₂'. Martin et al. (1975) describes how four constants C₁, C₂, C₃ and C₄ may be determined from a drained cyclic test. Byrne (1991) notes that the constant, C₁', can be derived from relative densities, “Dr”, as follows:

\[
C_1' = 7600(Dr)^{-2.5}
\]

Further, using an empirical relation between “Dr” and normalized standard penetration test values, “(N₁)₆₀”,

\[
Dr = 15(N₁)₆₀^{1/2}
\]

Then: C₁' = 8.7 (N₁)₆₀⁻¹.25

C₂' is then calculated from C₂' = 0.4C₁' in this case. Note that, as expected, the volumetric strain is larger for smaller values of the blow count (Byrne, 1991).

**Horita’s model**

Horita (1985) showed that the modelling simulated behavior of Monterey No.0/30 sand in a strain-controlled undrained cyclic triaxial test and stress-controlled undrained cyclic triaxial test.

A model for dilatant elasticity material during in undrained conventional triaxial test with volumetric strain increment \(\Delta \varepsilon_{v} = 0\) and radial stress increment \(\Delta \sigma_{r} = 0\) can be expressed as:
\[
\Delta \varepsilon_a = \frac{1}{3k} \Delta p' + \frac{1}{D_1} \Delta q \\
\Delta \varepsilon_r = \frac{1}{3k} \Delta p' + \frac{1}{D_2} \Delta q \\
\Delta p' = \frac{1}{3}(\Delta \sigma_1' + \Delta \sigma_2' + \Delta \sigma_3') \\
\Delta q = \sqrt{\frac{(\Delta \sigma_1' - \Delta \sigma_2')^2 + (\Delta \sigma_1' - \Delta \sigma_3')^2 + (\Delta \sigma_2' - \Delta \sigma_3')^2}{2}} \\
\]

For triaxial conditions, \( \sigma_2' = \Delta \sigma_3' \) and \( \Delta \sigma_2' = \Delta \sigma_3' \)

\[
\Delta \varepsilon_a = \Delta \varepsilon_r = \frac{1}{3}(\Delta \sigma_1' + 2\Delta \sigma_3') \\
\Delta q = (\Delta \sigma_1' - \Delta \sigma_3') \\
\]

Where \( \Delta \varepsilon_a \) and \( \Delta \varepsilon_r \) are axial and radial strain increments, respectively, \( \Delta p' \) and \( \Delta q \) are increments of effective mean stress and deviator stress, \( \Delta \sigma_1' \), \( \Delta \sigma_2' \), and \( \Delta \sigma_3' \) are the principal effective stress increments and \( K, D_1 \) and \( D_2 \) are material constants, \( D_1 = 3G \) and \( D_2 = -6G \), \( K \) is bulk modulus, \( G \) is shear modulus.

If a pair of volumetric strain \( \varepsilon_v \) and shear strain component \( \varepsilon_{\text{shear}} \) are selected for strain measures, the stress-strain relationship becomes

\[
\Delta \varepsilon_v = \frac{1}{k} \Delta p' + \left(\frac{1}{D_1} + \frac{2}{D_2}\right) \Delta q \\
\Delta \varepsilon_a = \frac{2}{3} \left(\frac{1}{D_1} - \frac{1}{D_2}\right) \Delta q \\
\Delta \varepsilon_r = \Delta \varepsilon_a + 2\Delta \varepsilon_r \\
\Delta u = K \left(\frac{1}{D_1} + \frac{2}{D_2}\right) \Delta q + \Delta p \\
\]

Where \( \Delta u \) and \( \Delta p \) are increments in pore water pressure and total mean stress. \( D_1 \) and \( D_2 \) control shear-induced pore water pressure which is not possible in an isotropic elastic material.

\[
F_1 = \frac{\Delta p'}{\Delta q} \quad \text{and} \quad F_2 = \frac{\Delta \varepsilon_a}{\Delta q} \\
\]

\( F_1 \) and \( F_2 \) are found to be constant in the undrained condition as

\[
F_1 = -K \left(\frac{1}{D_1} + \frac{2}{D_2}\right) \\
F_2 = \frac{2}{3} \left(\frac{1}{D_1} - \frac{1}{D_2}\right) \\
\]

Horita (1985) showed that the slope of equi-strain lines gradually increase and converges a critical slope at which the sand shows a large deformation and fails based on running
compression and extension test on Monterey No. 0/30 sand. A parabolic relationship can be adopted to represent the above-mentioned strain-hardening behavior of Monterey No. 0/30 sand as

\[ M - M_0 = \frac{\varepsilon_a}{\alpha + \frac{\varepsilon_a}{M_f - M_0}} \]

Where \( M \) and \( M_0 \) are the slopes of an equi-strain line and the zero-strain line, respectively, \( \varepsilon_a \) is the axial strain and \( M_f \) are materials constant. For compression test, \( M_0 = 0.75, M_f = 1.43 \), and \( \alpha = 0.00306 \), while in extension, \( M_0 = 0, M_f = -1.0 \) and \( \alpha = 0.0015 \).

The equation for equi-strain lines showed in simulation of drained conventional triaxial behavior.

\[ q = (\alpha + p') \cdot M \]

where \( q \) is a deviator stress, \( p' \) is an effective mean stress, \( \alpha \) is the intersection of an equi-strain line with the \( p' \) axis, and \( M \) is the slope of the equi-strain line. The parameter \( \alpha \) is assumed to be a constant equal to -3psi in both compression and extension.

In undrained triaxial compression,

\[ \Delta p' = \Delta p - \Delta u = b_c \varepsilon_a \]

\[ \Delta p' = -\frac{|\varepsilon_a|}{b_c + \frac{|\varepsilon_a|}{U_o}} \cdot p_o' \]

Where \( p_o' \) is the initial effective mean stress, \( \varepsilon_a \) is the axial strain, and \( b_c, b_e \) and \( U_o \) are material constants.

The simulation of undrained triaxial compression behavior requires three fundamental equations: (1) the equation for equi-strain line: \( q = (\alpha + p') \cdot M \) (2) the equation for hardening rule for equi-strain line: \( M = \frac{\varepsilon_a}{\alpha + \frac{\varepsilon_a}{M_f - M_0}} - M_0 \) (3) the equation for shear-induced pore water pressure in compression: \( \Delta p' = \Delta p - \Delta u = b_c \varepsilon_a \).

When it simulated undrained triaxial compression behavior with given stresses, the equations are solved in terms of stresses, and the simulation process by feeding stress increments is summarized as follows:

1) \( \Delta p \) and \( \Delta q \) : given
2) \( q = q_o + \Delta q \)
3) calculate \( \varepsilon_a \): \( \varepsilon_a = \frac{-B + \sqrt{B^2 - 4AC}}{2A} \) where \( A = \frac{b_c M_f}{M_f - M_0} \), \( B = \frac{M_f (\alpha + p'_o) - q}{M_f + M_0} - a M_0 b_c \).
\[ C = a M_o (\alpha + p'_o) - q < 0 \]

4) \( \Delta p' = b_c a \)
5) \( \Delta u = \Delta p - \Delta p' \)
6) \( p' = p'_o + \Delta p' \)

The simulation process of undrained triaxial extension tests by feeding stress increments is summarized as follows:

1) \( \Delta p \) and \( \Delta q \) : given
2) \( q = q_o + \Delta q \)
3) calculate \( e_a \):
\[
\varepsilon_a = \frac{-B-\sqrt{B^2-4AC}}{2A}
\]
where \( A = \frac{q}{U_o M_f} - \left( \frac{\alpha + p'_o}{U_o} - p'_o \right) \)
\[
B = q \left( \frac{b_e}{M_f} + \frac{a}{U_o} \right) - (\alpha + p'_o) g_e
\]
\[ C = a b_c q \]

4) \( \Delta p' = - \frac{p'_o \varepsilon_a}{b + \varepsilon_a U_o} \)
5) \( \Delta u = \Delta p - \Delta p' \)
6) \( p' = p'_o + \Delta p' \)

Horita (1985) run strain-controlled undrained cyclic triaxial test on Monterey No. 0/30 sand to simulate the response of a sand to strain-controlled undrained cyclic loading. The simulation consists of the following four steps: 1) Loading in compression 2) Unloading in compression 3) Loading in extension 4) Unloading in extension. Since in undrained condition no volumetric change occurs during shear, it is assumed that the Poisson’s ratio of an undrained specimen is 0.5. To account for nonlinear equi-strain lines, a nonlinear equation:
\[
q = \frac{A(1+BP')}{Bn} \left[ 1 - (1+BP')^n \right]
\]
where \( A, B, \) and \( n \) are determined from the shape of a equi-strain line as shown in Figure 8.7. The sequential procedure for simulation of a strain-controlled undrained cyclic loading behavior of saturated sand is presented in the flow chart in Figure 8.8.

Horita (1985) showed that the response of Monterey No.0/30 sand in a stress-controlled undrained triaxial test, were conducted at a frequency of 0.5Hz and stress ratio of 0.28, 0.32, and 0.37, can be simulated processes of stress-controlled undrained cyclic triaxial behavior in both compression and extension and the undrained rebound behavior.
As in the simulation of strain-controlled undrained cyclic triaxial behaviors, the equations involved in the simulation of stress-controlled cyclic triaxial behaviors are those for equi-strain lines, hardening rules for equi-strain lines, shear-induced pore water pressure, and undrained rebound behavior. The simulation procedure is summarized in a flow chart in Figure 8.9.

In the table 8.1, it showed that all parameter of trail 1 in Horital model have been used for simulating liquefaction behaviors of Monterey No. 0/30 sand. All parameter of Trail 2 showed in the table 8.2.

All parameter were measured from laboratory tests. In the Figure 8.10-1 and Figure 8.10-2, it showed that the comparison cyclic triaxial test results with simulation by Horita model in excess pore pressure versus number of cycles to liquefaction. Solid curves of figure 8.10-1 and figure 8.10-2 are measured from cyclic triaxial test. Dash curve is simulated by using Horital model in Figure 8.10-1 and Figure 8.10-2. In the trial 1, the simulated and measured curves had a gap, about 3-4 psi. However, in the beginning of number of cycles of the trial 2, the simulated and measured curves matched well. The simulated excess pore pressure from Horita’s model started to have lower value than the measure curve from cyclic triaxial test result after sixth cycle in the trial 2. The different value is less 1 psi between measurement and simulation curves in the trial 2.

Soil sample was saturated Monterey No. 0/30 sand, and run it in the stress-controlled undrained cyclic triaxal test. It was prepared relative density at 30% and run under stress ratio 0.4 and consolidation pressure 15psi, No. cycles to liquefaction =10 by Jungang Liu. The simulation from Horita’s model in generation pore water pressure was followed the chart in Figure 8.9. In the Figure 8.11, it showed that effective stress path (q versus p’) compared between measurement and simulation. Solid curve in Figure 8.11 showed the measurement from cyclic triaxial test, dash curve is simulated by Horita model. The measured and simulated curves as shown in Figure 8.11 matched well.
Figure 8.7 Curved Equi-Strain Line (Horita, 1985)
Figure 8.8 Flow Chart for Simulation on Strain-Controlled Undrained Cyclic Traxial Tests (Horita, 1985)
Figure 8.9 Flow Chart for Simulation on Stress-Controlled Undrained Cyclic Triaxial Tests (Horita, 1985)
### Table 8.1 All parameters in Horita model for trial 1 (Masakuni, Horita, 1985)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Description</th>
<th>Compression</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Intersection of an equi-strain line and the p’ axis in p’-q space</td>
<td>0.00087</td>
<td>0.0044</td>
</tr>
<tr>
<td>Mf</td>
<td>Ultimate slope of equi-strain line</td>
<td>1.12</td>
<td>-1</td>
</tr>
<tr>
<td>A</td>
<td>Parameter for a curved failure line in τ-σn space</td>
<td>2.86</td>
<td>1.47</td>
</tr>
<tr>
<td>B</td>
<td>Parameter for a curved failure line in τ-σn space</td>
<td>0.333</td>
<td>0.333</td>
</tr>
<tr>
<td>b</td>
<td>Parameter for shear-induced pore pressure</td>
<td>27</td>
<td>0.0256</td>
</tr>
<tr>
<td>F1</td>
<td>Undrained rebound moduli</td>
<td>0.27</td>
<td>0.08</td>
</tr>
<tr>
<td>F2</td>
<td>Undrained rebound moduli</td>
<td>0.0000659</td>
<td>0.0000185</td>
</tr>
</tbody>
</table>

**Figure 8.10-1** Comparison between Measured and Simulated Responses of Soil Samples in Generation of Pore Water Pressure. (Jungang Liu, 2019)
Table 8.2 All parameters in Horita model for trial 2 (Masakuni, Horita, 1985)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Description</th>
<th>Compression</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Intersection of an equi-strain line and the p’ axis in p’-q space</td>
<td>0.00067</td>
<td>0.0034</td>
</tr>
<tr>
<td>M_r</td>
<td>Ultimate slope of equi-strain line</td>
<td>1.43</td>
<td>-1.2</td>
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<tr>
<td>A</td>
<td>Parameter for a curved failure line in τ-σn space</td>
<td>2.76</td>
<td>1.475</td>
</tr>
<tr>
<td>B</td>
<td>Parameter for a curved failure line in τ-σn space</td>
<td>0.243</td>
<td>0.344</td>
</tr>
<tr>
<td>b</td>
<td>Parameter for shear-induced pore pressure</td>
<td>26</td>
<td>0.0258</td>
</tr>
<tr>
<td>F_1</td>
<td>Undrained rebound moduli</td>
<td>0.31</td>
<td>0.078</td>
</tr>
<tr>
<td>F_2</td>
<td>Undrained rebound moduli</td>
<td>0.0000662</td>
<td>0.0000175</td>
</tr>
</tbody>
</table>

Figure 8.10-2 Comparison between Measured and Simulated Responses of Soil Samples in Generation of Pore Water Pressure. (Jungang Liu, 2019)
Figure 8.11 Comparison between Measured and Simulated Responses of Saturated Monterey No. 0/30 Sand to Stress-Controlled Undrained Cyclic Loading with stress ratio = 0.4, No. cycles to liquefaction = 10 a) Effective Stress Path (Jungang Liu, 2019)
CHAPTER IX
STATISTICAL MODELING OF LIQUEFACTION RESISTANCE

Introduction

Descriptive statistics and inferential statistics are the two major branches in the science of statistics and statistical methodology. Descriptive statistics deals with summary and description of data and inferential statistics concerns with analysis of sample data to make inferences about a large set of data-a population, from which the sample is selected. Experimental research in engineering involves the use of experimental data - a sample, to infer the nature of same conceptual population that characterizes a phenomenon of interest to the experimenter. One of the most important application of inferential statistics in engineering involves estimating the mean value if a response variable or predicting some future values of the response variable based on knowledge of a set of related independent variables. A relationship used to relate a dependent (response) variable to a set of independent variables is generally referred to as, a regression model or a statistical model (Mendenhall and Sincich, 1991)

Selection of Variables

Initial Variable Selection

In this research, uniform Monterey sand with six different percent of fines content, prepared at three various relative densities and consolidated isotropically at the consolidation pressures of 15 psi and 30 psi. Cyclic triaxial test and cyclic hollow cylinder test were performed at cyclic stress ratios ranging from 0.2 to 0.4 and frequency of 0.5 Hz.

Stress ratio in cyclic triaxial and cyclic hollow cylinder tests causing initial liquefaction in 10 cycles, 30 cycles, 40 cycles and 50 cycles were chosen as dependent variables. The stress ratio causing initial liquefaction were determined from the test results presented in Chapter 4 and 5. Sixteen independent variables: 1 diameter corresponding to 50% finer in the particle-size

**Variable Reduction**

**Introduction**

A total of one hundred and sixteen data were obtained in the variable reduction. Variable reduction is undoubtedly one single most important task in the statistical model building. For the purpose of variable reduction, following procedures may be employed: 1. Examine the scatter diagrams and correlation matrix, 2. Study interaction and confounding terms in regression, 3. avoid collinearity, 4. Try different strategies for selecting variables during regression, 5. Perform factor analysis.

**Examine the Scatter Diagrams and Correlation Coefficient Matrix**

Examination of scatter diagrams of primary independent variables against dependent variables may reveal if high-order terms of primary independent variables are needed. Also, necessity for the transformation of primary independent variables may be implicated. This will help in deciding some interaction terms to be considered initially. Correlation matrix of dependent variables and all independent variables including interaction terms and transformed terms can reveal strength of correlation between variables and help to decide what independent variables to be included in further analysis.
Study Interaction and Confounding Term in Regression

A number of options are available in using statistical testing to evaluate interaction for a given regression model. One approach is to test globally for the pressure of any kind of interaction and then, if significant interaction is found, to identify particular interaction terms of importance by using other test. A second way to assess interaction is to test for interaction in a hierarchical sequence, beginning with highest-order term and proceeding sequentially to lower order terms if higher-order terms are not significant. This should help to decide what interaction terms to be included in the regression. For primary independent variable that are not related to interaction terms, one can start with regression analysis using smaller number of independent variables and consider the rest of independent variables as potential confounders then exclude those which are not confounders to reduce number of variables.

Avoid Collinearity

When several interaction terms are included in a regression model, problem of collinearity may arise. The term collinearity is used to indicate that one of the independent variables is a linear combination of the others. Consequently, avoiding collinearity means reducing number of variables. To assess collinearity, the associated $R^2$-values based on fitting models for each of suspicious independent variables against the rest of independent variables are examined. If any of these multiple $R^2$-values equals 1.0, then a perfect collinearity exists among that particular set of variables. A $R^2 >0.9$ indicates define collinearity problem, while $0.8< R^2<0.9$ usually implies collinearity problem, a $R^2 < 0.8$ usually signals no collinearity problem. Many interaction terms may be excluded through collinearity elimination. The goal of variable reduction may be to eliminate collinearity, to simplify data analysis, or to obtain a parsimonious and conceptually meaningful summary of data. By achieving these goals, reduction of variable is
in turn fulfilled. To determine the out-set of independent variables to avoid collinearity, correlation matrix of all the independent variables can be established and following procedures may also be applied:

1. Delete the single independent variable, $X_j$ with the smallest tolerance ($\text{Tol}_j = 1 - R_{j}^2$), thus defining the out-set of size 1. $R_{j}^2$ is the squared multiple correlation based on regressing $X_j$ against the rest of the independent variables.

2. Within the in-set, delete the single independent variable with the smallest tolerance (this calculation ignores the variable deleted in the previous step).

3. Compute the set of multiple squared correlations based on predicting each out-set variable from the set of in-set variables.

4. If the minimum multiple squared correlation from step 3 is too small, return the last deleted variable to the in-set and stop. Otherwise, deleting exactly one variable each time, repeat steps through 4 until the minimum multiple squared correlation is too small.

Perform Factor Analysis

Factor analysis is a multivariable method intended to explain relationships among several difficult-to-interpret, correlated variables in terms of a few conceptually meaningful, relatively independent factors. Factors analysis may be conducted at the very beginning of the statistical study to help visualizing relationships among all the variables involved in the study. This certainly will help the process of variable reduction even if the results of the factor analysis are not used directly in the subsequent regression analysis. If the factors identified in the factor analysis are to be used in the regression, care must be taken regarding the meaning of each factor which is a weighted linear combination of the original variables. In engineering analysis, variables usually bear some obvious physical meaning. It would be highly desirable if
meaningful physical meaning can be attached to the factors resulted from factor analysis through the involving original variables. Otherwise confusion may arise concerning the physical meaning of the factors used in the regression and the regression results may not be convenient to use.

**Final Variable Selection**

In an effort to reduce variables for soils tested at three various relative densities of 30%, 45% and 60%. The correlation coefficient matrix of the dependent variable, SRCTT10, SRCTT30, SRCTT40, SRCTT50, SRCTT30(Dr30), SRCTT30(Dr45), SRCHCT10, SRCHCT30 and the sixteen chosen primary independent variables as shown in Table 9.1, 9.2, 9.3 9.4, 9.5, 9.6, 9.7, 9.8 were examined. Three independent variables, deviator stress in cyclic triaxial test, DS (cyclic shear stress in cyclic hollow cylinder test, CSS); fine content (decimal), FC; consolidation pressure, CP. were eventually selected for final statistical analysis.
Table 9.1 Correlation coefficient matrix of the dependent variable, SRCTT10 and the sixteen chosen primary independent variables (Jungang Liu, 2019)

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Table 9.2 Correlation coefficient matrix of the dependent variable, SRCTT30 and the sixteen chosen primary independent variables (Jungang Liu, 2019)

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Table 9.3 Correlation coefficient matrix of the dependent variable, SRCTT40 and the sixteen chosen primary independent variables (Jungang Liu, 2019)

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257
Table 9.4 Correlation coefficient matrix of the dependent variable, SRCTT50 and the sixteen chosen primary independent variables (Jungang Liu, 2019)

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Table 9.5 Correlation coefficient matrix of the dependent variable, SRCTT30 (Dr30) and the sixteen chosen primary independent variables (Jungang Liu, 2019)

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Table 9.6 Correlation coefficient matrix of the dependent variable, SRCTT30 (Dr45) and the sixteen chosen primary independent variables (Jungang Liu, 2019)

| Variable                        | US8 | Cu  | Co  | elem | elem | Stress ratio | Void Ratio | Relative Density after | No. cycles | Fine Content (decimal) | Deviator Stress | Effective stress | Void ratio of fines | Overall void ratio | Fines Plasticity index | Void ratio of sand skeleton |
|---------------------------------|-----|-----|-----|------|------|--------------|------------|------------------------|------------|------------------------|----------------|----------------------|-------------------|---------------------|---------------------|------------------------|---------------------------|
| US8                             | 1   | 1   |     |      |      |              |            |                        |            |                        |                 |                      |                   |                     |                     |                         |
| Cu                              | -1  | 1   |     |      |      |              |            |                        |            |                        |                 |                      |                   |                     |                     |                         |
| Co                              |     | -1  | 1   |      |      |              |            |                        |            |                        |                 |                      |                   |                     |                     |                         |
| elem                            |     |     | -1  | 1    |      |              |            |                        |            |                        |                 |                      |                   |                     |                     |                         |
| elem                            |     |     |      | -1   | 1    |              |            |                        |            |                        |                 |                      |                   |                     |                     |                         |
| Stress ratio                    |     |     |      |      |      | 1.0E-16      | -1.0E-16   | -0.0E+00               | 1.0E+00   | 0.1501447              | -1.0E+00       | 0.40896611         | -0.22244          | 0.22244327         | 1                   |                         |
| Void Ratio                      |     |     |      |      |      | 0.0E+00      | 0.0E+00    | 0.0E+00                | 1.0E+00   | -0.1501447             | 1.0E+00         | -0.40896611        | -0.22244          | 0.22244327         | 1                   |                         |
| Relative Density after          |     |     |      |      |      |              |            |                        |            | 0.5168088              | -0.35352         | 0.35655856         | -0.02537288       | 1                   |                     |                         |
| No. cycles                      |     |     |      |      |      | -1.0E-14     | 1.0E-14    | 1.0E-14                | 1.0E-14   | -0.1405467             | 1.0E-14         | -0.49410768        | -0.38708537      | 0.31888965         | 1                   |                         |
| Fine Content (decimal)          |     |     |      |      |      | -2.0E-16     | 2.0E-16    | 2.0E-16                | 2.0E-16   | -0.8228927             | 2.0E-16         | 0.0226212          | -0.0226214       | -0.49410768        | 0.31888965         | 1                   |
| Deviator Stress                 |     |     |      |      |      | 0.0E+00      | 0.0E+00    | 0.0E+00                | 1.0E+00   | 0.5168088              | -0.35352         | 0.35655856         | -0.02537288       | 1                   |                     |                         |
| effective                       |     |     |      |      |      | -1.0E-16     | -1.0E-16   | -0.0E+00               | -1.0E+00  | 0.5168088              | -0.35352         | 0.35655856         | -0.02537288       | 1                   |                     |                         |
| void ratio of fines             |     |     |      |      |      | 0.1501447    | 1.0E+00    | 0.4105467              | 1.0E+00   | -0.1501447             | 1.0E+00         | -0.49410768        | -0.38708537      | 0.31888965         | 1                   |                     |
| overall void ratio              |     |     |      |      |      |              |            |                        |            | 0.0226212              | -0.0226214       | -0.49410768        | -0.38708537      | 0.31888965         | 1                   |                     |
| Fines                           |     |     |      |      |      |              |            |                        |            |                        |                 |                      |                   |                     |                     |                         |
| Plasticity index of sand skeleton |     |     |      |      |      |              |            |                        |            | 0.22244327             | -0.22244327     | -0.3555556          | -0.120302         | 0.0226212          | 0.15752273       | 1                   |
|                                |     |     |      |      |      |              |            |                        |            | 0.22244327             | -0.22244327     | -0.3555556          | -0.120302         | 0.0226212          | 0.15752273       | 1                   |
Table 9.7 Correlation coefficient matrix of the dependent variable, SRCHCT10 and the sixteen chosen primary independent variables (Jungang Liu, 2019)

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Table 9.8 Correlation coefficient matrix of the dependent variable, SRCHCT30 and the sixteen chosen primary independent variables (Jungang Liu, 2019)

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Regression Model for Liquefaction Resistance

Regression model for cyclic triaxial test

A statistical model was formulated to predict the liquefaction resistance of soils containing different fine contents by using cyclic triaxial and cyclic hollow cylinder test results.

For cyclic triaxial test results, stress ratio causing initial liquefaction in 10 cycles, SRCTT10, was chosen as the dependent variable. Through the coefficient of correlation analysis, the independent variables were reduced from the initial sixteen to the final three: deviator stress, DS; fine content (decimal), FC; consolidation pressure, CP.

Using the excel program, linear regression analysis was performed with one dependent variable SRCTT10 and three independent variables. The $R^2$-value for the regression equation is 1.0. A linear regression model involving DS, FC and CP as predictors was obtained as follows:

$$SRCTT10 = 0.4 + 0.033\times DS - 7.03\times10^{-7}\times FC - 0.027\times CP$$

Or $$SRCTT10 = 0.4 + 0.033\times DS - 0.027\times CP$$ (since coefficients of fine content was too small, one of independent variables was omitted.)

For cyclic triaxial test results, stress ratio causing initial liquefaction in 30 cycles, SRCTT30, was chosen as the dependent variable. Through the coefficient of correlation analysis, the independent variables were reduced from the initial sixteen to the final three: deviator stress, DS; fine content (decimal), FC; consolidation pressure, CP.

Using the excel program, linear regression analysis was performed with one dependent variable SRCTT30 and three independent variables. The $R^2$-value for the regression equation is 0.9. A linear regression model involving DS, FC and CP as predictors was obtained as follows:

$$SRCTT30 = 0.305 + 0.02158\times DS + 0.033\times FC - 0.014\times CP$$
For stress ratio causing initial liquefaction in 40 cycles, SRCTT40 as the dependent variable, the same independent variables as it in 10 and 30 cycles, the $R^2$-value for the regression equation is 0.965. A linear regression model involving DS, FC and CP as predictors was obtained as follows:

$$SRCTT40 = 0.2 + 0.033 \times DS + 3.474 \times 10^{-7} \times FC - 0.013 \times CP$$

Or

$$SRCTT40 = 0.2 + 0.033 \times DS - 0.013 \times CP$$ (since coefficients of fine content was too small, one of independent variables was neglected)

Stress ratio causing initial liquefaction in 50 cycles, SRCTT50, and three independent variables. The $R^2$-value for the regression equation is 0.928. A linear regression model involving DS, FC and CP as predictors was obtained as follows:

$$SRCTT50 = 0.3191 + 0.02 \times DS - 0.0746 \times FC - 0.0124 \times CP$$

For soil sample prepared at relative density 30% and 45% in cyclic triaxial test, stress ratio causing initial liquefaction in 30 cycles, SRCTT30(Dr30), SRCTT30(Dr45), were selected as the dependent variable. Through the coefficient of correlation analysis, the independent variables were chosen: deviator stress, DS; fine content, FC; consolidation pressure, CP.

Linear regression analysis was performed with one dependent variable SRCTT30 (Dr30) and three independent variables in data analysis of excel software. The $R^2$-value for the regression equation is 0.973. A linear regression model involving DS, FC and CP as predictors was obtained as follows:

$$SRCTT30 \text{ (Dr30)} = 0.223 + 0.017 \times DS + 0.068 \times FC - 0.008 \times CP$$

Stress ratio causing initial liquefaction in 30 cycles, SRCTT30 (Dr45), and three independent variables. The $R^2$-value for the regression equation is 0.942. A linear regression model involving DS, FC and CP as predictors was obtained as follows:
\[ SRCTT30_{(Dr45)} = 0.2883 + 0.0215 \times DS + 0.1276 \times FC - 0.0139 \times CP \]

**Regression model for cyclic hollow cylinder test**

For cyclic hollow cylinder test results, stress ratio causing initial liquefaction in 10 cycles, SR10, was decided as the dependent variable. The independent variables were picked: cyclic shear stress, CSS; fine content (decimal), FC; consolidation pressure, CP after the coefficient of correlation analysis.

Linear regression analysis was performed with one dependent variable SR10 and three independent variables in the excel program. The \( R^2 \)-value for the regression equation is 0.95. A linear regression model involving CSS, FC and CP as predictors was obtained as follows:

\[ SR10 = 0.345 + 0.0595 \times CSS - 0.001 \times FC - 0.021 \times CP \]

For cyclic hollow cylinder test results, stress ratio causing initial liquefaction in 30 cycles, SR30, was chosen as the dependent variable. Through the coefficient of correlation analysis, CSS, FC and CP were the independent variables. The \( R^2 \)-value for the regression equation is 0.948. A linear regression model involving CSS, FC and CP as predictors was obtained as follows:

\[ SR30 = 0.299 + 0.0358 \times CSS - 0.0133 \times FC - 0.011 \times CP \]
CHAPTER X
A REVISED PROCEDURE FOR EVALUATING LIQUEFACTION RESISTANCE OF SOIL WITH PLASTIC FINES

Introduction
In this study, the effect of fines contents on liquefaction resistance of a uniform Monterey No. 0/30 sand with different fines content was investigated using cyclic triaxial and cyclic hollow cylinder tests. A regression model to predict liquefaction resistance of soils containing fines was formulated. Based on the procedures for evaluating field liquefaction potential of sand deposits as proposed by Seed and Idriss (1981) and Seed et al. (1985), the new findings of this study can be applied to better understand the field evaluation of liquefaction potential of soils containing fines and effectively assess the liquefaction resistance of soils.

Procedure by Seed, et al

Seed and Idriss (1967) use the ratio of the earthquake-induced cyclic stress ratios (CSR) with the cyclic resistance ratios (CRR) of the soil to evaluate the potential of soil liquefaction. The soil’s CRR is usually correlated to an in-situ parameter, such as CPT penetration resistance, SPT blow count, or shear wave velocity, $V_s$. The procedure recommends the following equation for the evaluation of the earthquake-induced cyclic stress ratio (CSR) with an equivalent uniform shear stress of 65% of the maximum shear stress that the soil experienced during the seismic wave propagation:

$$CSR_{\text{M,}\sigma_v} = 0.65 \frac{\tau_{\text{max}}}{\sigma_v'}$$

where $\tau_{\text{max}}$ = maximum earthquake-induced shear stress, $\sigma_v'$ = effective overburden stress at depth, $z$, where the subscript indicate the specific, earthquake magnitude (moment magnitude, $M$) and in-situ effective overburden, $\sigma_v'$. The choice of the reference stress level, 0.65 $\tau_{\text{max}}$ was selected by Seed and Idriss (1967) and has been in use since. The value of $\tau_{\text{max}}$ can be estimated from dynamic response analyses, but such analyses must include a sufficient number of input acceleration time series and adequate site characterization details to be reasonably robust. Alternatively, the maximum shear stress can be estimated using the equation, developed as part of the Seed-Idriss Simplified Liquefaction Procedure, which is expressed as,

$$CSR_{\text{M,}\sigma_v} = 0.65 \frac{\sigma_v}{\sigma_v'} \frac{a_{\text{max}}}{g} r_d$$
where $\sigma_v =$ vertical total stress at depth $z$, $a_{\text{max}}/g =$ maximum horizontal acceleration (as a fraction of gravitational acceleration) at the ground surface, and $r_d =$ shear stress reduction factor that accounts the reduction of shear stress from that of a rigid to flexible soil column in the soil deposit.

Correlation between cyclic shear stress ratio causing liquefaction in the field and normalized corrected $(N_1)_{60}$ standard penetration resistance of sand for the earthquake magnitude of 7.5 was presented as shown in Figure 10.1.

![Figure 10.1](image_url)

**Fig. 10.1** SPT case histories of cohesionless soils with $\text{FC} \geq 35\%$ and the NCEER Workshop (1997) curve and the recommended curves for both clean sand and for FC = 35\% for $M = 7\frac{1}{2}$ and $\sigma'_v = 1$ atm (I.M.Idriss and R.W.Boulanger, 2004)

The stress-based liquefaction analysis framework for soil includes four functions that describe fundamental aspects of dynamic site response, penetration resistance, and soil characteristics and behavior. These four functions, along with the major factors affecting each, are: 1) $r_d = f(\text{depth}; \text{earthquake and ground motion characteristics}; \text{dynamic soil properties})$; 2) $C_N = f(\sigma'_v; \text{DR}; \text{FC})$; 3) $K_s = f(\sigma'_v; \text{DR}; \text{FC})$ (shown in chap 3); 4) MSF = $f(\text{earthquake and ground motion characteristics}; \text{DR}; \text{FC})$ (shown in chap 3)
Idriss (1999), in extending the work of Golesorkhi (1989), performed several hundred parametric site response analyses and concluded that, for the purpose of developing liquefaction evaluation procedures, the parameter $r_d$ could be expressed as,

$$ r_d = \exp \left[ \alpha (z) + \beta (z) \times M \right] $$

$$ \alpha (z) = -1.012 - 1.126 \sin \left( \frac{z}{11.73} + 5.133 \right) $$

$$ \beta (z) = 0.106 + 0.118 \sin \left( \frac{z}{11.28} + 5.142 \right) $$

where $z =$ depth below the ground surface in meters and the arguments inside the sin terms are in radians. Idriss and Boulanger (2010) summarize details regarding the soil profiles and input motions used in developing these equations. The resulting variations of $r_d$ with depth and magnitude are shown in Figure 10.2.

![Figure 10.2. Shear stress reduction factor, rd, relationship (I.M. Idriss and R.W. Boulanger, 2010)](image)

The $C_N$ relationship used was initially developed by Boulanger (2003b) based on: (1) a re-examination of published SPT calibration chamber test data covering $\sigma_v'$ of 0.7 to 5.4 atm (Marcuson and Bieganousky 1977a, 1977b); and (2) results of analyses for $\sigma_v'$ of 0.2 to 20 atm using the cone penetration theory of Salgado et al. (1997a, 1997b) which was shown to produce good agreement with a database of over 400 CPT calibration chamber tests with $\sigma_v'$ up to 7 atm.
Idriss and Boulanger (2003, 2008) subsequently recommended that the $D_R$-dependence of the $C_N$ relationship could be expressed in terms of $q_{c1Ncs}$ or $(N_1)_{60cs}$ as follows:

$$C_N = \left(\frac{P_a}{\sigma'_v}\right)^m \leq 1.7$$

$$m = 1.338 - 0.249(q_{c1Ncs})^{0.264}$$

$$m = 0.784 - 0.0768\sqrt{(N_1)_{60cs}}$$

with $q_{c1Ncs}$ limited to values between 21 and 254 and $(N_1)_{60cs}$ values limited to values ≤46 for use in these expressions.

The values of $C_N$ calculated using this equation are presented in Figure 10.3 (a) for a range of $q_{c1Ncs}$ and $(N_1)_{60cs}$ values and for effective overburden stresses up to 10 atm, and are compared to the Liao and Whitman (1986) relationship in Figure 3 (b) for effective overburden stresses up to 2 atm.

**Figure 10.3.** Overburden correction factor ($C_N$) relationship for CPT and SPT penetration resistances: (a) for $\sigma'_v/P_a = 0 - 10$, and (b) for $\sigma'_v/P_a = 0 – 2$ along with Liao and Whitman's (1986) relationship.
Proposed Procedure Cyclic Resistance Ratio from SPT based case history

SPT-based Case History Database from Idriss and Boulanger

The individual SPT-based liquefaction case histories data and key references are summarized by Idriss and Boulanger (2004, 2008) in Table 10.1. The total number of case histories in the database is 230, of which 115 cases had surface evidence of liquefaction, 112 cases had no surface evidence of liquefaction, and 3 cases were at the margin between liquefaction and no liquefaction.

Idriss and Boulanger (2004) primarily used cases summarized in the databases compiled by Seed et al. (1984) and Cetin et al. (2000, 2004), except that they excluded the Kobe proprietary cases that were listed in Cetin et al. (2004).

The Fear and McRoberts (1995) database was also a helpful reference for many of the case histories. The updated database described in this report incorporates the 44 Kobe proprietary cases which were provided by Professor Kohji Tokimatsu (2010), an additional 26 case histories summarized in Iai et al. (1989), and a small number of other additions. Data from the 1999 Kocaeli and Chi-Chi earthquakes have not yet been incorporated.

Idriss and Boulanger (2004, 2008) also primarily retained the values of critical depth, Nm, $\sigma_v$, $\sigma_v'$, and the product of the correction factors $C_E$, $C_R$, $C_B$ and $C_S$ listed by Seed et al. for the 1984 cases and by Cetin et al. for the 2000 cases.

Based on all 115 cases had surface evidence of liquefaction from Idriss and Boulanger (2004, 2008), it showed that the relationship between corrected SPT blow count ($N_1_{60}$) and various fines content in Figure 10.4. In the Figure 10.4, 115 cases proved that SPT blow count ($N_1_{60}$) small than 30 had very high potential in soil liquefaction. Most of database is that soil had SPT blow count small than 25 with fines content with less than 35%. There are only 11 cases database that soil with fines content from 50% to 90% had liquefied in the Figure 10.4.

In the table 10.2, it showed average of ($N_1_{60}$) and average of cyclic stress ratio (CSR) at different percentages of fines content. There is a relationship between average of ($N_1_{60}$) and different percentages of fines content in cases histories database.

The trend line of Figure 10.5 showed the relationship between fines content with average of ($N_1_{60}$). The $R^2$ of trend line is 0.8673. In the figure 10.5, it indicated clean fine content has the largest ($N_1_{60}$), ($N_1_{60}$) decreases as fines contents increase. Beyond 15%, average of ($N_1_{60}$) increases with a further increase in fines content.
In the Figure 10.6, it indicated that the relationship between averages of earthquake induced cyclic stress ratio (CSR) and average \((N_1)_{60}\) with various fines content (0%, 5%, 10%, 20%). It denoted that the 10 percent of fines content has the lowest average of \((N_1)_{60}\) and smaller average of earthquake induce cyclic stress ratio.

![FC vs (N1)60 from Idriss & Boulanger](image)

**Figure 10.4.** Case Histories from Idriss and Boulanger (all data for soil liquefy): \((N_1)_{60}\) versus FC (%) was created by Jungang Liu (2019).
Figure 10.5. Case Histories from Idriss & Boulanger: Average (N1)60 versus FC (0%-25%) was created by Jungang Liu (2019).
Figure 10.6. Case Histories from Idriss & Boulanger: Average of Earthquake induced Cyclic Stress Ratio (CSR) versus Average (N1)60 with different FC (0%-20%) was created by Jungang Liu (2019).
Table 10.1 Summary of SPT-based liquefaction case history data from Idriss and Boulanger (2004, 2008).

<table>
<thead>
<tr>
<th>Earthquake &amp; site</th>
<th>M</th>
<th>Lq</th>
<th>Avg CNF (m)</th>
<th>CNF (cm)</th>
<th>Avg N</th>
<th>Avg N600</th>
<th>CR</th>
<th>CE</th>
<th>CS</th>
<th>CS</th>
<th>FC (%)</th>
<th>N100s</th>
<th>C10</th>
<th>Kc</th>
<th>SWF</th>
<th>CSR</th>
<th>CSR for M&gt;7.5 (m&lt;1)</th>
<th>Primary source of data</th>
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<tbody>
<tr>
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<td>No</td>
<td>4.1</td>
<td>41.4</td>
<td>4.5</td>
<td>2.2</td>
<td>3.2</td>
<td>3</td>
<td>6</td>
<td>16</td>
<td>0.94</td>
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<td>1.0</td>
<td>1.0</td>
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<td>1.25</td>
<td>Idriss (2008), Seed et al. (1990), Cetin et al. (2004)</td>
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<td>IFAI 1, Tehsil, Pakistan</td>
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<td>No</td>
<td>4.1</td>
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<td>4.5</td>
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<td>3</td>
<td>6</td>
<td>16</td>
<td>0.94</td>
<td>1.69</td>
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<td>Idriss (2008), Seed et al. (1990), Cetin et al. (2004)</td>
</tr>
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<td>1.0</td>
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</tr>
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<td>6</td>
<td>16</td>
<td>0.94</td>
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<td>1.0</td>
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<td>Idriss (2008), Seed et al. (1990), Cetin et al. (2004)</td>
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<td>1.5</td>
<td>1.25</td>
<td>Idriss (2008), Seed et al. (1990), Cetin et al. (2004)</td>
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<tr>
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<td>1.25</td>
<td>Idriss (2008), Seed et al. (1990), Cetin et al. (2004)</td>
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<td>1.5</td>
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<td>Idriss (2008), Seed et al. (1990), Cetin et al. (2004)</td>
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<tr>
<td>IFAI 1, Tehsil, Pakistan</td>
<td>8.1</td>
<td>No</td>
<td>4.1</td>
<td>41.4</td>
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<td>1.0</td>
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<td>1.5</td>
<td>1.25</td>
<td>Idriss (2008), Seed et al. (1990), Cetin et al. (2004)</td>
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Table 10.1 Summary of SPT-based liquefaction case history data from Idriss and Boulanger (2004, 2008) (continue).
Table 10.1 Summary of SPT-based liquefaction case history data from Idriss and Boulanger (2004, 2008) (continue).
Table 10.2 Case histories data from Idriss and Boulanger: \((N_1)_{60}\) and CSR with different Fines Content data were created by Jungang Liu (2019).

<table>
<thead>
<tr>
<th></th>
<th>FC (%)</th>
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</thead>
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<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Total Samples</td>
<td>12</td>
</tr>
<tr>
<td>((N_1)_{60})</td>
<td></td>
</tr>
<tr>
<td>Ave. ((N_1)_{60})</td>
<td>15.63</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>4.92</td>
</tr>
<tr>
<td>CSR</td>
<td></td>
</tr>
<tr>
<td>Ave. CSR</td>
<td>0.30</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.15</td>
</tr>
</tbody>
</table>
SPT-based Cases History Database from Kohji Tokimatsu and Yoshiaki Yoshimi

Many investigators have reported field evidence of soil liquefaction during strong earthquakes of which more than 70 case histories in Japan during 10 earthquakes as well as about 20 supplemental data outside Japan are available as shown in Table 10.3.

Tokimatsu and Yoshimi (1983) also primarily retained the values of critical water stable depth, earthquake magnitude M, maximum acceleration, $\sigma_v$, $\sigma'_v$, SPT blow count $N_1$ (energy rod ratio of 78%), fine content (FC), median grain size ($D_{50}$), coefficient of uniformity (Cu), clay content (CC), gravel content (GC) and calculated cyclic stress ratio (CSR) in Table 10.3.

The individual case histories and key references are summarized in Table 10.3. The total number of case histories in the database is 95, of which 52 cases had surface evidence of liquefaction, 33 cases had no surface evidence of liquefaction, and 10 cases were at the margin between liquefaction and no liquefaction.

In Figure 10.7, it showed that SPT blow count $N_1$ versus different fines content from 0% to 65% from all 52 case database had liquefied. In Figure 10.7, 52 case databases confirmed that it will be liquefied at SPT blow count $N_1$ small than 20. Additional, 80 percent of database is that soil had SPT blow count small than 20 with fines content with less than 35%. There are only 7 cases database that soil with fines content from 40% to 65% had liquefied in the Figure 10.7.

For four different percent of fines content (0%, 5%, 10% and 20%), the average of $N_1$, average (without maximum and minimum) of $N_1$, average of calculating (CSR) and average (without maximum and minimum) of calculating CSR is shown in the Table 10.4.

In the figure 10.8, one set of data is the average of $N_1$ with different percentage of fines content, another data is the average (without maximum and minimum) of $N_1$ with different percentage of fines content. Two sets of data had the similar curve. The curve of figure 8 had the same shape to the track of figure 10.5 from Idriss and Boulanger (2004, 2008). The trend line of figure 10.8 showed the relationship between fines content with average of $N_1$. The $R^2$ of trend line is 0.9503. The clean fine content has the largest values N1 of 13.5 in the figure 10.8. The N1 decreases with increase fines content until it reach 10 percent of fines content. After that, the trend is expected to go up with increase fines content.

In the figure 10.9, it showed that the SPT blow count always has the lowest value at the 10 percent of fines content, and also had the smallest earthquake induced cyclic stress ratio.
From the figure 10.5 and 10.9, both data indicated that the clean fines content has the largest SPT blow count, and also 10% fine content of soil had the lowest SPT blow count.

In the Figure 10.10, two SPT-based cases history databases from Idress, et. and Yoshimi, et. are in one plot. Although two databases had two different threshold fines content (one is 10%, another is 15%), both curves are the same trend showed in the figure 10.10. Two databases showed that the clean fine content always had the largest values of SPT blow count. Case histories from different researchers showed that SPT blow count \( (N_{1})_{60} \) dropped with increasing percentage of fines content. After threshold fines content, the \( (N_{1})_{60} \) started to increase with increasing fines content.

**Figure 10.7.** Case Histories from Tokimatsu & Yoshimi (all data for soil liquefy): N1 versus FC (%) was created by Jungang Liu (2019).
Figure 10.8. Case Histories from Tokimatsu & Yoshimi: Average (N1), average-MM (N1) versus FC (0% - 20%) was created by Jungang Liu (2019).
Figure 10.9. Case Histories from Tokimatsu & Yoshimi: Average of Earthquake induced Cyclic Stress Ratio (CSR) versus Average N₁ with different FC (0%–20%) was created by Jungang Liu (2019).
Figure 10.10. SPT blow count with different fines content (0%-25%) in case histories data from Idriss & Boulanger and Tokimatsu & Yoshimi (all data for soil liquefy) created by Jungang Liu (2019).
Table 10.3 Summary of SPT-based liquefaction case history data from Tokimatsu and Yoshimi (1983).

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Table 10.3 Summary of SPT-based liquefaction case history data from Tokimatsu and Yoshimi (1983) (continue).

<table>
<thead>
<tr>
<th>Earthquake &amp; site</th>
<th>M</th>
<th>GWT</th>
<th>Depth $\sigma$</th>
<th>$\sigma'$</th>
<th>N</th>
<th>N1</th>
<th>$\sigma_{\text{avg}}$</th>
<th>$\sigma_{\text{max}}$</th>
<th>FC</th>
<th>D50</th>
<th>Cu</th>
<th>CC</th>
<th>GC</th>
<th>Liq</th>
<th>Primary source of data</th>
</tr>
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<td>7</td>
<td>1.8</td>
<td>3.3</td>
<td>0.63</td>
<td>0.42</td>
<td>10</td>
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<td>2</td>
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<td>No(t)</td>
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<td>0.51</td>
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<td>No(t)</td>
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<td>No(t)</td>
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<td>15.6</td>
<td>0.24</td>
<td>0.245</td>
<td>0</td>
<td>0.4</td>
<td>Yes (a)</td>
<td>No(t)</td>
<td>No(t)</td>
<td>No(t)</td>
</tr>
<tr>
<td>Ofunato (06/12/1978)</td>
<td>7</td>
<td>1.8</td>
<td>5.0</td>
<td>0.95</td>
<td>0.51</td>
<td>3</td>
<td>2.5</td>
<td>0.24</td>
<td>0.324</td>
<td>60</td>
<td>0.04</td>
<td>Yes (a)</td>
<td>No(t)</td>
<td>No(t)</td>
<td>No(t)</td>
</tr>
<tr>
<td>Ofunato (06/12/1978)</td>
<td>7</td>
<td>1.8</td>
<td>4.3</td>
<td>0.82</td>
<td>0.48</td>
<td>13</td>
<td>15.6</td>
<td>0.24</td>
<td>0.245</td>
<td>0</td>
<td>0.4</td>
<td>Yes (a)</td>
<td>No(t)</td>
<td>No(t)</td>
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<td>2.5</td>
<td>0.24</td>
<td>0.324</td>
<td>60</td>
<td>0.04</td>
<td>Yes (a)</td>
<td>No(t)</td>
<td>No(t)</td>
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<td>13</td>
<td>15.6</td>
<td>0.24</td>
<td>0.245</td>
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<td>0.4</td>
<td>Yes (a)</td>
<td>No(t)</td>
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<td>3</td>
<td>2.5</td>
<td>0.24</td>
<td>0.324</td>
<td>60</td>
<td>0.04</td>
<td>Yes (a)</td>
<td>No(t)</td>
<td>No(t)</td>
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<td>Ofunato (06/12/1978)</td>
<td>7</td>
<td>1.8</td>
<td>4.3</td>
<td>0.82</td>
<td>0.48</td>
<td>13</td>
<td>15.6</td>
<td>0.24</td>
<td>0.245</td>
<td>0</td>
<td>0.4</td>
<td>Yes (a)</td>
<td>No(t)</td>
<td>No(t)</td>
<td>No(t)</td>
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<td>Ofunato (06/12/1978)</td>
<td>7</td>
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<td>5.0</td>
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<td>0.51</td>
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<td>2.5</td>
<td>0.24</td>
<td>0.324</td>
<td>60</td>
<td>0.04</td>
<td>Yes (a)</td>
<td>No(t)</td>
<td>No(t)</td>
<td>No(t)</td>
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<tr>
<td>Ofunato (06/12/1978)</td>
<td>7</td>
<td>1.8</td>
<td>4.3</td>
<td>0.82</td>
<td>0.48</td>
<td>13</td>
<td>15.6</td>
<td>0.24</td>
<td>0.245</td>
<td>0</td>
<td>0.4</td>
<td>Yes (a)</td>
<td>No(t)</td>
<td>No(t)</td>
<td>No(t)</td>
</tr>
</tbody>
</table>
Table 10.4 Case histories data from Idriss and Boulanger: $N_1$ and CSR with different Fines Content data were created by Jungang Liu (2019).

<table>
<thead>
<tr>
<th>FC (%)</th>
<th>0</th>
<th>0+--5</th>
<th>5+--10</th>
<th>10+--20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Samples</td>
<td>6</td>
<td>8</td>
<td>8</td>
<td>3</td>
</tr>
<tr>
<td>$N_1$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ave. $N_1$</td>
<td>13.40</td>
<td>10.88</td>
<td>7.13</td>
<td>8.37</td>
</tr>
<tr>
<td>Ave without Max Min</td>
<td>12.88</td>
<td>10.65</td>
<td>7.53</td>
<td></td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>4.03</td>
<td>2.69</td>
<td>3.10</td>
<td>2.20</td>
</tr>
<tr>
<td>Calculate CSR</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ave. CSR</td>
<td>0.18</td>
<td>0.14</td>
<td>0.13</td>
<td>0.15</td>
</tr>
<tr>
<td>Ave without Max Min</td>
<td>0.18</td>
<td>0.14</td>
<td>0.13</td>
<td></td>
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<tr>
<td>Standard Deviation</td>
<td>0.05</td>
<td>0.05</td>
<td>0.02</td>
<td>0.03</td>
</tr>
</tbody>
</table>
Cyclic Resistance Ratio from SPT based laboratory tests data

Calculating SPT Blow Count \((N_1)_{60}\) Procedure

Several early investigations were focused on the relationship between relative density and standard penetration resistance. Research regarding such relationship has been conducted (Gibbs and Holtz, 1957; Meyerhof, 1957; Skempton, 1986; Schultze & Menzenbach, 1961; M.Cubrinovshi & K.Ishihara, 2001). In this study, it showed that the relationship between soil sample properties from laboratory tests such as relative density, confining stress, fines contents, and standard penetration resistance from field tests.

There are six steps to calculate \((N_1)_{60}\) using soil sample properties from lab tests.

Step 1: Calculate SPT blow count.

a) For clean sand,

Meyerhof (1957), the penetration resistance is assumed to increase with the square of the relative density and be in direct proportion to the effective overburden pressure of the sand

\[
N = (17 + 24 \frac{\sigma'_v}{98}) D_r^2
\]

where \(N\) is the SPT blow count, \(\sigma'_v\) is the effective overburden pressure in kPa and \(D_r\) is the relative density expressed as a ratio, not a percentage.

Skempton (1986) expressed in a general form

\[
N = (a + b \frac{\sigma'_v}{98}) D_r^2
\]

Substituting \(\sigma'_v = 98\) kPa, the expression is reduced to

\[
\frac{N_1}{D_r^2} = a + b
\]

where \(N_1\) is the normalized penetration resistance to an overburden pressure of 98 kPa, i.e. 1 kgf/cm\(^2\). Note that in the original definition of Meyerhof the ratio \(\frac{N_1}{D_r^2}\) or the parameter \(a + b\) was assumed to take a value of 41.

Schultze & Menzenbach (1961). Their data although presented as a logarithmic relationship, fit closely to the equation

\[
\frac{N}{D_r^2} = 17 + 22\sigma'_v
\]
b) For soil containing fines

For each undisturbed sample, the limiting void ratios ($e_{\text{max}}$, $e_{\text{min}}$) and relative density were evaluated by standard laboratory test procedures while the N1 value corresponding to the undisturbed sample was calculated from the known SPT blow count and sampling depth. The following empirical correlation by M.Cubrinovshi & K.Ishihara (2001) between the SPT N value and $D_r$ is derived:

$$N = \frac{9D_r^2}{(e_{\text{max}}-e_{\text{min}})^{3.7}} \left(\frac{\sigma'_v}{98}\right)^{1.2}$$

where $D_r$ is defined as a ratio and $\sigma'_v$ is given in kPa. It is important to note that, in this expression, the SPT blow count corresponds to an energy rod ratio of about 78% of the theoretical free-fall energy.

Step 2 (only for soil containing fines): Relationship between $e_{\text{max}}$, $e_{\text{min}}$, and Fines Content

Misko Cubrinovski and Kenji Ishihara (2002) showed the relationship between void ratio range and fines content, $(e_{\text{max}}-e_{\text{min}})$ is plotted against fines content in Figure 10.11. A regression equation by Misko Cubrinovski and Kenji Ishihara (2002) relating maximum void ratio $e_{\text{max}}$, minimum void ratio $e_{\text{min}}$, and fines content, FC is following equation in the below.

$$(e_{\text{max}}-e_{\text{min}}) = 0.43+0.0086*FC, \quad 0<FC<30\%$$

$$(e_{\text{max}}-e_{\text{min}}) = 0.57+0.004*FC, \quad 30 \leq FC \leq 75\%$$

**Figure 10.11.** Relationship between void ratio range and fines content of sandy soils (Misko Cubrinovshi & Kenji. Ishihara, 2002)
Step 3: Calculate $N_{60}$

$N_{60}$ can be converted by $N$ values determined from the first step with a known or estimated ERr. Table 10.5 showed that four countries have different rod energy ratios and hammers.

$$N_{60} = N \frac{ERr}{60}$$

Table 10.5 Summary of Rod Energy Ratios (Skempton, A.W. 1986)

<table>
<thead>
<tr>
<th>Country</th>
<th>Hammer</th>
<th>Release</th>
<th>ERr: %</th>
<th>ERr/60</th>
</tr>
</thead>
<tbody>
<tr>
<td>Japan</td>
<td>Donut</td>
<td>Tombi</td>
<td>78</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>Donut</td>
<td>2 turns of rope</td>
<td>65</td>
<td>1.1</td>
</tr>
<tr>
<td>China</td>
<td>Pilcon type</td>
<td>Trip</td>
<td>60</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Donut</td>
<td>Manual</td>
<td>55</td>
<td>0.9</td>
</tr>
<tr>
<td>USA</td>
<td>Safety</td>
<td>2 turns of rope</td>
<td>55</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>Donut</td>
<td>2 turns of rope</td>
<td>45</td>
<td>0.75</td>
</tr>
<tr>
<td>UK</td>
<td>Pilcon, Dando, old standard</td>
<td>Trip</td>
<td>60</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 turns of rope</td>
<td>50</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Step 4: Calculate $(N_1)_{60}$

In the field and laboratory tests described each of the sands (with one exception) is sufficiently uniform with regard to grain size and relative density to be treated as a unit, and the below count at $\sigma_v' = 1 \text{ ton/ft}^2$ is found by direct interpolation. In general, however, it is necessary to be able to estimate the $N_1$ value for any particular test, and this is done by means of the formula:

$$N_1 = C_N N$$

The corresponding limits for $C_N$ are

$$C_N = \frac{2}{1 + \sigma_v'}$$

Peck, Hanson & Thornburn (1974)

$$C_N = 0.77 \log \left( \frac{20}{\sigma_v'} \right)$$

Liao and Whitman (1986) presented the currently held overburden correction, termed $(N_1)_{60}$. The $(N_1)_{60}$ blow count is given as:
\[ (N_1)_{60} = N_{60} \sqrt{\frac{2000 \text{psi}}{\sigma_z'}} \]

where \( \sigma_z' \) is vertical effective stress where the sample was recovered.

**Calculation of SPT Blow Count based Laboratory Test Data**

In this study, one hundred fourteen cyclic triaxial and thirty seven cyclic hollow cylinder tests were performed to calculate SPT blow count \((N_1)_{60}\) by following the procedure in 10.3.2.1. After calculating \((N_1)_{60}\) using soil sample properties from laboratory tests, it showed that the relationship between standard penetration resistance and fines content.

**Calculation of SPT blow count based cyclic triaxial test data**

Ninety-six cyclic triaxial tests were performed on the uniform Monterey No. 0/30 sand with six different percentages of fine content (5%, 10%, 15%, 25%, 35% and 45%) and plasticity index 20. Eighteen cyclic triaxal tests were run on the uniform Monterey No.0/30 clean sand.

In Figure 10.11, it indicated that the relationship between SPT blow count \((N_1)_{60}\) and different percentage of fines content in cyclic triaxial tests. In cyclic triaxial test, soil samples prepared at three different relative densities of 30%, 45% and 60%, and also included six different percentage of fines content. Although three different relative densities, the SPT blow count \((N_1)_{60}\) had the biggest standard penetration resistance under clean sand. The \((N_1)_{60}\) decreases with increase percentage of fines content and increases with increasing percentage of relative densities.

In figure 10.12, soil sample, prepared at 30% relative density in cyclic triaxal test, had the biggest SPT blow count \((N_1)_{60}\) when zero percentage of fine content. The SPT blow count \((N_1)_{60}\) decrease with increase percentage of fines content until reach threshold fines contents at 25%. After 25% of fines content, SPT blow count \((N_1)_{60}\) started to go up at 35% of fines content.
In the figure 10.13 and 10.14, it showed that SPT blow count \((N_1)_{60}\) decreased with increase percentage of fines content under relative densities at 45% and 60% in cyclic triaxial test.

In the figure 10.15, it showed that calculating \((N_1)_{60}\) versus cyclic stress ratio in cyclic triaxial test on the relative density of 30% under consolidation pressure of 15psi with different fines content. Three different cyclic stress ratios of 0.2, 0.3 and 0.4 were applied in cyclic triaxial tests. The clean sand had the largest SPT blow count \((N_1)_{60}\) under three different cyclic stress ratios and consolidation pressure 15psi. The line of 25% of fines content located in the right side of curve in the figure 10.9. However the line of fines content 35% found between the line of fine content 15% and the line of fine content 25%. It mean the lowest SPT blow count \((N_1)_{60}\) existed in fine content of 25% under three cyclic stress ratios.

In the figure 10.16, it showed that calculating \((N_1)_{60}\) versus cyclic stress ratio in cyclic triaxial test on the relative density of 30% under consolidation pressure of 30psi with different fines content. All lines of different fines content sited in similar to the figure 10.9. Fine content of 25% had the lowest SPT blow count \((N_1)_{60}\) in cyclic triaxial test. In the figure 10.17 and 10.18, it showed that SPT blow count \((N_1)_{60}\) versus cyclic stress ratio on the relative density of 45% under consolidation pressure of 15 psi and 30psi with different fines content. In figure 10.17 and 10.18, fine content of 5% always located in the right side of the curves, and gave the largest \((N_1)_{60}\). Both figures showed that SPT blow count \((N_1)_{60}\) dropped with increase percentage of fines content. They indicated that SPT blow count \((N_1)_{60}\) increased with increase relative density.

In the figure 10.19 and 10.20, it showed that SPT blow count \((N_1)_{60}\) versus cyclic stress ratio on the relative density of 60% under consolidation pressure of 15 psi and 30psi with
different fines content. Both figures gave the same results from soil sample under relative densities of 30% and 45%.
Figure 10.11 Cyclic Triaxial test results for calculating \((N1)_{60}\) From \(FC=0\%-35\%\), prepared at \(Dr= 30\%, 45\%\) and \(60\%\). (Jungang Liu, 2019).
Figure 10.12. Cyclic Triaxial test results for calculating (N1)60 From FC=0%-35%, prepared at Dr= 30% (Jungang Liu, 2019)
Figure 10.13. Cyclic Triaxial test results for calculating (N1)60 From FC=5%-35%, prepared at Dr= 45% (Jungang Liu, 2019).
Figure 10.14. Cyclic Triaxial test results for calculating (N1)60 From FC=0%-35%, prepared at Dr= 60% (Jungang Liu, 2019).
**Figure 10.15** Calculating $(N_1)_{60}$ versus cyclic stress ratio from cyclic triaxial test on relative density of 30% under consolidation pressure 15psi with different fines content. (Jungang Liu, 2019).
Figure 10.16 Calculating (N₁)₆₀ versus cyclic stress ratio in cyclic triaxial test on the relative density of 30% under consolidation pressure of 30psi with different fines content (Jungang Liu, 2019).
Figure 10.17 Calculating $(N_1)_{60}$ versus cyclic stress ratio in cyclic triaxial test on the relative density of 45% under consolidation pressure of 15 psi with different fines content (Jungang Liu, 2019).
Figure 10.18. calculating $(N_1)_{60}$ versus cyclic stress ratio in cyclic triaxial test on the relative density of 45% under consolidation pressure of 30 psi with different fines content (Jungang Liu, 2019).
Figure 10.19 Calculating $(N_1)_{60}$ versus cyclic stress ratio in cyclic triaxial test on the relative density of 60% under consolidation pressure of 15 psi with different fines content (Jungang Liu, 2019).
Figure 10.20 Calculating (N1)60 versus cyclic stress ratio in cyclic triaxial test on the relative density of 60% under consolidation pressure of 30 psi with different fines content (Jungang Liu, 2019).
Calculation of SPT blow bount based cyclic hollow cylinder test data

Twenty cyclic hollow cylinder tests were performed on the uniform Monterey No. 0/30 sand with five different percentages of fine content (5%, 10%, 15%, 25% and 35%) and plasticity index 20. Seventeen cyclic hollow cylinder tests were run on the uniform Monterey No.0/30 clean sand.

Calculating SPT blow count \((N_1)_{60}\) from cyclic hollow cylinder test results, it indicated that the relationship between \((N_1)_{60}\) and different percentage of fines content. In figure 10.21, the trend is similar to the cyclic triaxial test under relative density at 30%. In figure 10.22, the SPT blow count \((N_1)_{60}\) had the largest values about 20 at 0% of fine content. The value of SPT blow count \((N_1)_{60}\) start to drop after increasing fines content. Figure 10.23 showed that two curves is the same trend to in cyclic triaxial test results under two different relative densities.

The trend of figure 10.24, it showed that the relationship between \((N_1)_{60}\), calculated from both cyclic triaxial and cyclic hollow cylinder test results, with different percentage of fines content. The \(R^2\) of trend line is 0.9531. In the Figure 10.24, soil samples prepared at Dr=30% with different percentage of fines content, run under consolidation pressure 15 psis and 30 psi on both laboratory tests. Although two database are from two different laboratory testing results, both curves are the same trend showed in the figure10.24. Both databases indicated that clean fines content had the biggest value of calculated \((N_1)_{60}\). Both curves showed that the \((N_1)_{60}\) is dropping with increasing fines content, but after passing fines content of 25%, SPT blow count starts to increase.

The \(R^2\) of trend line in figure10.25 is 0.9669. In the Figure10.25, soil samples prepared at Dr=60% with different percentage of fines content, run under consolidation pressure 15 psis and 30 psi in both cyclic triaxial and cyclic hollow cylinder test. Although two database are from two different laboratory testing results and different percentage of relative density, both curves are the same trend showed in the figure10.24 and10.25. Both databases indicated that clean fines content had the biggest value of calculated \((N_1)_{60}\). Both curves showed that the \((N_1)_{60}\) is dropping with increasing fines content, but after passing fines content of 25%, SPT blow count starts to increase.
Figure 10.21 Cyclic Hollow Cylinder test results for calculating (N1)60 from FC=0%-35%, prepared at Dr=30% (Jungang Liu, 2019).
Figure 10.22 Cyclic Hollow Cylinder test results for calculating (N1)60 From FC = 0%-35%, prepared at Dr= 60% (Jungang Liu, 2019).
Figure 10.23 Cyclic Hollow Cylinder test results for calculating $(N1)_{60}$ from FC=$0\%-35\%$, prepared at Dr= 30% and 60% (Jungang Liu, 2019).
Figure 10.24 Cyclic Triaxial test and Cyclic Hollow Cylinder test results for calculating (N1)60 from FC=0%–35%, prepared at Dr=30% (Jungang Liu, 2019).
Figure 10.25 Cyclic Triaxial test and Cyclic Hollow Cylinder test results for calculating (N1)60 From FC=0%–35%, prepared at Dr=60% (Jungang Liu, 2019).
Comment on the New Procedure

In this research, the field case histories data from Idriss et al. (2004, 2008) and Youshimi et al. (1983) indicated that the soil with no fine content had the largest value of field test results SPT below counts in the figure 10.26. SPT below count, from field case histories, began to drop with increase percentage of fine content, but after passing threshold fines content, SPT below count tend to get bigger.

Based entirely on laboratory test results, it also showed that SPT blow count decreases with increase fines content until threshold fines content. In the figure 10.26, it showed that four different SPT below counts, two from field case histories and another two from calculating SPT below count based on laboratory test results, had the largest value when soil had no fines content. Threshold fines content existed range from 10% to 25% on SPT-based case histories in the figure 10.26. In laboratory test results in CTT and CHCT on Dr=30% & 60% soil samples, SPT blow count decreases with increase fines content until threshold fines content, ranges from 25% to 35% in figure 10.26.

In the figure 10.26, it showed that soils containing fines content from 10%-15% had the lowest number of SPT below count. For evaluating field liquefaction potential for soils containing fines content, the figure 10.26 should be recommended. The results of the procedure should be applied to the evaluation of liquefaction resistance of soil containing fines content from 5% to 25%. Specially, soil with different percentage of fines content should be paid more attention for evaluating liquefaction potentials.
Figure 10.26. Calculating $(N_1)_{60}$ from cyclic triaxial and cyclic hollow cylinder tests on relative density 30\% & 60\% and consolidation pressure 15psi & 30psi, $(N_1)_{60}$ from case histories versus different fines content (Jungang Liu, 2019).
CHAPTER XI
SUMMARY, CONCLUSION AND RECOMMENDATIONS FOR FUTURE STUDIES

Summary

In this study, existing methods for liquefaction potential analysis and factors affecting liquefaction resistance of soils were reviewed. One hundred fourteen isotopically consolidated undrain cyclic triaxial tests and thirty-seven cyclic hollow cylinder test were performed to investigate the effect of fines content on liquefaction resistance of soils. A uniform Monterey No.0/30 sand and Leyden clay (was sieved through a #200 sieve to remove any impurities) were involved. Fines with different consistency were prepared by mixing a sand and a silt at different percentage of composition.

To compare and relate the soil liquefaction resistance found by cyclic triaxial and cyclic hollow cylinder test results on uniform clean Monterey No.0/30 sand and soil sample with different percentage of fines content.

The cyclic triaxial and cyclic hollow cylinder liquefaction resistance of soils were expressed in terms of liquefaction potential curves. From the liquefaction potential curves, stress ratio in cyclic triaxial and cyclic hollow cylinder tests causing initial liquefaction in 10 cycles, 30 cycles, 40 cycles and 50 cycles were chosen as dependent variables in the regression model. Three independent variables of regression models, deviator stress in cyclic triaxial test, DS (cyclic shear stress in cyclic hollow cylinder test, CSS); fine content (decimal), FC; consolidation pressure, CP. were eventually selected for final statistical analysis.

In addition to the evaluation of liquefaction potential, an excess pore pressure generation was performed from laboratory test results. Comparison of excess pore pressure generation between simulations from Horita’s constitutive model and measured from cyclic triaxial test is
included in this research. The new findings of this study can be applied to better understand the field evaluation of liquefaction potential of soils containing fines and effectively assess the liquefaction resistance of soils.

**Conclusions**

In this study, there are some following conclusions may be drawn:

1. The average of correction factor $C_r'$ is 0.52 between cyclic triaxial and hollow cylinder test at the same relative densities of 30 percent and 60 percent of uniform clean sand.

2. The average of correction factor $C_r'$ is 0.557 between cyclic triaxial and hollow cylinder test at Dr =30% and 60% of samples with different fines.

3. At a constant overall void ratio and PI, the increase in fines content up to 15% in the clean sand causes a decrease in liquefaction resistance. Beyond 15%, the further increases in fines content results in increase in liquefaction resistance. 15% is the threshold fines content observed from laboratory test results in this study.

4. Threshold Fine Content is expected to occur when sand-dominated behavior passes to fine dominated behavior. In this study, threshold fine content is approximately 15% based on cyclic triaxial and hollow cylinder test results.

5. Liquefaction resistance of soils containing fines can be predicted by regression model which involves SRCTT10, SRCTT30, SRCTT40, SRCTT50, SRCTT30(Dr30), SRCHCT10, SRCHCT30 as indicators of liquefaction resistance while DS (CSS), FC and CP as indicators of cyclic stress, fines content, and consolidation pressure.

6. Generation of excess pore water pressure due to cyclic stress is greatly affected by the fines content.
7. Horita’s constitutive model can predict generation of excess pore water pressure due to cyclic stress.

8. After studying case histories databases and lab test results, the clean sand had the biggest number of SPT blow count, and also SPT blow count decreased with fines content increased. After passing threshold fines content, SPT blow count started to increase. For evaluating field liquefaction potential for soils containing fines content, soil with 10% - 30% of fines content should be paid more attention.

**Recommendation for Future Studies**

In this study, there are some recommendations showed in below:

1. Perform SPT tests in large model test device “Tiger Cage”.

2. Perform cyclic triaxial and hollow cylinder test on undistributed field soil samples.

3. Evaluating threshold fine content on different soil types in laboratory tests.

4. For evaluating field liquefaction potential for soils containing fines content, Figure 10.26 need to be add more different soil types. It can help engineerings better understand the field evaluation of liquefaction potential of soils containing fines and effectively assess the liquefaction resistance of soils.
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APPENDIX A

A. Field Methods for Soil Liquefaction Resistance Evaluation

A.1. Other Techniques

A.1.1 Becker Penetration Test and Large Penetration Test

Becker penetration test (BPT) and large penetration test (LPT) have been used in soils with large particles (e.g., gravels and cobbles) that can interfere with the accuracy of SPTs and CPTs or even preclude their use. The BPT uses a double-acting diesel pile hammer to drive into the ground a 168-mm-diameter, 3-m-long double-walled casing with a closed bit. The BPT test provides a continuous driving record, from which the blow count is the number of hammer blows required to drive the casing each 300 mm (1 ft) into the ground. The LPT is similar to an SPT, except that it uses a larger split-spoon sampler and a larger hammer to drive it.

The BPT depends on a number of factors that affect the energy delivered to the casing tip, including the diesel hammer’s energy efficiency and the friction along the entire casing. Correlations between BPT and SPT values in sand deposits are used to convert BPT blow counts into equivalent SPT N values for use in liquefaction analyses. Attempts have been made to further standardize the BPT and better understand its mechanics, but significant concerns remain about its repeatability and general interpretation. More details about the BPT procedures and related issues are provided by Harder (1997) and Sy (1997).

Several different LPTs have been developed around the world that have sampler outer diameters of 7.3–14 cm (as compared with 5.1 cm for the SPT) and hammer potential energies of 1.2–5.9 times the potential energy for the SPT hammer. Penetration resistances from LPTs have been correlated with those from SPTs, so the LPT values can be converted into equivalent SPT \( N_{60} \) values for use in liquefaction evaluations. Daniel et al. (2003) showed that wave equation...
analyses of the different penetration tests provided a rational means for assimilating various empirical LPT-SPT correlations. They further noted the importance of energy measurements for obtaining reliable LPT penetration resistances.

A.1.2 Seismic Wave Velocity

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).

Measured shear wave velocities can be normalized to a standard effective overburden pressure of 1 ton/ft² (96 kPa) by

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

(2.1)

where \( \sigma'_v \) is in tons/ft² 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 2.27). 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 2.28).

The observation that the shear wave velocity of sand is insensitive to factors (e.g., soil fabric, over consolidation 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, $G_{\text{max}}$, 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 2.29).
Figure 2.27 Chart for evaluation of liquefaction potential from shear wave velocity and peak ground acceleration (0 cycles). (After Stokoe et al., 1988.)

Figure 2.28 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.)
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

\( C_v \) = correction factor to account for overburden pressure. \( C_v = \left(\frac{100}{\sigma_{vo}'}\right)^{0.25} \)

\( \sigma_{vo} ' \) = vertical effective stress kPa

\( V_s \) = shear wave velocity measured in field
Figure 2.30 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 2.30 showed that the corrected shear wave velocity $V_{s1}$, and then by intersecting the appropriate fines content curve, the cyclic resistance ratio is obtained.

The first is the global $V_s$ database presented by Kayen et al. (2004). Fig.2.31 presents 60% of the global $V_{s1}$ data and the corresponding probability curves, and the present lower-bound CRR--$V_{s1}$ curve and that of Andrus and Stokoe (2000) are also plotted. The present curve separates all the liquefied case data properly in a slightly conservative way, and approximately corresponds to that of $PL=0.05$ in the range of $100 \text{ m/s} < V_{s1} < 200 \text{ m/s}$ where most of the data concentrate, and to that of $PL=0.2$ when $V_{s1} > 200 \text{ m/s}$. 
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. 2.30, 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.
Figure 2.29 Chart to predict liquefaction in clean sands from shear wave velocity and maximum acceleration (Stokoe, et al. 1988)
A.1.3. Other Techniques

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.
APPENDIX B

B. Factors Affecting Liquefaction Resistance of Soils

B.1 Effects of Laboratory Factor

B.1.1 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.

**B.1.2 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.

**B.1.3 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 C.1. In Figure C.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. 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 changed 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.
Figure C.1 Effect of loading wave form on cycles to initial liquefaction for moist-tamped Specimen

B.1.4 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 0.01 Hz, cyclic strength was independent of frequency effect while above 0.01 Hz, cyclic strength tend to increase with increasing frequency (Samuelson, 1981).
B.1.5 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.

B.1.6 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).

B.1.7 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.

When pressure during consolidation is applied to a sample through a rubber membrane, the membrane deforms and is pushed into the pore spaces between the grains. This results in
expulsion of some pore water from the sample, without a change in void ratio of the sample. Thus the measured volume change during consolidation must be corrected for membrane penetration when void ratio is calculated.

There are a number of theoretical studies (Molenkamp and Luger, 1981; Baldi and Nova, 1984; Kramer et al., 1990) summarized by Ali et al., (1995) suggesting the form of the equation for membrane penetration. For practical purposes, membrane penetration can be quantified in terms of a normalized membrane penetration:

$$\varepsilon_m = \frac{\Delta V_m}{A_s \log \left(\frac{p_1'}{p_2'}\right)}$$

where $\varepsilon_m$ = normalized membrane penetration

$\Delta V_m$ = volume change due to membrane penetration

$A_s$ = sample area covered by the membrane ($2\pi rh$ for a cylindrical sample)

$p_1', p_2'$ = net pressure acting across acting the membrane before and after the volume change

For sands, $\varepsilon_m$ is primarily dependent on grain size, assuming other factors such as membrane thickness and modulus are content.

### B.1.8 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.

B.1.9 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.

Y.P Vaid, J.D. Stedman and S.Sivathayalan (2001) showed that in cyclic loading the effect of increasing confining stress at a given static shear generally decreased the resistance to liquefaction. However, at the loosest states the increase in confining stress had little effect.

**B.1.10 Cyclic Stress Amplitude and Number of Cyclic Stress Cycles**

In their laboratory study, Seed and Lee (1966) concluded that the larger the stress or stain, 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, Lee, 1997).
B.1.11 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 0.1 mm 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.

B.1.12 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.

B.1.13 Lateral Earth Pressure ($K_0$) and Over consolidation Ratio

A Study on 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, uncedmented, 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.

**B.1.14 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.
APPENDIX C

C. Procedure for Fabricating Rubber Membranes

C.1 Initial Step

C.1.1 Regulate the oven to 158±9°C (70±5°C).

C.1.2 Be sure the proper amounts of coagulant and latex are in their respective containers
(Always be sure there is enough coagulant and latex in the containers to cover the largest
mandrel, but not so much as to cause overflow during the dipping process.)

C.1.3 Stir the latex vigorously (Stir the latex and allow it to settle for 45 minutes before
dipping the mandrel into it so that any bubbles caused by the stirring action will be
dispersed. The latex must be stirred well before each use because a thick film of
concentrated latex will form on the top. Inadequate stirring will cause non-uniformity in
the membrane thickness)

C.2 Clean the mandrel(s) to be dipped.

C.2.1 Wash mandrels with detergent.

C.2.2 Rinse mandrels thoroughly in warm water.

C.2.3 Place mandrels in the oven to dry. A 15-20 minute drying time is usually sufficient.

C.3 Dip the mandrel in the coagulant while the mandrel is still warm from the drying process.

C.3.1 Immerse the entire mandrel briefly into the coagulant.

C.3.2 Allow excess coagulant to drip off the mandrel.

C.3.3 Inspect the mandrel for unwetted spots.

C.3.4 Place the mandrel back into the oven for 25 minutes. (This step drives off the
methanol and leaves a sticky film of calcium nitrate on the surface of the mandrel. An
oven-curing time of more than 25 minutes will cause the calcium nitrate to crystallize,
resulting in spotty concentrations of latex on the mandrel)
C.4 Dip the mandrel in the latex immediately after removing the mandrel from the oven.

C.4.1 Prior to dipping the mandrel in the latex, inspect the surface of the latex in the container to make sure it is free from air bubbles and impurities. A spoon can be used to scoop the surface clean.

C.4.2 Slowly immerse the mandrel into the latex. Take care not to trap air between the latex and the mandrel surface.

C.4.3 Dwell time begins when the mandrel is completely submerged. (A dwell time of ten seconds is sufficient to obtain a thin membrane with good strength and sensitivity. This is the type normally used by the Geotechnical Engineering Bureau. However, because membrane thickness is directly proportional to dwell time, a thinner or thicker membrane may be obtained by decreasing or increasing the dwell time)

C.4.4 Slowly remove the mandrel from the latex and allow the excess to drip off.

C.4.5 Inspect the latex coating on the mandrel for any uncovered areas. (A very small hole can be repaired by gluing a small patch of rubber membrane over the hole with rubber cement. Do this after the membrane has oven cured and while the membrane is still on the mandrel. A patch with rounded corners is most effective. A large uncovered area is difficult to repair. The procedure should be restarted from beginning)

C.5 Initial curing stage.

C.5.1 Place the mandrel in the oven at 158±9° F (70±5°C) for three hours.

C.6 Strip the membrane off the mandrel.

C.6.1 With the membrane still on the mandrel, cut along the top and bottom edges of the membrane with a sharp razor blade or Exact-o-knife. The cut must be smooth and even. Apply a thin coat of rubber cement 0.5 in. (12.7 mm) wide along each edge. Allow the
cement to become tacky, and then carefully roll the membrane edges down 0.5 in. (12.7 mm). This procedure creates strong, tear resistant edges.

C.6.2 Dust the membrane with talcum powder. This prevents the membrane from sticking to itself when being stripped from the mandrel.

C.6.3 Carefully pull the membrane down the mandrel, stopping at intervals to dust the inside portion of the membrane with talcum powder. (Avoid excessively stretching the membrane as it is not fully cured and will tear easily)

C.7 Final curing stages

C.7.1 Completely submerge the membrane in warm water for three hours or in cold water overnight. This will remove any latent ammonia from the membrane.

C.7.2 Remove the membrane from the water and allow it to air-dry. (Do not subject the membrane to stretching until it is completely dry. It is very weak and will tear quite easily)

C.7.3 Trim any rough edges from the membrane.

C.7.4 Store the membrane in a dry place away from any light source. (Petroleum products will destroy natural rubber. Therefore, do not expose the membranes to petroleum based oils, petroleum jelly, etc.) Tight closing cardboard boxes of sufficient size would be a good way to store the membranes.
### APPENDIX D

**D. Cyclic Triaxial and Hollow Cylinder Tests Results**

<table>
<thead>
<tr>
<th>Relative Density after saturation (%)</th>
<th>Stress Ratio</th>
<th>Void Ratio</th>
<th>No. cycles to reach initial liquefaction</th>
<th>Deviator Stress (psi)</th>
<th>Consolidation Pressure (psi)</th>
<th>Fine Content (%)</th>
<th>Plasticity Index, PI</th>
<th>Sample No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>30.22</td>
<td>0.2</td>
<td>0.7315</td>
<td>32</td>
<td>6</td>
<td>15</td>
<td>5</td>
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<td>30.32</td>
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<td>19</td>
<td>9</td>
<td>15</td>
<td>5</td>
<td>20</td>
<td>2</td>
</tr>
<tr>
<td>30.01</td>
<td>0.4</td>
<td>0.7320</td>
<td>10</td>
<td>12</td>
<td>15</td>
<td>5</td>
<td>20</td>
<td>3</td>
</tr>
<tr>
<td>30.01</td>
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<td>0.7320</td>
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<td>12</td>
<td>30</td>
<td>5</td>
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<td>29.8</td>
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<td>0.7325</td>
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<td>18</td>
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<td>20</td>
<td>5</td>
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<td>0.7317</td>
<td>30</td>
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<td>5</td>
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<td>6</td>
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<td>6</td>
<td>15</td>
<td>10</td>
<td>20</td>
<td>7</td>
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<tr>
<td>30.67</td>
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<td>0.7304</td>
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<td>15</td>
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<td>20</td>
<td>8</td>
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<td>0.7321</td>
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<td>15</td>
<td>10</td>
<td>20</td>
<td>9</td>
</tr>
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<td>Void Ratio</td>
<td>No. cycles to reach initial liquefaction</td>
<td>Deviator Stress (psi)</td>
<td>consolidation pressure (psi)</td>
<td>Fine Content (%)</td>
<td>Plasticity index, PI</td>
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Figure D1. Test results (a) and (b) on Sample No.1.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D2. Test results (a) and (b) on Sample No.2.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D3. Test results (a) and (b) on Sample No.3.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D3. Test results (a) and (b) on Sample No.3.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D4. Test results (a) and (b) on Sample No.4.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D5. Test results (a) and (b) on Sample No.5.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D6. Test results (a) and (b) on Sample No.6.
Figure D7. Test results (a) and (b) on Sample No. 7.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

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Figure D8. Test results (a) and (b) on Sample No.8.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D9. Test results (a) and (b) on Sample No.9.
Figure D10. Test results (a) and (b) on Sample No.10.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D11. Test results (a) and (b) on Sample No.11.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D12. Test results (a) and (b) on Sample No.12.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D12. Test results (a) and (b) on Sample No.12.
Figure D13. Test results (a) and (b) on Sample No.13.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D13. Test results (a) and (b) on Sample No.13.
Figure D14. Test results (a) and (b) on Sample No.14.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D14. Test results (a) and (b) on Sample No.14.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D15. Test results (a) and (b) on Sample No.15.
Figure D16. Test results (a) and (b) on Sample No. 16.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess pore water pressure versus number of cycles to liquefaction

Figure D16. Test results (a) and (b) on Sample No. 16.
Figure D17. Test results (a) and (b) on Sample No.17.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D17. Test results (a) and (b) on Sample No.17.
Figure D18. Test results (a) and (b) on Sample No.18.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D19. Test results (a) and (b) on Sample No.19.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 20. Test results (a) and (b) on Sample No.20.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 21. Test results (a) and (b) on Sample No.21.

(a) Cyclic deviator stress( psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 22. Test results (a) and (b) on Sample No.22.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 23. Test results (a) and (b) on Sample No.23.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D 24. Test results (a) and (b) on Sample No.24.
Figure D 25. Test results (a) and (b) on Sample No.25.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D 25. Test results (a) and (b) on Sample No.25.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D 26. Test results (a) and (b) on Sample No.26.
Figure D 27. Test results (a) and (b) on Sample No.27.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 28. Test results (a) and (b) on Sample No.28.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 29. Test results (a) and (b) on Sample No.29.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 30. Test results (a) and (b) on Sample No.30.

(a) Cyclic deviator stress(psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 31. Test results (a) and (b) on Sample No.31.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D 32. Test results (a) and (b) on Sample No.32.
Figure D 33. Test results (a) and (b) on Sample No.33.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 34. Test results (a) and (b) on Sample No.34.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D 35. Test results (a) and (b) on Sample No.35.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D 36. Test results (a) and (b) on Sample No.36.
Figure D37. Test results (a) and (b) on Sample No.37.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D38. Test results (a) and (b) on Sample No.38.
Figure D39. Test results (a) and (b) on Sample No.39.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D40. Test results (a) and (b) on Sample No.40.
Figure D41. Test results (a) and (b) on Sample No.41.
Figure D42. Test results (a) and (b) on Sample No.42.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D43. Test results (a) and (b) on Sample No.43.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D44. Test results (a) and (b) on Sample No.44.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D45. Test results (a) and (b) on Sample No.45.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D46. Test results (a) and (b) on Sample No.46.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D47. Test results (a) and (b) on Sample No.47.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D48. Test results (a) and (b) on Sample No.48.
Figure D49. Test results (a) and (b) on Sample No.49.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D50. Test results (a) and (b) on Sample No.50.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

Excess pore water pressure versus number of cycles to liquefaction

Figure D51. Test results (a) and (b) on Sample No.51.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D52. Test results (a) and (b) on Sample No.52.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D53. Test results (a) and (b) on Sample No.53.
Figure D54. Test results (a) and (b) on Sample No.54.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D55. Test results (a) and (b) on Sample No.55.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D56. Test results (a) and (b) on Sample No.56.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(a) Excess Pore water pressure versus number of cycles to liquefaction
Figure D57. Test results (a) and (b) on Sample No.57.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D58. Test results (a) and (b) on Sample No. 58.
Figure D59. Test results (a) and (b) on Sample No. 59.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D60. Test results (a) and (b) on Sample No. 60.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D61. Test results (a) and (b) on Sample No.61.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure (psi) versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D62. Test results (a) and (b) on Sample No.62.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D63. Test results (a) and (b) on Sample No.63.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D64. Test results (a) and (b) on Sample No.64.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D65. Test results (a) and (b) on Sample No.65.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D66. Test results (a) and (b) on Sample No.66.
Figure D67. Test results (a) and (b) on Sample No.67.
Figure D68. Test results (a) and (b) on Sample No.68.
Figure D69. Test results (a) and (b) on Sample No.69.
Figure D70. Test results (a) and (b) on Sample No.70.
Figure D71. Test results (a) and (b) on Sample No.71.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D72. Test results (a) and (b) on Sample No.72.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D73. Test results (a) and (b) on Sample No.73.
Figure D74. Test results (a) and (b) on Sample No.74.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D75. Test results (a) and (b) on Sample No.75.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D76. Test results (a) and (b) on Sample No.76.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D77. Test results (a) and (b) on Sample No.77.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D78. Test results (a) and (b) on Sample No.78.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D79. Test results (a) and (b) on Sample No.79.
Figure D80. Test results (a) and (b) on Sample No.80.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D81. Test results (a) and (b) on Sample No.81.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D82. Test results (a) and (b) on Sample No.82.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D83. Test results (a) and (b) on Sample No. 83.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D84. Test results (a) and (b) on Sample No. 84.
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D85. Test results (a) and (b) on Sample No.85.
Figure D86. Test results (a) and (b) on Sample No.86.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D87. Test results (a) and (b) on Sample No.87.
Figure D88. Test results (a) and (b) on Sample No.88.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D89. Test results (a) and (b) on Sample No.89.
Figure D90. Test results (a) and (b) on Sample No.90.
Figure D91. Test results (a) and (b) on Sample No. 91.
Figure D92. Test results (a) and (b) on Sample No.92.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
Figure D93. Test results (a) and (b) on Sample No.93.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure D94. Test results (a) and (b) on Sample No.94.
Figure D95. Test results (a) and (b) on Sample No.95.
Figure D96. Test results (a) and (b) on Sample No.96.

(a) Cyclic deviator stress (psi) versus number of cycles to liquefaction

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E1. Test Results (a) and (b) on sample No.1 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E2. Test Results (a) and (b) on sample No.2 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction.

Figure E3. Test Results (a) and (b) on Sample No.3 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E4. Test Results (a) and (b) on Sample No.4 in CHCT.
Figure E5. Test Results (a) and (b) on Sample No.5 in CHCT.

(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E6. Test Results (a) and (b) on Sample No.6 in CHCT.
Figure E7. Test Results (a) and (b) on Sample No.7 in CHCT.

(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E8. Test Results (a) and (b) on Sample No.8 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E9. Test Results (a) and (b) on Sample No.9 in CHCT.
Figure E10. Test Results (a) and (b) on Sample No.10 in CHCT.

(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E11. Test Results (a) and (b) on Sample No.11 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E12. Test Results (a) and (b) on Sample No.12 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E13. Test Results (a) and (b) on Sample No. 13 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E14. Test Results (a) and (b) on Sample No.14 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E15. Test Results (a) and (b) on Sample No.15 in CHCT.
Figure E16. Test Results (a) and (b) on Sample No.16 in CHCT.

(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E17. Test Results (a) and (b) on Sample No.17 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E18. Test Results (a) and (b) on Sample No.18 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E19. Test Results (a) and (b) on Sample No.19 in CHCT.
(a) Cyclic shear stress (psi) versus number of cycles to liquefaction.

(b) Excess Pore water pressure versus number of cycles to liquefaction

Figure E20. Test Results (a) and (b) on Sample No.20 in CHCT.