The Effects Of Non-Plastic and Plastic Fines On The Liquefaction Of Sandy Soils

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Dissertation submitted to the Faculty of the
Virginia Polytechnic Institute and State University
in Partial fulfillment of the requirements for the degree of

Doctor of Philosophy
in
Civil Engineering

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10 December 1999

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(ABSTRACT)

The presence of silt and clay particles has long been thought to affect the behavior of a sand under cyclic loading. Unfortunately, a review of studies published in the literature reveals that no clear conclusions can be drawn as to how altering fines content and plasticity actually affects the liquefaction resistance of a sand. In fact, the literature contains what appears to be contradictory evidence. There is a need to clarify the effects of fines content and plasticity on the liquefaction resistance of sandy soils, and to determine methods for accounting for these effects in engineering practice.

In order to help answer these questions, a program of research in the form of a laboratory parametric study intended to clarify the effects which varying fines content and plasticity have upon the liquefaction resistance of sandy sands was undertaken. The program of research consisted of a large number of cyclic triaxial tests performed on two sands with varying quantities of plastic and non-plastic fines. The program of research also examined the applicability of plasticity based liquefaction criteria and the effects of fines content and plasticity on pore pressure generation. Lastly, a review of how the findings of this study may affect the manner in which simplified analyses are performed in engineering practice was made.

The results of the study performed are used to clarify the effects of non-plastic fines content and resolve the majority of the inconsistencies in the literature. The effects of plastic fines content and fines plasticity are shown to be different than has been previously reported. The validity of plasticity based liquefaction criteria is established, the mechanism responsible for their validity is explained, and a new simplified criteria proposed. The effects of fines content and plasticity on pore pressure generation are discussed, and several recommendations are made for implementing the findings of this study into engineering practice.

Keywords: Liquefaction, Fines, Silt, Clay, Liquefaction Criteria, Pore Pressures, Simplified Procedure, Fines Correction, Cyclic Triaxial Test

This work is dedicated to my family, without whose love and support it could never have been attempted, much less completed

Facilis descensus Averno,
noctes atque dies patet atri ianua Ditis,
sed revocare gradum superasque evadere ad auras,
hoc opus, hic labor est.

- Vergil

Acknowledgements

From beginning to end, my career as a doctoral candidate has spanned just over ten years. To fully thank everyone involved in making this project possible and for helping to make me the person and engineer that I have become would require a second volume at least as large as the document before the reader. If I have left anyone off this list please realize that you are only missing from this page, not from my heart.

I would like to thank my advisor, Doctor James R. Martin for his support and the freedom to pursue this study, for better or worse, in the manner that I thought best. I would also like to thank my committee for their time, efforts and valuable input.

I would like to thank my family for their support, encouragement and love during the creation of this work and in my daily life.

I would like to thank my fellow graduate students and co-workers for their friendship, input, and support over the years. This group includes, but is certainly not limited to, Chris and Diane Baxter, Ken Berry, Fernando Bolinaga, Ron Boyer, Tony Brizendine, Ira Brotman, Jim Coffey, Mary Jane Contos, George Filz, Trish Gallagher, Mike Galli, Mark Gutberlet, Dick Gutschow, Ivan Hee, Matt Helmers, Glenn Hermanson, Jeff Huffman, Ben Jarosz, Craig Johnson, Carolynn Jordan, Mia Kannik, Randy Kuzniakowski, Christian Lovern, Lorena Manriquez, Steve McMullen, Clark Morrison, Aaron Muck, Grigg Mullen, Kathy Murtagh, John Pappas, Larry Perko, Toni Poe, Eric Pond, Gretchen Rau, Levi Regalado, Andy Rose, Keith Royston, Scott Saunders, Mike Scarlett, Jen Schaefer, Will Shallenberger, C. J. Smith, Sharon Stoller, Trevor Thomas, Steve Winter, and Aaron Zdinak.

And a very special and heartfelt thanks to the special friends who have made it possible for me to keep some small fraction of my sanity over the last 10 years. Mere words will never be able to repay the debt I owe. Thanks to my mother Wynette, my sister Antoinette, my Brother Dominic and his family, The Ellingboe Family, Mike Arnold, Sam Lodge, Mark OrrNick Harman, Will Bassett, Rafael and Jean Castro, Jon Porter, Jerry Tracy, Karen McClellan, Ron Boyle, Scott Mills, Brad Hardesty, Jennifer Scruggs Brilliant, Harry Cook, Bruce Lacina, Jim Hawkins, Govi Kanann, Munish Kapoor, J.T. McGinnis, Jeff McGregor, Brian Metcalfe, and Russell Green.

Table Of Contents

List of Tables

List of Figures

CHAPTER 1: INTRODUCTION

Derived from the Latin verb “liquefacere”, meaning to melt, to dissolve, or to weaken, liquefaction is the term commonly used to describe the sudden, dramatic strength loss which sometimes occurs in sands during seismic loading. While most frequently associated with cohesionless soils and dynamic loadings, it has been reported in many types of soils under both dynamic and static loadings.

The liquefaction of sands during earthquakes has occurred throughout recorded history, and certainly before that, however it was not until the early 1960’s that scientific research into the subject began in earnest. Since the 1964 Anchorage, Alaska, and Nigata, Japan earthquakes, great strides have been made in understanding the mechanisms behind liquefaction and the conditions that make soils susceptible to it.

1.1 Statement Of The Problem

It has been understood since the 1960's that the presence of silt and clay particles will in some manner affect the resistance of a sand to liquefaction. However, a review of studies published in the literature shows that no clear conclusions can be drawn as to in what manner altering the fines content affects the liquefaction resistance of a sand under cyclic loading. This is particularly true for soils containing non-plastic, i.e. silty, fines. Numerous laboratory studies have been performed, and have produced what appear to be conflicting results. Studies have reported that increasing the silt content in a sand will increase the liquefaction resistance of the sand, decrease the liquefaction resistance of the sand, or decrease the liquefaction resistance until some limiting silt content is reached, and then increase its resistance. Additionally, several studies have shown that the liquefaction resistance of a silty sand is more closely related to its sand skeleton void ratio than to its silt content. The need to clarify the effects of non-plastic fines on the

liquefaction susceptibility of sandy soils forms the first major point of investigation in this research.

The presence of plastic or clayey fines is generally considered to decrease the liquefaction susceptibility of a soil. Numerous field studies have shown that soils with more than 10 or 15 percent fines do not liquefy during earthquakes. During the 1970's, Engineers in the Peoples Republic of China (PRC) developed a set of criteria in their building codes, commonly referred to as the Chinese criteria, which deem certain soils as "non-liquefiable" due to their plastic nature. How the introduction of clayey fines actually affects the liquefaction susceptibility of a soil and the validity of plasticity based liquefaction criteria form major points of investigation of this research.

The manner in which pore pressure generation varies as the quantity and type of fine-grained material in a sand increases is an important aspect of this investigation. In addition to affecting the liquefaction behavior of the soil, the rate and magnitude of pore pressure generation during seismic loading has a profound effect on the strength of the soil and the stability of structures constructed from, or founded on it. Pore pressure generation during cyclic loading can be examined either in terms of the number of cycles of loading that the soil has undergone or else in terms of the strain which that loading has produced in the soil mass. Both criteria will be used to examine the effects of fines content and plasticity on pore pressure generation.

Lastly, it is important to examine in what manner the findings of this study will affect the manner in which working engineers perform simplified liquefaction analyses. Are the charts currently in use adequately conservative, or are they overly so? Are the corrections currently made for non-plastic fines correct? Are new corrections needed for plastic fines, or are the Chinese criteria valid and can be incorporated into current practices? A summary of these issues will conclude this research program.

In summary, the problems under investigation in this research program can be broken down into five separate issues:

1) The effects of non-plastic fines content on the liquefaction resistance of sandy soils
2) The effects of plastic fines content and plasticity on the liquefaction resistance of sandy soils
3) The validity and applicability of plasticity based liquefaction criteria
4) The effects of plastic and non-plastic fines on pore pressure generation.
5) How do the findings on the first four issues affect the manner in which simplified liquefaction analyses are performed by practicing engineers?

1.2 Scope Of The Research

In order to clarify the questions previously outlined, a program of research in the form of a laboratory parametric study intended to clarify the effects which varying fines content and plasticity have upon the liquefaction resistance of sandy sands was undertaken. The program of research also examined the applicability of plasticity based liquefaction criteria and the effects of fines content and plasticity on pore pressure generation. Lastly, a review of how the findings of this study may affect the manner in which simplified analyses are performed in engineering practice was made.

During the first portion of the study a series of cyclic triaxial tests were run in order to study the effects of non-plastic silt content on the liquefaction resistance of sandy soils. Tests were run on two sands in which silt content was varied while either the gross void ratio, the sand skeleton void ratio, or the soil specific relative density was held constant. For each combination of silt content and density, a minimum of three specimens were tested at various cyclic stress ratios (CSR's) in order to produce a series of curves which plot cyclic stress ratio against the number of cycles required to cause liquefaction. From these curves, the cyclic resistance of the soil was determined for each combination of silt

content and density. The variation in cyclic resistance with silt content was then examined in terms of the gross void ratio, the sand skeleton void ratio, and the relative density of the specimens tested.

During the second portion of the study a series of cyclic triaxial tests was run in order to study the effects of plastic fines content and plasticity on the liquefaction resistance of a clayey sand. Tests were run at a constant soil specific relative density for various combinations of fines content and plasticity. The composition of the fines, and thus their plasticity, was varied by combining different proportions of non-plastic silt, kaolinite, and bentonite. The cyclic resistance was then examined in terms of the fines content, the clay content, the liquid limit, the plasticity index, and the activity of the soil tested.

Jennings (1980) presents a listing of the parameters used by engineers in the People's Republic of China to separate soils which are considered liquefiable from those considered non-liquefiable. Soils meeting the Chinese criteria or other plasticity based liquefaction criteria are considered to be non-liquefiable. The results of the cyclic triaxial testing program were used to examine the validity of these criteria and to determine if there are any fundamental behavioral differences between soils meeting or not meeting the criteria.

Data from the cyclic triaxial tests were analyzed to evaluate the effects of fines content and plasticity on the pore pressure generation characteristics of silty and clayey sands. Pore pressure generation was evaluated both in terms of the number of cycles of loading required to cause a given rise in pore pressure and in terms of the amount of strain require to achieve a certain level of pore pressure ratio.

Lastly, whether the findings of the current research will have any effect on the manner in which simplified liquefaction analyses are currently performed in engineering practice was examined.

1.3 Outline Of This Document

Following this introductory section, Chapter 2 of this document presents the results of a literature review performed to learn what is currently understood about the effects of non-plastic and plastic fines content and fines plasticity on the liquefaction of sandy soils. Next, Chapter 3 consists of a discussion of the soils tested, a brief review of cyclic triaxial testing and the factors which affect it, and a detailed review of the cyclic triaxial test methodology used in this study. Chapter 4 presents the findings of the study on the effects of non-plastic fines content on the liquefaction of sandy soils. One global theory is presented and used to reconcile the differences found in the literature. Chapter 5 presents the findings of the effects of plastic fines content and plasticity on the liquefaction of sandy soils, and examines the validity of the Chinese criteria and other plasticity based liquefaction criteria and the mechanisms behind them. Chapter 6 examines the manner in which plastic and non-plastic fines affect pore pressure generation in sands during cyclic loading. Chapter 7 examines how the findings of this study may impact the manner in which simplified liquefaction analyses are performed in engineering practice. Chapter 8 presents a summary of conclusions drawn from this study and suggestions for further studies in this area. A series of appendices containing the parameters and data for the individual cyclic triaxial tests, data from the index density testing program and information on the limiting silt content follow Chapter 9.

Table 1-1 contains a listing of some of the terms and equations used in this document and their definitions.

Table 1-1: Definition of terms and appropriate equations	
Parameter	Applicable Equations and Definitions
Gross Void Ratio, e_g	$e_g = \frac{V_v}{V_s}$ V_v = Volume of voids V_s = Volume of solids
Sand Skeleton Void Ratio, e_{ss}	$e_{ss} = \frac{V_{vss}}{V_{ss}}$ V_{vss} = Volume of voids formed by the sand skeleton V_{ss} = Volume of sand solids
Silt Fraction Void Ratio, e_m	$e_m = \frac{V_v}{V_{sm}}$ V_{sm} = Volume of silt solids V_{vss} = Volume of voids formed by the silt skeleton
Gross Relative Density, Dr	$Dr = \frac{e_{max,cs} - e_g}{e_{max,cs} - e_{min,cs}} x100\%$ $e_{max,cs}$ = maximum index void ratio of the clean sand $e_{min,cs}$ = minimum index void ratio of the clean sand
Soil Specific Relative Density, Dr_s	$Dr = \frac{e_{max,s} - e_g}{e_{max,s} - e_{min,s}} x100\%$ $e_{max,s}$ = maximum index void ratio of the sand/silt mixture $e_{min,s}$ = minimum index void ratio of the sand/silt mixture
Limiting Silt Content	The percentage, by weight, of silt below which the soil structure consists of silt contained in a sand matrix and above which it consists of sand grains suspended in a silt matrix.
Initial Liquefaction	The point at which the pore pressure in the specimen first becomes equal to the cell pressure, creating a condition of zero effective stress

Table 1-1: Definition of terms and appropriate equations	
Cyclic Resistance	The cyclic stress ratio required to achieve liquefaction in a specified number of cycles.
Normalized Cyclic Resistance	The cyclic resistance of a soil is normalized by dividing the cyclic resistance at the given silt content by the cyclic resistance of the clean base sand having the same density, effective confining stress, and failure criteria.

CHAPTER 2: LITERATURE REVIEW

The published results of geotechnical studies were examined in order to determine the state of knowledge on the effects of fines content and plasticity on the liquefaction resistance and pore pressure generation characteristics of sandy soils. Additionally, a review of literature related to plasticity based liquefaction criteria was performed. The results of this review are summarized in Table 2-1, and are presented herein.

2.1 The Effects of Fine Content and Plasticity on Liquefaction Resistance

Both clean sands and sands containing fines have been shown to be liquefiable in the field (Mogami and Kubo (1953); Robertson and Campenella (1985); and Holzer et al. (1989)) and in the laboratory (Lee and Seed (1967a); Chang et al. (1982); and Koester (1994)). Additionally, non-plastic silts, most notably mine tailings, have also been found to be susceptible to liquefaction (Dobry and Alvarez (1967); Okusa et al. (1980); and Garga and McKay (1984)). A review of the literature, however, shows conflicting evidence as to the effect which fines have on the liquefaction resistance or cyclic strength of a sand. The main factors that are reviewed here are the effects of non-plastic fines content and the effects of plastic fines content and plasticity on the liquefaction resistance of sandy soils.

2.1.1 The Effects Of Non-Plastic Fine Content

There is no clear consensus in the literature as to the effect which increasing non-plastic fines content has upon the liquefaction resistance of a sand. Both field and laboratory studies have been performed, and the results of these studies indicate that increasing the non-plastic fines content in a sand will either increase the liquefaction resistance of the sand, decrease the liquefaction resistance of the sand, or decreases the liquefaction resistance until some limiting fines content is reached, and then increases its resistance.

To further complicate issues, some researchers have shown that the liquefaction resistance of silty sands is not a function of the silt content of the soil so much as it is a function of the soil's sand skeleton void ratio.

2.1.1.1 Field Studies

Field studies following major earthquakes have produced conflicting evidence as to the effects of silt on the liquefaction resistance of sands. Based upon case histories of actual soil behavior during earthquakes, there is evidence that soils with greater fines contents are less likely to liquefy in a seismic event. Okashi (1970) observed that during the 1964 Nigata earthquake in Japan, sands were more likely to liquefy if they had fines content of less than 10 percent. Additionally, Fei (1991) reports that for the 1976 Tangshan earthquake in China the liquefaction resistance of silty soils increased with increasing fines content. Finally, Tokimatsu and Yoshimi (1983) found in a study of 17 worldwide earthquakes that 50 percent of the liquefied soil had fines contents of less than 5 percent. They also found that sands with fines contents greater than 10 percent had a greater liquefaction resistance than clean sands at the same SPT blowcount.

While some research has shown that an increase in fines content results in an increase in liquefaction resistance, other research has shown the opposite effect. Tronsco and Verdugo (1985) report that mine tailings dams constructed of soils with higher silt contents are more likely to liquefy than similar dams constructed of sands with lower silt contents. Chang, Yeh, and Kaufman (1982) note that case studies reveal that most liquefaction resulting from earthquakes has occurred in silty sands and sandy silts.

Dobry and Alverez (1967), Okusa, Anma, and Maikuma (1980), and Garga and McKay (1984) each report cases of mine tailings dams constructed with up to one hundred percent silt-sized particle liquefying during earthquakes in Chile and Japan. All of the fines involved were either silts of low plasticity or non-plastic silts.

Field based methods for determining liquefaction susceptibility, such as methods based on SPT blowcounts or CPT measurements, must account for the presence of fines in the soil (Tatsuoka et al, 1980). Seed et al (1985) modified the cyclic stress ratio (CSR) versus normalized SPT blow count curves originally proposed by Seed and Idriss (1971) to account for the increase in liquefaction resistance provided by an increased fines content. The revised chart provides a series of curves for 5 percent, 15 percent, and 35 percent fines. These curves indicate that, for a given blowcount, a larger CSR it required to liquefy a soil with a higher fines content.

2.1.1.2 Laboratory Studies

As previously noted, there is a great discrepancy in the literature as to the effects which increasing the non-plastic, i.e. silty, fines content has upon the liquefaction resistance of a sandy soil. A brief review of these differing results follows.

Several investigators have found that the cyclic resistance of a sandy soil increases with increasing silt content. For specimens prepared to a constant gross void ratio, Chang et al. (1982) found that after a small initial drop, cyclic resistance increased dramatically with increasing silt content. The cyclic resistance increased nearly linearly with silt content until a silt content of 60 percent was reached, increasing to a cyclic resistance between 50 and 60 percent greater than that of the clean sand. Similarly, Dezfulian (1982) reported a trend of increasing cyclic resistance with increasing silt content. Both studies used silts with either some small level of plasticity or a measurable clay fraction. This trend of increasing cyclic strength with increasing fines content can be seen in the data for a sand tested at different fines contents by Chang et al (1982) which is plotted in Figure 2-1.

Numerous authors have reported a decrease in cyclic resistance with increasing silt content. Shen et al. (1977), Tronsco and Verdugo (1985), and Vaid (1994) have all reported this trend for specimens prepared either to a constant gross void ratio or a

constant dry density. The decreases in cyclic resistance were marked, decreasing as much as 60 percent from their clean sand values for an increase in silt content of 30 percent (Tronsco and Verdugo, 1985). This trend of decreasing cyclic strength with increasing fines content can be seen in the data for tailings sands tested by Tronsco and Verdugo shown in Figure 2-2.

Rather than a simple decrease in cyclic resistance with increasing fines contents, several investigators have reported that the cyclic resistance of the sand first decreased as the fines content increased and then increased after crossing some threshold fines content. Koester (1994) and Law and Ling (1992) found that for specimens prepared to a constant gross void ratio, as silt content increased the cyclic resistance of the soil decreased until some limiting silt content was reached, at which point the cyclic resistance began increasing. Koester (1994) reported a decrease in cyclic resistance to less than one-quarter of the clean sand cyclic resistance at a silt content of 20 percent, followed by an increase in cyclic resistance to 32 percent of the clean sand value at a silt content of 60 percent. Unlike Chang et al. (1982), and Dezfulian (1982), neither of these studies reported increases in cyclic resistance to levels greater than those determined for the clean sand. This fluctuation of cyclic strength with increasing fines content is clearly shown in Koester's results, which are presented in Figure 2-3.

Several studies have shown that cyclic resistance is more closely related to sand skeleton void ratio than it is to gross void ratio, gross relative density, or fines content. Finn, Ledbetter, and Wu (1994) found that at the same gross void ratio, the cyclic strength of a sand decreases with increasing fines content. They also found that at the same sand skeleton void ratio, cyclic strength remains constant with increasing fine content, as long as the fines can be accommodated in the void spaces created by the sand skeleton.

Not all soils however, exhibit a constant cyclic resistance with a constant sand skeleton void ratio. Shen et al. (1977), Kuerbis et al. (1988), and Vaid (1994) have shown that for

specimens prepared to constant sand skeleton void ratios, the cyclic resistance of a sand does not remain constant, but increases with increasing silt content.

Clearly, based upon the conflicting evidence presented in the literature, the fines content of a sandy soil does not alone provide a definitive measure of its liquefaction potential.

2.1.2 The Effects of Plastic Fines Content and Plasticity And Plasticity Based Liquefaction Criteria

There is general agreement in the literature as to the effect which the quantity and plasticity of the fine-grained material has on the liquefaction resistance of a sandy soil. There is agreement that whether the fine grained material is silt or clay, or more importantly, whether it behaves plastically or non-plastically, tends to make an important, consistent difference in the cyclic strength of the soil. The majority of studies have shown that the presence of plastic fines tend to increase the liquefaction resistance of a soil.

2.1.2.1 Field Studies

The effect of clay content on the liquefaction resistance of sandy soils has also been clearly established in field studies. Seed, Idriss, and Arango (1983) concluded that if a soil has a clay content greater than 20 percent it will not liquefy. A study of worldwide earthquakes by Tokimatsu and Yoshimi (1983) came to the same conclusion. The increase in cyclic strength that accompanies an increase in plasticity index is shown in Figure 2-4, from Ishihara (1996).

2.1.2.2 Laboratory Studies

Several laboratory studies have shown a strong correlation between an increased plasticity of the fine-grained portion of the soil and the increased liquefaction resistance of that soil. Ishihara and Koseki (1989) found that while there was no clear correlation between either clay content or fines content and liquefaction resistance, increasing plasticity index

consistently increased liquefaction resistance. Yasuda, Wakamatsu, and Nagase (1994) also found that increasing plasticity index increased liquefaction resistance.

Only Koester (1994) provides evidence that would appear to indicate that soil plasticity is not a controlling factor in liquefaction resistance in soils with plastic fines. He found that while at a given void ratio, fine type and plasticity play a minor role in liquefaction resistance, they exert far less influence than the percentage of fines in the soil.

2.2.3 Plasticity Based Liquefaction Criteria

Jennings (1980) presents a listing of the "thresholds to liquefaction" used by engineers in the People's Republic of China to separate soils which are considered liquefiable from those considered non-liquefiable. Soils meeting these criteria are considered to be non-liquefiable and include those with plasticity indexes greater than 10, clay contents greater than 10 percent, relative densities greater than 75 percent, and void ratios less than 0.80. Other criteria presented are related to epicentral distance, intensity, grain size and gradation, the depth of the sand layer, and the depth of the water table.

Seed et al. (1973) in their review of the slides that occurred in the Lower San Fernando Dam during the February 1971 San Fernando earthquake presented a modified form of the Chinese criteria. As reported by Marcuson et al. (1990), soils with greater than 15 percent material finer than 0.005 mm, liquid limits greater than 35 percent, and water contents less than 90 percent of the liquid limit should be safe from liquefaction.

Finn, Ledbetter, and Wu (1994) recommended that changes to be made to the Chinese criteria to account for uncertainty and differences in the liquid limit determination between the ASTM and the Chinese standard. They recommended decreasing the fines content by 5 percent, the liquid limit by 1 percent and the water content by 2 percent.

Koester (1994) recommend that a further change be made to the criteria proposed by Finn, Ledbetter, and Wu (1994) to better account for differences in the liquid limit determination between the ASTM and the Chinese standard. He suggested increasing the liquid limit criteria to a value of 36 percent.

2.2 The Effects Of Fines Content And Plasticity On Pore Pressure Generation

The rate and magnitude of pore pressure generation may have important effects on the shear strength, stability, and settlement characteristics of a soil mass, even if the soil does not liquefy. Similarly, the peak pore pressure generated may affect the stability of structure founded on, or in the soil mass.

2.2.1 Rate And Magnitude Of Pore Pressure Generation

There are two methods of examining the rate and magnitude of pore pressure generation during cyclic loading which have been reported in the literature. The first is to examine the pore pressures generated in relation to the ratio of the number of cycles of loading applied to the number of cycles required to cause liquefaction. This is the method used by Lee and Albaisa (1974). Pore pressures may also be measured in terms of the strain required to generate them. This is the approached taken by Dobry et al (1982).

2.2.1.1 Lee and Albaisa's Method

Lee and Albaisa (1974) performed a study of pore pressure generation during undrained cyclic loading. They found that the pore pressure in the sample rose steadily as the sample was loaded. They illustrated this behavior by plotting the ratio of pore pressure generated to the initial effective confining pressure against the ratio of the cycle number to the number of cycles required to cause initial liquefaction. As seen in Figure 2-5, the pore pressure generation curves fall within a relatively small band for tests performed on specimens prepared to a wide range of densities and consolidation pressures.

2.2.1.2 Dobry Et Al's Method

Dobry et al (1982) examined pore pressure generation during undrained cyclic loading as a function of shear strain. They found that for a constant number of loading cycles, the relationship between pore pressure generation and shear strain is essentially identical over a wide range of relative densities. Additionally, as can be seen in Figure 2-6, pore pressure do not begin to increase until some level of cyclic strain, deemed the threshold strain, is reached.

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1953	Mogami and Kubo	For loam with a PI of 34 percent, shear strength during vibration decreases with increasing acceleration of vibration
1961	Florin and Ivanov	All cohesionless materials can liquefy
1967	Dobry and Alvarez	Liquefaction occurred in numerous Chilean tailings dams with between 25 and 95 percent silt during the 1965 M7 earthquake.
1967a	Lee and Seed	For a compacted silt with a PI of 9, cyclic strength increases with: A) Increasing density B) Increasing confining pressure C) Increasing consolidation stress ratio
1968	Lee and Fitton	1) At constant confining stress and relative density, fine silty sands have the lowest cyclic strength 2) Grain size has more effect on cyclic strength than grain shape or grain size distribution 3) Peak pore pressure response decreases with increasing grain size 4) For granular soils, cyclic strength decreases with decreasing grain size 5) In silt and clay, cyclic strength increases with decreasing grain size 7) Clayey fines may improve cyclic strength considerably, while silty fines may tend to decrease the cyclic strength
1970	Ohsaki (as reported by Tokimatsu and Yoshimi, 1983)	Based upon data from the 1964 Nigata Earthquake, soil is likely to liquefy if: 1) Coefficient of uniformity, Cu < 5 2) Fines content < 10 percent
1971	Seed and Idriss	Very fine sands, with D_{50} approximately equal to 0.08 mm, are most susceptible to liquefaction
1974	Lee and Albaisa	The relationship between pore pressure ratio and the ratio of the cycle to the number of cycles required for 100 percent pore pressure ratio forms a small band for a given sand over a large range of densities and confining stresses

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1977	Ishihara, Sodekawa, and Tanaka	For soil with 0 to 100 percent fines, fines with PI of 20 1) As D_{50} increases, maximum pore pressure ratio increases 2) As percent fines increases, cyclic strength increases 3) For a given fines content, cyclic strength increases as OCR increases. Difference increases with increasing fines content 4) For a given D_{50}, cyclic strength increases as OCR increases. Difference increases with decreasing D_{50}.
1977	Shen et al	For samples prepared to a constant dry density: 1) As fines content increases, cyclic resistance decreases 2) For a constant sand skeleton void ratio, cyclic resistance increases
1979	Wang (as reported by Finn et al, 1994)	Soil is susceptible to significant strength loss or liquefaction if: 1) Percentage of particles smaller than 0.005 mm is < 15 percent 2) LL <= 35 percent 3) Natural W.C. >= 0.9LL 4) Liquidity Index >= 0.75
1980	Jennings	In Chinese building codes, soils meeting certain criteria are considered to be non-liquefiable. These include soils with: 1) plasticity indexes greater than 10 2) clay contents greater than 10 percent 3) relative densities greater than 75 percent 4) void ratios less than 0.80. 5) Other criteria are related to epicentral distance, intensity, grain size and gradation, the depth of the sand layer, and the depth of the water table.

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1980	Okusa, Anma, and Maikuma	Mine tailings with greater than 90 percent fines, but no plasticity, liquefied during the 1978 Izu-Oshima-Kinkai Earthquake in Japan. 1) Liquefied material was subrounded silt with grain to grain contact with some clay attached to larger particle faces. 2) Non-liquefied material was silt-sized particles wrapped in clay with no grain to grain contact.
1980	Tatsuoka et al	Must account for effect of grain size on blow count. Failure to do this leads to an overly conservative analysis for fine or silty sands
1981	Ishihara, Yasuda, and Yokota	For mine tailings composed of silty sands and sandy silts, cyclic strength: 1) Is independent of void ratio 2) Is independent of grain size 3) Increases with increasing plasticity index
1981	Tokimatsu and Yoshimi	D_{50} has little effect on liquefaction resistance if $D_{50} > 0.25$ mm
1982	Dezfulian	For undisturbed specimens of silty and clayey sands at various densities, cyclic resistance increases with increasing fines content.
1982	Dobry et al.	1) For a given number of cycles, the relationship between pore pressure ratio and shear strain is essentially identical over a wide range of densities 2) Pore pressures do not begin to develop until the threshold strain is developed

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1982	Chang, Yeh, and Kaufman	1) Case studies reveal that most liquefaction events have occurred in silty sands and sandy silts 2)The effect of gradation is less than the effect of mean grain size 3) Cyclic strength of a silty sand decreases from 0 to 10 percent silt content then increases to a silt content of 60 percent where it levels off. 4) At 10 percent silt content, sand grain to grain contact still prevails 5) Above 60 percent silt content, sand grains are merely floating in the silt matrix 6) As the number of cycles to failure increases, the effects of D_{50} and C_u become less important to cyclic resistance 7) For clean sands, cyclic strength increases with increasing D_{50} 8) Fine sands more susceptible to liquefaction than coarse sands 9) Differences in permeability due to differences in silt content lead to differences in pore pressure development
1982	W.L. Finn	Based on Chinese criteria, soil is liquefiable if the PI<10 and the clay content <10 percent. New seismic code proposed for soils with D_{50}>0.05 mm and granular soil > 40 percent
1982	Prakash and Puri	For loessial silts with 98 percent fines, PI<10, and percent clay <3 percent 1) Undisturbed samples stronger than remolded samples 2) It required significantly more cycles at a given CSR to produce pore pressure equal to the confining pressure than to produce 10 percent double amplitude axial strain, possibly due to cohesion delaying pore pressure development.

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1983	Seed, Idriss, and Arango	1) For sands with D_{50} < 0.25 mm use standard CSR vs. $N_{1,60}$ curves 2) For silty sands and silts plotting above the A-Line with D_{50} < 0.15 mm use $N_1 = N_{1,measured} + 7.5$ and use standard CSR vs. $N_{1,60}$ curves 3) Based upon Chinese criteria, clays may suffer significant strength loss during seismic shaking if: A) percent finer than 0.005 mm < 15 percent B) Liquid limit < 35 percent C) Water content > 0.9 x LL D) If it plots above the A-Line on the Atterberg chart, run tests to determine the cyclic loading characteristics of the soil 4) If clayey soil has clay content greater than 20 percent or a water content less than 90 percent of the liquid limit, it will not liquefy
1983	Tokimatsu and Yoshimi	1) In 1923 Kanto Earthquake, the Old Arakama bridge foundation settled more on clean sands than on silty sands despite clean sands having higher N values 2) In the same earthquake, sands with < 8 percent fines settled more than sands with >20 percent fines 3) Based upon field investigation following 17 world-wide earthquakes: A) 50 percent of liquefied soil had < 5 percent fines B) No liquefied soil had >20 percent clay 4) Sands with fines content > 10 percent have greater liquefaction resistance at same SPT blowcount
1984	EL Hosri, Biarez, and Hicher	Cyclic strength of silty clay increases with decreasing void ratio and increasing relative density.

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1984	Garga and McKay	Isotropically and anisotropically consolidated cyclic triaxial tests on undisturbed and remolded samples of 33 mine tailings materials. Materials had from 0 to 100 percent fines, but negligible plasticity. All results were adjusted to 50 percent D_r : 1) Undisturbed samples had higher cyclic strength than remolded samples 2) Non-tailings sands were stronger and coarser (fewer fines). 3) Cyclic strength increases with increasing consolidation ratio 4) In isotropically consolidated tests, soils with D_{50} between 0.1 and 0.3 mm has lowest cyclic strength
1985	Robertson and Campanella	1) Liquefaction resistance increases with decreasing D_{50} below a D_{50} less than approximately 0.25 mm 2) Cyclic stress ratio to cause liquefaction versus cone tip penetration is a function of grain size. Separate curves for $D_{50} < 0.15$ mm and $D_{50} > 0.25$ mm.
1985	Seed, Tokimatsu, Harder, and Chung	New CSR vs. $N_{1,60}$ curves based upon fines content. New curves for <5 percent, 15 percent and 35 percent fines content. The type or plasticity of fines is not taken into account.

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1985	Tronsco and Verdugo	1) Tailings dams with low silt content are more resistant to liquefaction than dams with higher silt contents. Possibly because increasing silt content decreases permeability and thus pore pressure dissipation. 2) At a constant void ratio of approximately 0.9: A) As silt content increases, soil becomes more compressive, less dilative. B) For a given shear strain, shear modulus, G, decreases as silt content increases. C) Cyclic strength increases by 270 percent as silt content decreases from 30 percent to 0 percent. This effect is more noticeable in the 0 to 15 percent range than in the 22 to 30 percent range.
1987	Seed	In liquefaction analysis of sands with fines, correct for fines by adding a $\Delta N_{1,60}$ factor to the SPT blowcount before entering CSR vs. blowcount chart. The type or plasticity of fines is not taken into account.
1988	Kuerbis, Negussey, and Vaid	1) At constant silt content, resistance increases as void ratio decreases 2) At constant void ratio, resistance decreases as silt content increases
1988	Shibata and Temparaksa	If $q_{c,crit}$ is the critical cone tip penetration dividing liquefiable and non-liquefiable for a given CSR: 1) For $D_{50} > 0.25$ mm, $q_{c,crit}$ is not a function of D_{50} 2) For $D_{50} < 0.25$ mm, $q_{c,crit}$ is a function of D_{50}. As D_{50} decreases, liquefaction potential decreases
1989	Holzer et al	During Imperial Valley earthquake of 1979 liquefaction occurred in silts with as little as 7 percent sand and up to 10 percent clay

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1989	Ishihara and Koseki	There is no clear correlation between either clay content or fines content and cyclic strength. PI correlates well. As PI increases, cyclic strength increases.
1990	Tronsco	Non-plastic fines decrease cyclic strength because they "...occupy the voids between the coarser soils smoothing the roughness of the coarser particles and reducing the interlocking shear strength mechanisms while at the same time they are decreasing the permeability by increasing the tortuosity for eventual flow."
1991	Fei	1) In remolded samples cyclic strength decreases as fines content increases based on fines with PI<10 2) In undisturbed samples cyclic strength initially decreases as fines content increases, then increases for fines content greater than 10 percent based on fines with PI<10. This is due to the effects of soil structure. 3) Residual strength is a function of pore pressure generation, which is a function of fines content. 4) The cone tip resistance q_c dividing liquefiable and non-liquefiable soils is a function of fines content.
1992	Law and Ling	For specimens prepared to a constant gross void ratio, as silt content increased, the cyclic resistance of a soil decreased until some limiting silt content was reached, at which point the cyclic resistance began increasing

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1994	Finn, Ledbetter and Wu	For non-plastic fines: 1) At same $N_{1,60}$, cyclic strength increases with increasing fine content 2) At same gross void ratio, cyclic strength decreases with increasing fines content 3) At same sand skeleton void ratio, cyclic strength remains constant with increasing fine content as long as they can be accommodated in the void space 4) Modify the Chinese Criteria by decreasing the fines content by 5 percent, the liquid limit by 1 percent, and the water content by 2 percent
1994	Koester	Based on results of 500 undrained cyclic triaxial tests on samples with a sand skeleton void ratio corresponding to a relative density of 50 percent: 1) Lowest cyclic strength occurs at 20 to 26 percent fines 2) Fine type (i.e. plasticity) is less important than percentage 3) Cyclic strength decreases with increasing fines content to approximately 20 percent fines, then increases. Cyclic strength was still lower at 60 percent fines than at 12 percent fines. 4) Lowest lower bound strength occurred for well graded sands at 20 to 30 percent fines 5) Residual strength of sand with 20 percent silt is very low 6) Post-liquefaction monotonic loading leads to a dilative response. 7) Liquefaction becomes inevitable at a pore pressure ratio of 70 percent for clean sands and 80 percent for sands with fines. Once this threshold pore pressure ratio is reached strain occurs more rapidly in sands with fines than in clean sands.

Table 2-1: Summary of literature review on the effects of fines on the liquefaction resistance of sandy soils		
YEAR	**INVESTIGATOR**	**FINDINGS**
1994	Singh	1) At 50 percent relative density sands with 10, 20, or 30 percent silt have slightly lower resistances to liquefaction than clean sand at the same relative density. 2) At constant void ratio, cyclic strength increases with increasing silt content. This may be due to increasing relative density as more silt is added at a constant void ratio.
1994	Vaid	1) In compression, sand exhibits increasing dilatancy with increasing silt content 2) In extension, sand exhibits only slightly increasing contractivness with increasing silt content 3) Silty sands are liquefiable only in extension not in compression 4) At constant silt content, resistance increases as void ratio decreases 5) At constant void ratio, resistance increases as silt content decreases 6) At constant sand skeleton void ratio, resistance increases slightly as silt content increases.
1994	Yasuda, Wakamatsu, and Nagase	1) As fines percentage increases, cyclic strength increases slightly 2) As clay percentage increases, cyclic strength increases slightly 3) As PI increases, cyclic strength increases 4) Undisturbed "aged" samples of silty sand are stronger than remolded samples 5) Strength gain with time is more rapid for silty sands than for clean sands 5) Increase in SPT blowcounts from 1 year and 50 years after fill placement is most significant if soil contains more than 40 percent fines

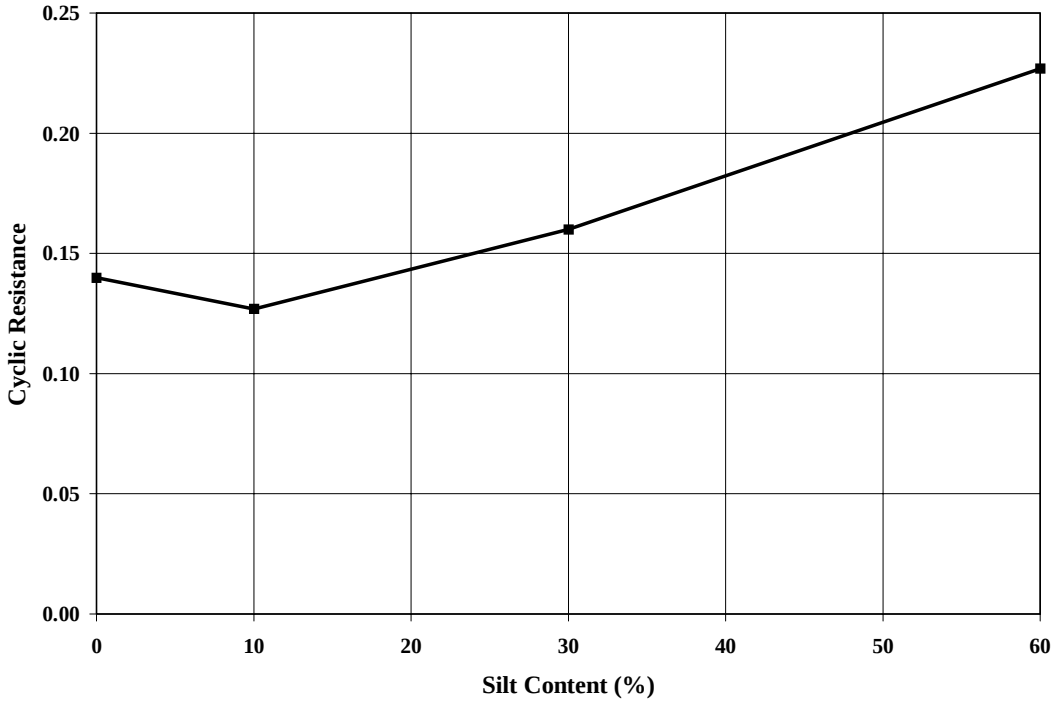

Figure 2-1: Increase in cyclic resistance with increase in silt content
(After Chang et al., 1982)

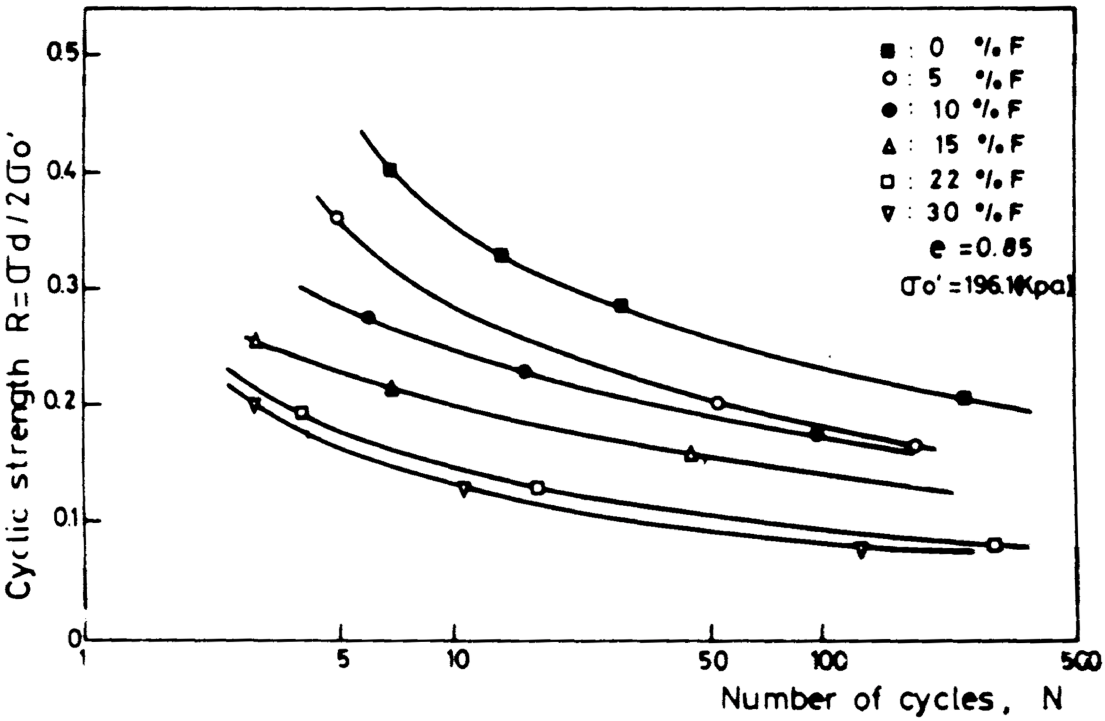

Figure 2-2: Decrease in cyclic resistance with increase in silt content
(From Tronsco and Verdugo, 1985)

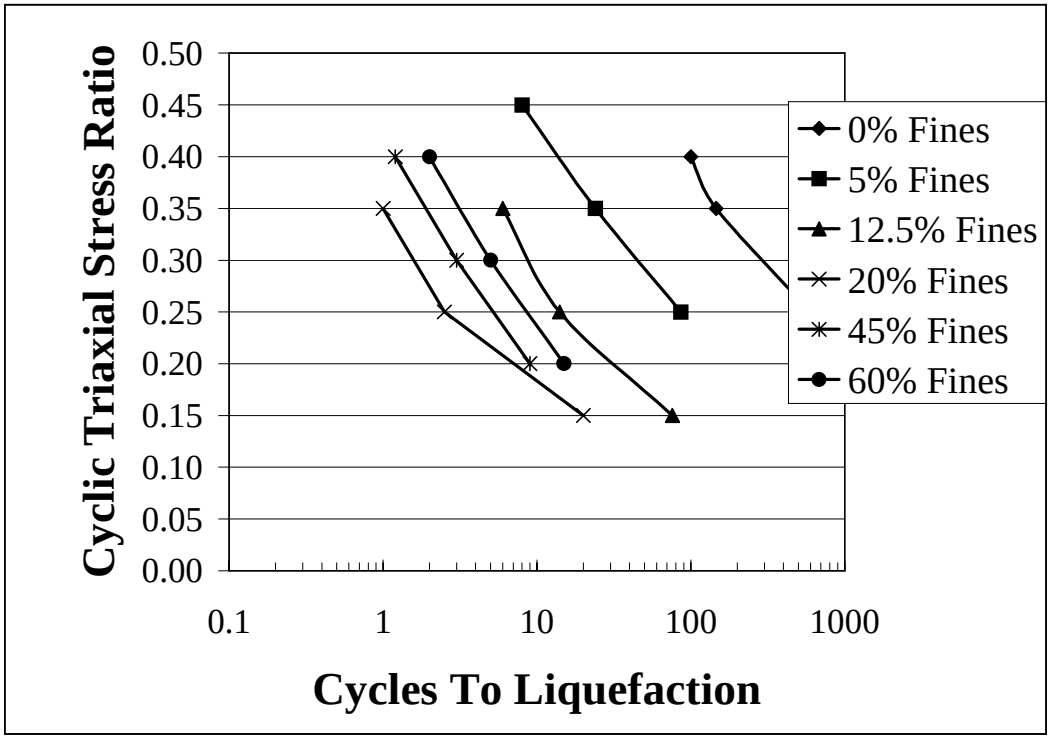

Figure 2-3: Decrease and then increase in cyclic resistance with increase in silt content (After Koester, 1994)

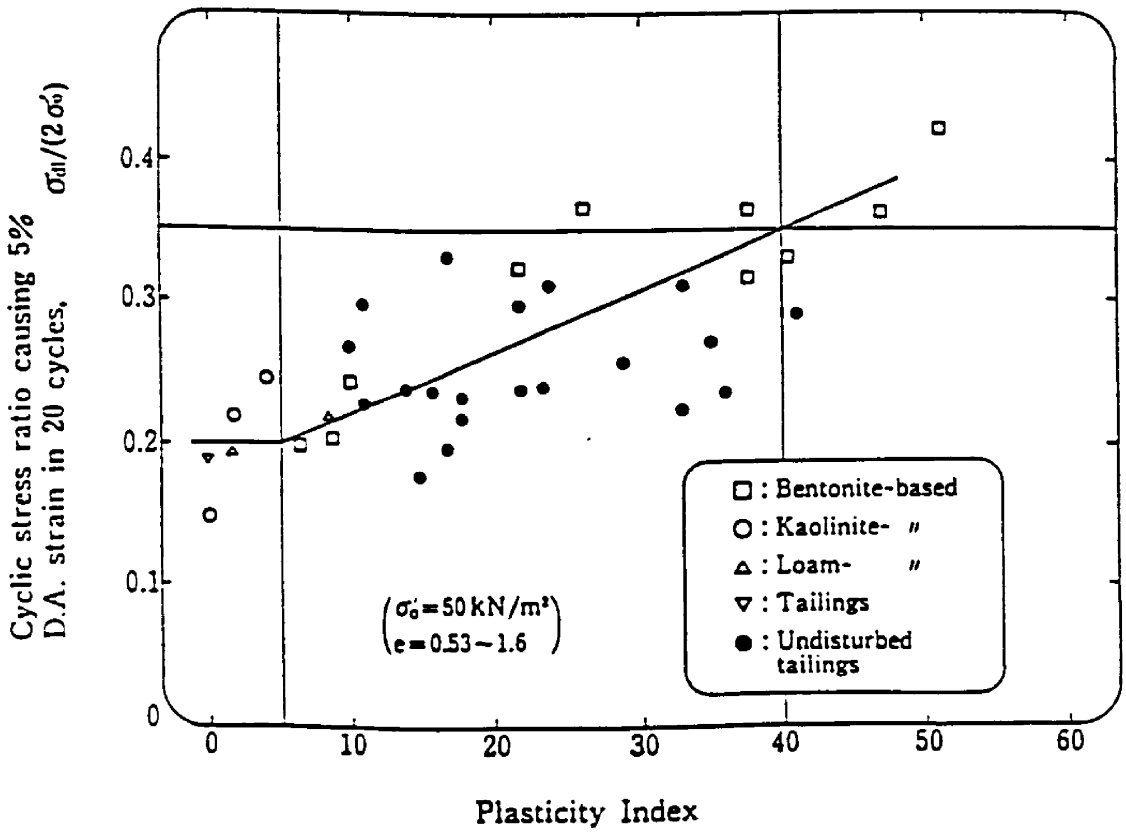

Figure 2-4: Increase in cyclic resistance with increase in plasticity index
(From Ishihara and Koseki, 1989)

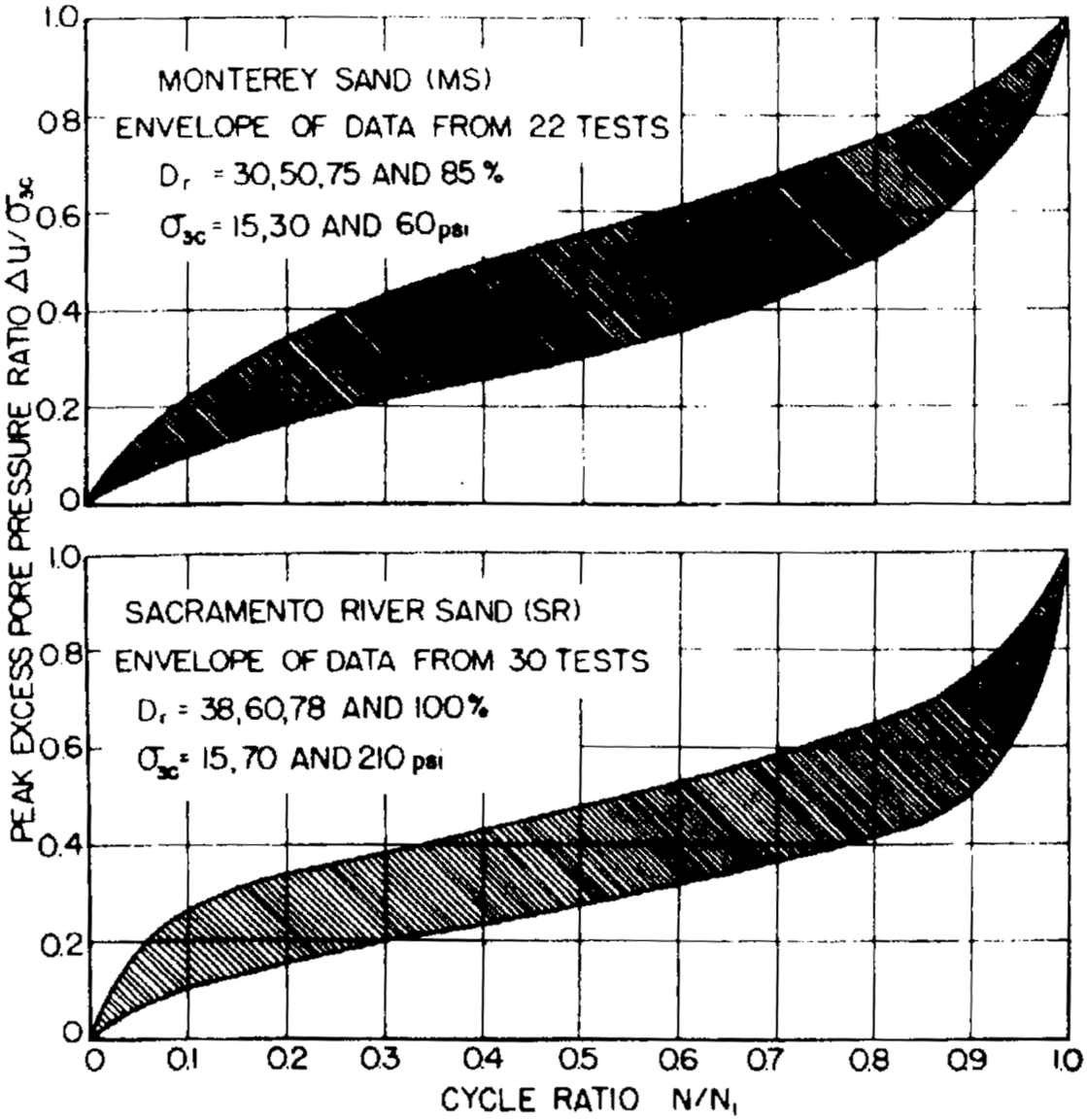

Figure 2-5: Pore pressure generation characteristics for two sands
(From Lee and Albaisa, 1974)

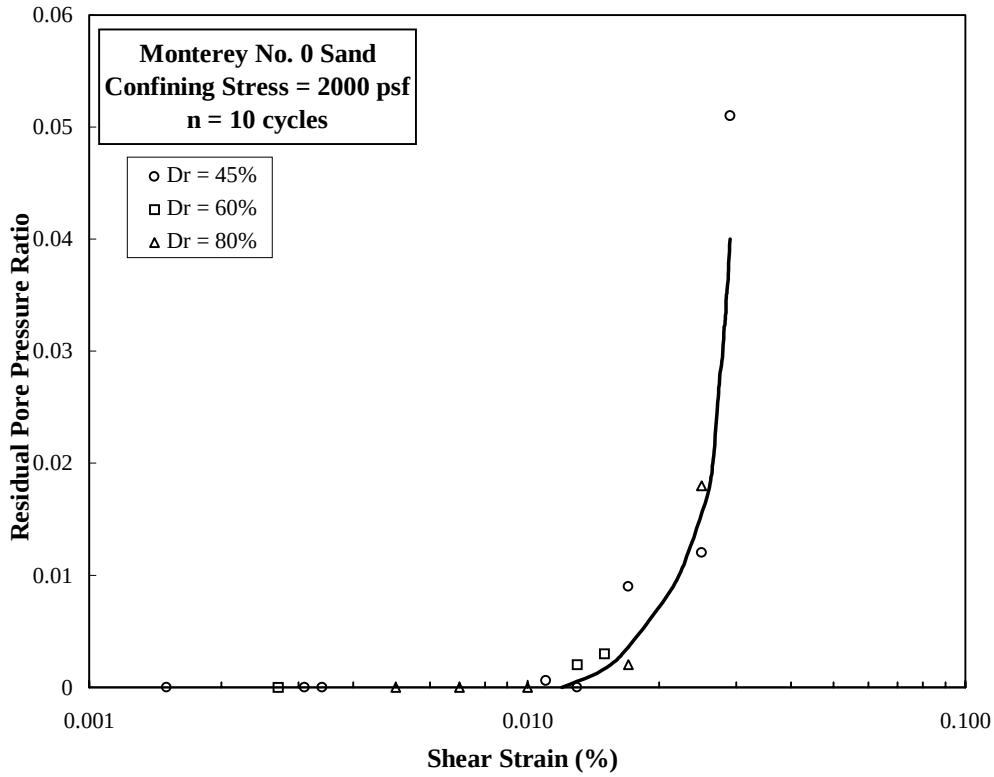

Figure 2-6: Pore pressure generation characteristics as a function of shear strain (After Dobry et al, 1974)

CHAPTER 3: THE LABORATORY TESTING PROGRAM

This chapter discusses the details of the laboratory testing program performed to clarify the manner in which fines content and plasticity affect the liquefaction susceptibility of sandy soils. First, the component soils and soil mixtures tested are discussed. This discussion focuses on the index testing program and the index properties of the soils. Next, a brief review of cyclic triaxial testing is presented with an emphasis on the differences between in-situ and laboratory conditions and loadings. Lastly, following an explanation of some of the testing parameters, which affect cyclic strength, a detailed test methodology is presented.

3.1 Soils Tested

In order to determine the effects of fines content and plasticity on the liquefaction susceptibility of sandy soils, cyclic triaxial tests were performed on specimens formed from various mixtures of sand, silt, and clay. These soil mixtures were formed by adding fine-grained materials to one of the two base sands. The fine-grained material consisted of
non-plastic silt, kaolinite, bentonite or various combinations thereof.

3.1.1 Sands

Two base sands were used in the study. The first was Yatesville sand, which consists of the coarse fraction of Yatesville silty sand, which was obtained from a dam site in Louisa County, Kentucky. It is a poorly graded, medium to fine sand, having approximately 99 percent passing the No. 20 sieve (0.84 mm), 45 percent passing the No. 100 sieve (0.15 mm), and a mean grain size, D_{50}, of 0.18 mm. It is light brown in color, has a specific gravity of 2.72, and its grains are sub-angular to sub-rounded in shape.

The second base sand used in the testing program was Monterey No. 0/30 sand, a commercially available sand from California. It has the same mineralogy and a similar gradation to Monterey No. 0 sand, which has been used in numerous liquefaction studies in the past (Silver et al., 1976). Monterey No. 0/30 sand is a poorly graded, medium to fine sand, having over 98 percent passing the No. 20 sieve (0.84 mm) and retained on the No. 100 sieve (0.15 mm). It has a mean grain size, D_{50}, of 0.43 mm. It is brown in color, has a specific gravity of 2.65, and its grains are sub-angular to sub-rounded in shape. With a smaller percentage of material passing the Number 100 sieve and a larger mean grain size, the Monterey No. 0/30 sand is somewhat coarser than the Yatesville sand.

3.1.2 Silt

The silt used in the study was derived from the fine-grained portion of the Yatesville silty sand. It has a maximum grain size of 0.074 mm, a minimum grain size of 0.004 mm, and a mean grain size, D50, of 0.03 mm. The silt is light brown in color, has a specific gravity of 2.78, and, as it has no discernible liquid or plastic limit, classifies as a non-plastic silt (ML).

3.1.3 Clays

The two clays used for the plastic portion of the fine-grained material consisted of commercially available kaolinite and bentonite. The kaolinite used in this study had been obtained by Virginia Tech prior to the beginning of this research program from a commercial supplier in Georgia. It is white in color, has a specific gravity of 2.65, and classifies as a highly plastic clay (CH) with a liquid limit of 58, a plastic limit of 27, and a plasticity index of 31.

The bentonite used in this study was obtained from a commercial supplier for a study of slurry walls concurrently on-going at Virginia Tech. It is light gray in color, has a specific gravity of 2.65, and classifies as a highly plastic clay (CH) with a liquid limit of 385, a plastic limit of 42, and a plasticity index of 343.

Complete index data for the component soils are presented in Table 3 -1. Grain size distribution curves for the sands and silt are presented in Figure 3-1.

3.1.4 Soil Mixtures with Non-Plastic Fines

In order to study the effects of non-plastic fines content on the liquefaction resistance of sands, eight combinations of sand and silt were created using each base sand. Silt contents of the soil mixtures varied from 4 to 75 percent by weight. Additionally, tests were performed using each of the clean base sands and the pure silt. A complete listing of the soil mixtures tested, as well as information on their complete index properties, are presented in Table 3-2 for the soil mixtures using Yatesville sand as the base sand and Table 3-3 for the soil mixtures using Monterey No. 0/30 sand as the base sand.

3.1.5 Soil Mixtures with Plastic Fines

In order to study the effects of plastic fines content and fines plasticity on the liquefaction resistance of sands, combinations of sand and fine-grained material were created using the Yatesville sand. Fines contents of the soil mixtures varied from 4 to 37 percent by weight, with clay contents varying from 2 to 37 percent. The fine-grained material consisted of either pure kaolinite, pure bentonite, or a mixture of the two. Additional tests were run with a combination of silt and clay as the fine-grained material. A complete listing of the soils mixtures tested, as well as information on their plasticity, is presented in Table 3-4.

3.2 Index Testing

A complete index testing program was performed on the two base sands, the Yatesville silt, and on all soil mixtures in which the fines where non-plastic in nature. Index properties including grain size distribution, specific gravity, and maximum and minimum

index void ratios were determined for each soil mixture. Additionally, plasticity tests were performed on those soil mixtures that contained either kaolinite or bentonite.

Details of the index tests performed are presented below. The index properties of the individual soil mixtures are summarized in Table 3-2 for the Yatesville sand and silt mixtures, in Table 3-3 for the Monterey sand and silt mixtures and in Table 3-4 for the Yatesville sand and plastic fines mixtures.

3.2.1 Grain Size Characteristics

The grain size distribution of each soil was determined in general accordance with ASTM D 422, Standard Test Method for Particle Size Analysis of Soils. Both mechanical grain size analyses and hydrometer tests were performed. The mean grain size D_{50}, the coefficient of uniformity, C_u, and the coefficient of curvature, C_c, for each sand, silt, and mixture of silt and sand are presented in Tables 3-1 through 3-3.

3.2.2 Maximum And Minimum Void Ratios

There is no ASTM procedure for determining maximum density for cohesionless soils which is applicable over the entire range of silt contents investigated. The vibratory table method (ASTM D 4253, Standard Test Method For Maximum Index Density Of Soils Using A Vibrating Table) is limited to soils with a maximum fines content of 15 percent, while proctor tests do not always produce accurate, repeatable results for clean sands. Therefore, vibratory table as well as both Standard and Modified Proctor tests were performed upon each sand and silt mixture. In agreement with the findings of Lee and Fitton (1968), the vibratory table tests yielded maximum dry densities similar to those produced by the Modified Proctor test. Because the vibratory table tests were found to give more repeatable results, they were used to define the maximum index densities and minimum index void ratios used in this study.

Similarly, there is no ASTM procedure for determining minimum density for cohesionless soils which is applicable over the entire range of silt contents investigated in this study. The minimum density method (ASTM D- 4254, Standard Test Method For Minimum Index Density Of Soils And Calculation Of Relative Density) is limited to soils with a maximum fines content of 15 percent. Despite this limitation, the maximum index void ratio and corresponding minimum index density for each soil mixture was determined in general accordance with the method presented in the specification. Both Method B and Method C of this standard were performed and found to give similar results. Method B consists of filling a tube, the base of which is sitting within a mold of known weight and volume, with dry soil and then slowly lifting the tube so that the soil flows out and fills the mold. The soil is then struck level with the top of the mold, the mold and soil weighed, and the density determined. Method C consists of partially filling a glass cylinder with a known weight of dry soil and slowly inverting it. The height of the soil in the cylinder is then measured and the density determined. The values of minimum index density and maximum index void ratio determined by these two methods were similar. Those determined using Methods B were found to be slightly more consistent, and were used in the study.

Complete results of the index density testing program perfromed on the Yatesville sand and silt mixtures are included in Appendix C: Index Density Testing.

The maximum and minimum void ratios determined for the sand and silt mixtures were assumed to remain essentially constant when the non-plastic silt was replaced with plastic fines and thus were assumed to be applicable for the soils with corresponding plastic fines contents. A plot of the maximum and minimum index void ratios versus silt content for both the Monterey and Yatesville sand mixtures is presented in Figure 3-2.

3.2.3 Specific Gravity

Specific gravities were determined in general accordance with ASTM D 854, Standard Test Method For Specific Gravity Of Soil for each sand silt and clay. The specific gravity for each soil mixture was then determined based upon the percentage of each component soil present the mixture. The specific gravities for each sand, silt, and mixture of silt and sand are presented in Tables 3-1 through 3-3.

3.2.4 Soil Plasticity

The plasticity of the various soil mixtures with plastic fines was determined for the fraction of the material passing the No. 40 sieve in general accordance with ASTM D 4318 Standard Test Method For Liquid Limit, Plastic Limit, And Plasticity Index Of Soils.

First, the Yatesville silt, the bentonite, and the kaolinite were tested in their pure form to determine their Atterberg Limits. For the silt it was impossible at any water content to prevent the groove from closing in less than seven blows, therefore the silt was deemed non-plastic in accordance with Section 11.4 of the standard. Both liquid and plastic limits were determined for the bentonite and kaolinite.

Liquid and plastic limit tests were determined for each soil mixture containing plastic fines. Some soils, most notably those with low clay contents, were found to have liquid limits but not plastic limits and were thus identified as being non-plastic. Atterberg limit data for the soil mixtures containing clayey fines are presented in Table 3-4.

3.3 Cyclic Triaxial Testing

Cyclic triaxial tests attempt to model the loads applied to a soil mass by an earthquake. There are several differences between the loads applied to a soil mass in the field and

those applied to a specimen in a cyclic triaxial test. Additionally, numerous factors can affect the resistance to liquefaction developed in a cyclic triaxial specimen and make it differ from that which might be developed in the field. These include the method of specimen preparation, the relative density of the specimen, the particle size and gradation of the soil, the size of the specimen, the ratio of principal stresses during consolidation, the shape of the loading function, the frequency of application of the loading function, and the degree of saturation in the specimen.

Following a brief review of the basic theory behind cyclic triaxial testing, the differences between cyclic triaxial and in-situ loadings will be discussed. Next, the factors that may affect a cyclic triaxial specimen are summarized, and lastly a review of the definition of liquefaction is presented.

3.3.1 Basic Theory Of Cyclic Triaxial Testing

The liquefaction resistance or cyclic strength of a soil is often measured in the laboratory using reconstituted specimens tested in cyclic triaxial tests. The specimen is formed within a latex membrane inside the triaxial cell, saturated, consolidated to some stress condition and then loaded with a pulsating deviator load. As the deviator load cycles between compression and tension, pore water pressures in the specimen increases, the effective stress decreases, and the specimen undergoes axial straining. The specimen is said to have liquefied when either the pore pressure becomes equal to the initial effective confining stress (i.e. when the effective stress acting on the specimen becomes equal to zero) or when some level of axial strain is achieved.

The cyclic stress ratio is the ratio of the applied shearing stress to the effective confining stress. In a cyclic triaxial test, the applied shearing stress on the plane of interest is taken to be one-half of the applied deviator stress. Therefore the cyclic stress ratio is simply the ratio of the applied deviator stress to twice the initial effective confining stress. The

CSR is then plotted against the number of cycles of deviator stress required to cause liquefaction. This process is repeated for several cyclic stress ratios and a cyclic resistance curve is produced. A cyclic resistance curve for Monterey # 0 sand tested at a relative density of 60 percent is shown in Figure 3-3 (Silver et al., 1976).

3.3.2 The Differences Between Cyclic Triaxial And In-Situ Earthquake Loadings

Cyclic triaxial tests are commonly used to measure the cyclic strength or liquefaction resistance of soils. The manner in which the stresses are applied to an element of soil in the field are quite different than the manner in which the stresses are applied in a cyclic triaxial test. Seed and Lee (1966) present the actual shear conditions affecting a soil element in the field under level ground, which consist of a series of horizontal, reversing shear stresses acting on a horizontal planes in conjunction with a constant vertical stress. These shear stresses result from the upward propagation of shear waves through the soil column. An idealized representation of these stress conditions provided by Seed and Lee (1966) are shown in Figure 3-4.

Cyclic triaxial tests attempt to model these stress conditions by applying a pulsating deviatoric stress to the specimen while maintaining a constant confining stress on the specimen and preventing drainage. The stresses on a plane inclined at 45 degrees to the axis of the specimen are similar, though not identical, to the stresses acting on a horizontal plane in the field.

3.3.2.1 Change in Total Confining Stress

The primary difference between the field conditions and those on an inclined plane within the specimen is an increase or decrease in the all-around confining stress equal to one-half the deviatoric stress. That is, in order to match the stress conditions experienced in the field, it would be necessary to decrease or increase the cell pressure by one half the deviator stress as the deviator stress is respectively increased or decreased. This combination of stress conditions as originally presented by Seed and Lee (1966) is shown

in Figure 3-5. However, because the specimen is saturated, this change in cell pressure would be mirrored by a corresponding change in pore pressure, and thus there would be no change in effective stress within the specimen. Because the deformation of the specimen is a function of the effective, not the total, stresses acting on it, the same deformations will occur whether the cell pressure is cycled or not. The recorded pore pressure will, however, be off by something between plus or minus one half the deviator stress, dependent upon where in the cycle it is measured.

3.3.2.2 Stress Paths

Another major difference between the field and laboratory loadings can be seen in the stress paths for the two conditions. As shown in Figure 3-6, because the field loading consists of applying a component of pure shear to an anisotropically consolidated element, the stress path is simply a vertical line which never crosses the zero shear stress (i.e. $q = 0$) axis. This shows that at no point in the loading is the specimen in the ground under a condition of zero shear stress. The stress path for the cyclic triaxial test however can be seen in Figure 3-7 to cross the zero stress boundary twice during each cycle of loading.

3.3.2.3 Rotation of Principal Stresses

Another difference in the loading patterns undergone by a soil element in the field and a soil specimen in a cyclic triaxial test involves the rotation of principal stresses. Assuming a level ground condition, the horizontal and vertical planes are the planes of principal stress prior to loading for both an element in the field and a specimen in a triaxial cell. For the element in the field, as a component of pure shear is added to the horizontal plane (and a complimentary shear stress develops on the vertical plane), the orientation of the planes of principal stress rotate smoothly to some new orientation which is dependent upon the stresses applied.

For the triaxial specimen, only a rotation of zero or 90 degrees is possible due to the axial nature of the loading. In other words, the principal stresses in a triaxial test will always act vertically and horizontally and any rotation in principle stress that does occur, occurs because of a reversal of the direction of the major and minor principle stresses. Such a rotation of principle stresses occurs during the transition from the loading to the unloading portion of the cycle during a test on an isotropically consolidated specimen. As the axial load decreases to below the cell pressure, the major principle stress rotates from the vertical to the horizontal direction. Similarly, at the transition from the unloading to the loading portion of the cycle the major principle stress rotates from the horizontal direction back to the vertical direction.

3.3.2.3 Intermediate Principal Stress

The stress conditions acting on a specimen in a triaxial cell also differ from those acting on an element in the field with regard to the intermediate principal stress. For an isotropically consolidated specimen in a triaxial test, the intermediate principal stress, which always acts horizontally, is equal to the minor principal stress during the compression portion of the cycle, when the major principle stress is acting vertically. It is equal to the major principal stress during the extension portion of the cycle, when the major principle stress is acting horizontally.

For anisotropically consolidated specimens, whether the intermediate principal stress alternates between the major and minor principal stresses depends upon the deviator stress applied during consolidation and the deviator stress applied during shear. If the deviator stress applied during consolidation is larger than the deviator stress applied during shear, the vertical stress never decreases to a value less than that of the horizontal stress. Therefore, the axial stress is always the major stress and the intermediate principal stress is always equal to the minor principal stress. If the deviator stress applied during consolidation is smaller than the deviator stress applied during shear, the intermediate

principal stress will alternate between the major and minor principal stresses as occurs in the isotropically consolidated specimen.

These differences in intermediate principal stress in turn affect the mean confining stress, which is generally taken to be the average of the three principle stresses. Ishihara (1993) has shown that the cyclic resistance of a soil is more closely related to its mean effective confining stress than to either its vertical or horizontal effective confining stress.

3.3.2.4 Uniformity of Loadings

The loading that a soil element in the ground receives during an earthquake is non-uniform, both in terms of its amplitude and their frequency. These variations would be very hard to model in the laboratory. To overcome this difficulty the concept of equivalent uniform cycles is applied (Seed and Idriss, 1971).

The loading applied to a specimen during a cyclic triaxial is essentially constant in both amplitude and frequency and is most often applied in the form of a uniform, repeating sine wave. The cyclic stress ratio applied is usually assumed to be a function of some percentage (most frequently 65 percent) of the maximum acceleration of the design earthquake. Thus by assuming the applied stress (or acceleration) to be some percentage of the peak stress (or acceleration) the loading can be converted from the non-uniform, erratic loading seen in the field, to a uniform equivalent loading, easily applicable in the laboratory.

3.3.2.5 Components of Loadings

An actual earthquake would apply stresses to a soil element in the field in several directions simultaneously. This multi-directional loading can be seen in Figure 3-8 that shows three components of the acceleration time history recorded at Yerba Buena Island during the 1989 Loma Prieta earthquake. Two perpendicular horizontal and one vertical components were recorded. A cyclic triaxial test, however, only employs one component

of loading. This is analogous to one horizontal component of the earthquake motion acting on the plane inclined at 45 degrees within the specimen. Studies by Seed and Pyke (1975) have shown that the difference in loading between the multi-component loading in the field and the single component loading in the lab results in the laboratory specimens failing at a stress which is approximately 10 percent higher than that required to cause failure in the field.

3.3.2.6 Correction Factors

The net result of the differences between the stress conditions in the field and in the lab is that specimens tested in the laboratory fail at cyclic stress ratios which are higher than those which would be required to fail similar specimens in the field in the same number of cycles. In order to apply the cyclic resistances obtained in the lab to soils in the field, the cyclic resistances must be corrected before it is applied to the field conditions. This is most commonly done by applying a reduction factor, Cr, to the CSR's determined in the lab. This reduction factor is commonly taken to be equal to 0.57 for soils with an at-rest lateral earth pressure coefficient, K_o, of 0.4 and to be equal to 0.9 for soils with a K_o value of 1.0 (Seed, 1979).

3.3.3 Factors Affecting Cyclic Resistance

Numerous factors affect the resistance to liquefaction measured in a cyclic triaxial specimen. These factors include the mean grain size of the soil, the void ratio and the relative density of the specimen, the method of specimen preparation, the size of the specimen, the ratio of the principal stresses acting on the specimen during consolidation, the shape of the loading function, the frequency of loading, and the degree of saturation in the specimen. Silver et al. (1976) and Townsend (1978) have provided excellent reviews of many of these factors.

For a given sand tested at a specified density, several of these factors, including mean grain size, void ratio, and relative density, are beyond the control of the test operator. The

remaining factors are a function of the test methodology and can be controlled by the operator. The operator-controlled factor that has the greatest effect on the liquefaction susceptibility of a specimen is the method used to form the specimen. Other factors include specimen size, the shape of the loading function, the frequency of loading, the ratio of principle stresses during consolidation, the degree of saturation in the specimen, and the membrane and area corrections applied to the test data. A brief discussion of the effects of each of the operator dependent parameters and how they were handled in this study follows.

3.3.3.1 Specimen Preparation Method

Moist tamping in multiple layers was chosen as the method of specimen preparation for this project. While it does not mimic the deposition processes of natural deposits, it offers several advantages over either pluviation or vibration. Moist tamping eliminates the problems of particle segregation associated with pluviation through either air or water and it is capable of producing specimens with consistent void ratios over a fairly wide range of densities. These factors make it ideal for a parametric study such as this in which the two main factors being evaluated are fines content and density.

The current study was not intended to exactly mirror field conditions, but to identify and quantify the changes in liquefaction resistance which occur as the fines content of a sand increases. While the moist tamping method of sample preparation does not mimic the natural deposition processes of silty sands, it was selected because of the high level of control over sample density it provides. Although the moist tamping method produces somewhat different soil fabrics and cyclic resistances than might be found in natural soil deposits (although not necessarily different than those found in man-made fills), because a consistent method of sample preparation was used, these differences should affect all of the specimens tested. Therefore, while the specific magnitudes of the parameters measured may be different from those determined using other sample preparation

methods, because these differences occur in each specimen tested, they should not alter the trends determined in this study.

In order to ensure that the specimens are consistent in density throughout the specimen, the specimens were produced using the undercompaction technique recommended by Ladd (1978) and modified by Chan (1985). As each layer is placed, not only the layer being placed, but also the layers below it, are densified. This means that if an uniform weight of soil and a uniform layer height were used throughout the compaction process, the resulting specimen would be denser at the bottom than at the top. To compensate for this, layers are placed at increasing density from bottom to top. Chan's method calls for a constant difference in relative density of one percent between each lift, with the lifts below the middle lift placed at lower relative densities than the average, the middle lift placed at the average relative density desired, and the lifts above the middle lift placed at higher relative densities than the average. For example, if an overall relative density of 50 percent is desired and a difference of one percent relative density between layers is selected, the seven layers would be placed from bottom to top at relative densities of 47, 48, 49, 50, 51, 52, and 53 percent respectively. Because the lift height was held constant, the weight of soil in each layer was varied to achieve the desired relative density.

3.3.3.2 Specimen Size

Lee and Fitton (1969) prepared and tested coarse and fine sands with different specimen diameters in order to determine the effects of specimen size on cyclic strength. The specimens tested were either 1.4 inches or 2.8 inches in diameter, were prepared to a constant relative density, and were tested at a constant confining pressure. They found less than a 5 percent variation in liquefaction resistance between the two specimen diameters with the larger specimens being slightly weaker. This variation, however, was within the scatter in the data and was therefore not considered to be a significant effect.

For this study, all specimens were constructed with a diameter of 2.8 inches (71 mm) using seven lifts with a constant lift height of 0.87 inches (22 mm). This produced specimens with a pre-consolidation height of 6.06 inches (154 mm). The resulting height to diameter ratio of 2.2 is within the range of 2 to 3 recommended by ASTM for triaxial specimens.

3.3.3.3 Shape Of The Loading Pattern

Silver et al. (1976) and Mulilis, Townsend and Horz (1978) found that the cyclic strength of a soil is strongly influenced by the shape of the loading pattern. Several waveforms were examined including, in order of increasing cyclic strength produced, rectangular, degraded rectangular or triangular, and sine waves. Sine waves were found to produce cyclic strengths, which were 15 to 30 percent higher than those, produced under rectangular waves of the same maximum amplitude do. This strength increase is due to the severe velocity changes that occur during loading with the rectangular form and their effect on pore pressure. Additionally, a sine wave applies a lower average load than a rectangular wave with the same peak amplitude due to its longer rise time. This lower average stress results in a higher cyclic strength. The effects of varying wave shapes on cyclic stress can be seen in Figure 3-9 (after Silver et al. 1976) which plots cyclic resistance curves for Monterey #0 sand under various wave forms.

While the testing system offers several options as to the shape of the loading function, a sinusoidal shaped function was used as recommended by Silver (1977).

3.3.3.4 Frequency Of Application Of Loading

Several investigators, including Lee and Fitton (1968) and Mulilis (1975), have examined the effect of the frequency of loading on cyclic strength. Lee and Fitton found that slower loading frequencies produced slightly lower cyclic strengths, while Mulilis found the opposite to be true. As neither study found that varying the frequency between 1 and 60 hertz varied the strength by more than 10 percent, it would appear reasonable to assume

that small changes in frequency of loading, such as from 1/2 to 1 Hz, should not have any measurable effect on the cyclic strength of the soil being tested. Tests performed by the author on Monterey No. 0/30 sand show that this is indeed the case. The results of these tests are shown in Figure 3-10, where it can be seen that for specimens tested at 1 Hz and ½ Hz there is no difference in the cyclic stress ratio required to cause initial liquefaction in either 10 or 15 cycles.

Although some of the early tests on the Monterey sand and the Monterey sand and silt mixtures were run at a period of 2 seconds (a frequency of 0.5 Hertz), a period of 1 second (a frequency of 1 Hertz) was used for the majority of the testing program.

3.3.3.5 Consolidation Ratio

Lee and Seed (1967b) tested specimens of a medium sand, a fine sand, and a compacted silt consolidated to various principal stress ratios in order to determine the effect of consolidation stress ratio on cyclic strength. They found that for a given density and minor principle consolidation stress, the deviator stress required to produce 20 percent double amplitude strain in 100 cycles increased with increasing consolidation ratio. Ishihara (1996) has shown that this trend is due to the increase in mean consolidation stress that occurs with increasing consolidation ratio.

All specimens tested in this study were isotropically consolidated to an effective stress of 100 kilopascals (approximately 14.7 psi). As the consolidation ratio for the specimen was unity, the effects of changing consolidation ratio and mean effective stress were not encountered.

3.3.3.6 Degree Of Saturation

Mulilis, Townsend, and Horz (1978) examined the effect of the degree of saturation in a specimen on its liquefaction characteristics. They found that in tests on Monterey sand there was no significant differences in liquefaction tendencies in specimens with values

of Skempton's pore pressure parameter B (Skempton 1954) varying between 0.91 and 0.97. Chaney (1976) however found that varying the B value between 0.91 and 0.99 could have a large effect on the cyclic strength of the specimen depending on soil type, density and initial confining pressure. Therefor for this study a minimum acceptable B value of 0.94 was chosen. This is in line with the recommendations of Silver et al. (1976) and Ladd (1978), and is higher than the minimum value of 0.91 advocated by Mulilis, Townsend, and Horz (1978).

3.3.3.6 Membrane And Area Corrections

The latex membranes used to isolate the soil from the cell water had a thickness of 0.0125 inches (0.32 mm). Due to the thinness of the membrane and in keeping with the standard of practice (Silver, 1977), no membrane correction was applied to the measured data. Additionally, no area correction was applied to the cyclic triaxial loading data in accordance with standard practice as outlined by Chan (1985).

3.3.4 Definition of Liquefaction

The point at which liquefaction occurs during a cyclic triaxial test can be defined in several ways. The most common definition used in laboratory testing is that liquefaction has occurred when the pore pressure in the specimen first equals the initial effective confining (or isotropic consolidation) stress. This results in a temporary condition of zero effective stress in the specimen and is usually referred to as initial liquefaction.

Liquefaction is also often defined using some level of single or double amplitude axial strain. Single amplitude axial strain is defined as the total strain that occurs during a half cycle of loading, either in tension or compression. Double amplitude axial strain is defined as the total strain that occurs between any two adjacent peak compressive and tensile strains, as is shown in Figure 3-11. Values of 2.5 percent, 5 percent, 10 percent, or 20 percent are commonly used to define liquefaction. For this study, the number of cycles of loading required to cause initial liquefaction, 1 percent double amplitude axial

strain, and 2.5 percent double amplitude axial strain was determined for each test. Some tests, most notably those with high relative densities, large silt contents, or a large amounts of plastic fines, did not achieve 5 percent double amplitude axial strain. The number of cycles of loading required to cause 5 percent double amplitude axial strain was however determined for each test that achieved that strain level.

3.4 Testing Equipment And Test Methodology

3.4.1 Testing Equipment

The cyclic triaxial testing for this study was performed on an automated triaxial testing system designed by Clarence Chan and built by the Soil Engineering Equipment Company of San Francisco, CA. This systems uses closed-loop feedback systems to control the loadings. A five-channel signal processor built by Paul Gross Associates of Scottsdale, AZ controls both the data acquisition system and the closed loop feedback system.

The equipment is controlled by software run on a dedicated personal computer. The system software performs a wide variety of functions and tests automatically, including backpressure saturation, isotropic and anisotropic, including K_o, consolidation, and drained and undrained, monotonic and cyclic, triaxial tests. It also allows for translation of the recorded data to ASCII format, and for the graphical plotting of the data.

During testing, five channels of data were measured by the system, transferred to the dedicated computer via a data acquisition card, and stored a test specific data file. Axial load was measured using a load cell, axial displacement was measured using an LVDT, and cell pressure, effective pressure (cell pressure minus pore or back pressure), and volume change were all measured using pore pressure transducers.

3.4.2 Test Methodology

The cyclic triaxial test procedure used during this study was closely based upon the procedures recommended by M. L. Silver (1977) in his report to the U. S. Nuclear Regulatory Commission. Slight modifications to these methodologies were made, most notably in the details of the moist tamping method used in the specimen preparation.

3.4.2.1 Specimen Preparation

After the desired void ratio and density for the test had been determined, the soil was prepared by weighing the proper weight of dry soil for each of the seven lifts into separate containers. The appropriate amount of water was then added to the dry soil. The soil and water were then mixed and the containers sealed to prevent changes in water content. Soils with large fines contents were allowed to sit overnight to insure uniform water content distribution throughout the soil.

The weight of water added to each lift was determined by the water content required to achieve 50 percent saturation in the tamped lift. Fifty percent saturation was chosen instead of the 70 percent recommended by Silver (1977), because it allowed for the preparation of specimens over a greater range of densities.

Prior to the beginning specimen preparation, the volume change device was filled with deaired water. The effective stress and the volume change transducers were then bled to insure accurate measurements of their respective parameters.

Next the sides of the lower platen of the triaxial cell were coated with a water-based lubricant, the end of the latex membrane slipped over the platen and secured in place with two rubber O-rings. At this time, two rubber O-rings which were later used to secure the membrane to the top platen were placed over the top of the forming jacket. Once the membrane had been secured to the lower platen, a sheet of filter paper was wrapped

around the lower platen and the membrane. The specimen forming jacket was then slipped over the membrane and platen and checked to assure that it was level. The filter paper between the jacket and the membrane was used to ensure that the membrane did not seal off the vacuum holes in the forming jacket, thus allowing for a uniform application of the vacuum around the jacket. A vacuum was applied to the forming jacket by means of a vacuum line, the membrane carefully pulled over the top of the jacket, and then pulled tight against the jacket by the vacuum. At this time any wrinkles or loose spots in the membrane were removed. The top jacket was again checked to ensure that it was level, insuring a specimen with vertical sides which were perpendicular to the top and bottom surfaces.

Once the membrane and jacket have been prepared, the tamper support arms used to position and support the soil tamper cross bar during specimen preparation were attached to the posts that support the top of the triaxial cell. The soil tamper was then zeroed by setting the cross bar of the soil tamper on the tamper support arms and adjusting the two sliding blocks on the tamper rod so that, when the tamping foot was resting on the lower platen, the blocks were in contact with each other and the top of the cross bar. With the soil tamper thus zeroed, the height of the tamping foot above the top of the lower platen, which is the specimen height at any time during specimen construction, is simply the distance between the two sliding blocks. This distance, and thus the height of the top of the lift being constructed, was controlled through the use of a series of spacer bars corresponding to the individual lift heights.

The soil for the first lift was spooned into the jacket and approximately leveled with the spoon. The height of the tamper foot corresponding to the first layer was set using the appropriate spacer bar to locate the lower sliding block. The soil tamper was then set on the support arms with the tamping foot pulled up above the level of the uncompacted soil. With the edge of the tamper foot against the membrane, the soil was pressed down to its correct height by moving the tamping foot around the inner edge of the jacket. This

tamping process requires several trips around the jacket, beginning lightly to allow the soil to become uniformly distributed throughout the lift, and then with increasing pressure to bring the soil to its final density. As the tamping foot was one-half the diameter of the jacket, by moving the foot around the inner edge of the jacket, the soil was uniformly compacted across the surface of the layer.

Once the layer had been compacted to its correct height, the tamper was removed and the soil surface was scarified to a depth of approximately 2 millimeters. The soil for the next lift was then spooned into the jacket and the process was repeated until the soil for the uppermost layer had been spooned into the jacket. The uppermost layer was compacted to a height approximately 1/64th of an inch greater than its final height, and the tamper and the tamper support arms removed.

3.4.2.2 Final Specimen Preparation And Equipment Assemblage

The top of the triaxial cell (with the upper platen attached to the axial loading rod) was then bolted into place. The sides of the upper platen were then coated with a water-based lubricant, the upper drainage line attached, and the platen lowered onto the top of the specimen. The top of the axial loading rod was then lightly tapped with a mallet until the desired specimen height was achieved. This process assured both good contact between the upper platen and the specimen and a correct specimen height. Once the axial loading rod had been locked into place, the membrane was pulled off the forming jacket and onto the top platen. It was then secured with the O-rings that had previously been placed around the forming jacket.

At this point a small vacuum (approximately 3 psi.) was applied to the specimen through the top drainage line to provide a positive effective stress on the specimen. Once this internal vacuum had been applied, the external vacuum applied to the membrane during specimen preparation was removed by disconnecting the vacuum hose. Next, the forming jacket was removed from the specimen, the O-rings repositioned so that they were

securely fitted into the grooves in the sides of the platens and the specimen dimensions measured and recorded. The diameter of the specimen was measured to the nearest 0.001 inch at three heights (approximately the quarter points of the specimen) using a Pi tape. These values were then averaged and corrected for the membrane thickness. The height of the specimen was again measured to the nearest 0.01 inches by measuring the height that the axial loading rod protrudes above its locking collar. This measurement was then converted to the specimen height by comparing it against a similar measurement made on a specimen blank of known height, which had previously been recorded. These measurements and other relevant parameters were recorded on a data sheet such as that shown in Figure 3-12.

Once the specimen's dimensions were known, the density, void ratio, relative density, and initial degree of saturation of the specimen were calculated. If these values were within the acceptable ranges, the acrylic cylinder that forms the walls of the triaxial cell was lowered into place and the cell filled with water to within approximately one-half inch of the top. This air gap was left at the top to allow the axial load rod attached to the upper platen to move in and out of the cell without affecting the cell pressure.

The cell assembly was then placed into the loading frame and the axial loading rod was attached to the load cell with a threaded coupling. The load frame cross bar was bolted into place, the cell pressure line attached to the cell via a valve in the cell top, and the LVDT roughly centered. This initial LVDT reading was recorded on the data sheet to allow for the monitoring of the changes in specimen height which occur during back pressure saturation and consolidation. The computer program TRIAXIAL that operates the system was started at this time and when the pressure source was turned on a cell pressure of 20 kPa (2.9 psi.) was applied to the specimen. The drainage valves on the cell were now closed and the vacuum removed from the specimen.

3.4.2.3 Initial Specimen Saturation

In order to improve the initial saturation of the specimen, carbon dioxide (CO_2) was allowed to flow through the specimen at a low pressure in order to replace the air in the specimen pores. This was done because CO_2 has a much higher solubility in water than does air, allowing a higher degree of saturation to be reached at lower backpressures.

Once a stream of nearly pure CO_2 was found to be flowing out of the specimen, which normally took 7 to 10 minutes, the flow of CO_2 was stopped, and a tank of deaired water was attached to the drainage line on the bottom platen. The deaired water was then allowed to flow upward through the specimen in order to increase the degree of saturation. A gradient of less than 5 was used during this step in order to minimize specimen disturbance. To insure that the gradient remained below 5, the gradient was established by raising the tank containing the deaired water to an appropriate height. No vacuum was used to draw the de-aired water through the specimen.

Once the desired volume of deaired water had flowed through the specimen, the drainage valves on the cell were again closed and the vacuum and deaired water lines removed. The drainage lines were then connected to the volume change device. At this time, the next phase of the software was initiated, the cell pressure was automatically raised to 50 kPa, and the axial loading rod was unlocked.

3.4.2.4 Back Pressure Saturation

Backpressure saturation of the specimen was controlled to a large extent by the computer software. Generally, the saturation process was performed under an effective stress of 50 kPa (7.3 psi.) with the operator controlling the level of the back pressure and the computer applying the cell pressure require to maintain a constant effective stress on the specimen. The backpressure was raised in small increments (less than 20 kPa) and the B value periodically checked until a value of 0.94 or greater was achieved, indicating that the specimen was essentially saturated.

3.4.2.5 Consolidation

Once an acceptable B value had been obtained, the specimens were isotropically consolidated to an effective stress of 100 kPa (14.7 psi). Specimens were allowed to consolidate for approximately one cycle of secondary compression. For the materials with low fines contents this time was approximately 2 minutes, while for the pure silt a consolidation time of 90 minutes was used.

Following consolidation, the changes in specimen volume and height that occurred during backpressure saturation and consolidation were recorded. These changes were applied to the respective initial values and the final specimen void ratios were calculated. These post-consolidation void ratios were used to calculate the post-consolidation relative densities reported elsewhere in this dissertation.

3.4.2.6 Cyclic Loading

Once consolidation was complete, the proper parameters were input into the computer, the drainage lines were closed, and the specimen was loaded cyclically. The parameters input into the computer include the peak to peak deviator stress corresponding to the desired cyclic stress ratio, the period of the loading function (1 or 2 seconds), the shape of the loading function (sinusoidal), the number of cycles of loading (limited to 200 cycles by the computer software), and the limiting single amplitude axial strain at which the test will be terminated (typically 6 percent axial strain).

Once the necessary information had been input, any necessary adjustments to the LVDT or steady pressure were made, and valve A was closed to prevent drainage and to allow measurement of the pore pressure in the specimen. The test was then started and the specimen cyclically loaded until failure.

If the specimen failed to achieve the limiting strain within the limit of 200 cycles, the valve between the drainage line and the volume change device (valve B) was closed to

prevent drainage from the specimen and the valve connecting the pore pressure and volume change transducers (valve A) was opened (this is required by the computer program to initiate the testing process). The cyclic loading test procedure was then repeated. Just before the initiation of loading, valve A was again closed and valve B opened. Loading of the specimen was then repeated. If the limiting strain was not reached in the following 200 cycles, this process was repeated until either the limiting strain or an effective stress of zero was achieved.

When testing of the specimen had been completed, the system was shut down and the pressure source turned off. Next, the triaxial cell was removed from the loading frame, the pressure line replaced with a venting plug, the cell drained, and the acrylic cell wall removed. The upper drainage line was then disconnected from the upper platen, the membrane detached from the upper platen and the specimen and membrane removed. If the specimen contained silty fines, the soil was the placed in a bowl and dried for reuse. If the specimen contained clayey fines it was first soaked in a dispersing agent (sodium metahexaphosphate), then the fine grained material was removed by washing the soil over a No. 200 sieve, and the sand re-used. This prevented the plasticity of the clayey fines being altered by drying in the oven and possibly altering the behavior of the soil.

3.4.2.7 Conversion of Data Files

In order to use the data in most DOS or windows based programs, it is necessary to convert the data file to an ASCII format. The program CONVASC performed this conversion. To execute the conversion, the command “convasc grdata\filename” where “filename” is the actual name of the data file is entered following the C:\Triax> prompt. This converts the original data file, which is denoted by a .dat suffix, to an space delineated ASCII file denoted by a .prn suffix.

3.4.2.8 Graphical Examination of Data

A visual examination of the test data can be performed using the utility program GRPLOT. To execute the program, the command "grplot grdata\filename" (where "filename" is the actual name of the data file) is entered following the C:\Triax> prompt. Numerous parameters such as total or effective stress, axial or volumetric strains, and pore water pressures can each be plotted against time. This is very useful for preliminary assessments of the number of cycles to initial liquefaction or to some strain level. Additionally, the program can plot several varieties of stress paths, including Cambridge and MIT types.

3.4.3 Calibration Of The Test Methodology

In order to ensure that the testing methods were producing reasonable cyclic strengths, a series of calibration tests were first run using the proposed methodology on a sand with know cyclic strength properties. The sand used was Monterey #0/30 which is very similar to the Monterey #0 sand used in the standard sand testing program conducted by Silver et al. (1976) in the 1970's.

A series of calibration tests were performed using Monterey #0/30 sand. The results of the standard sand testing program (Silver et al., 1976) are shown along with the results of the calibration testing program in Figure 3-13. It can be seen from the figure that the test methodology produces results that are in good agreement with the published data.

3.5 Correction Of Cyclic Stress Ratios

Following the completion of the laboratory portion of the study it was discovered that some of the deviator stresses applied by the test equipment during the cyclic loading portion of the test were incorrect, resulting in the application of cyclic stress ratios different than those specified by the operator. While the peak amplitudes of the stresses applied during each cycle were incorrect, the peak amplitude of stress applied remained

nearly constant from cycle to cycle. Additionally, the shape of the loading function remained a sinusoid as specified, and the loading rate was correct at either 0.5 or 1 Hertz.

Although most of these differences in the specified and actual cyclic stress ratios were small (less than 10 percent), a computer code was written to analyze the data files and determine the CSR which was actually applied to the specimen based upon the peak loadings applied during each cycle. All data presented in this report reflects the cyclic stress ratios actually applied to the specimen.

Table 3-1: Index properties for component soils

	Yatesville Sand	**Monterey Sand**	**Yatesville Silt**	**Kaolinite**	**Bentonite**
USCS Classification Symbol	SP	SP	ML	CH	CH
Maximum Grain Size (mm)	0.90	0.8	0.074	N.D.	N.D.
Median Grain Size, D50 (mm)	0.18	0.43	0.03	N.D.	N.D.
Minimum Grain Size (mm)	0.07	0.07	0.00	N.D.	N.D.
Coefficient Of Uniformity, Cu	2.4	1.5	4.4	N.D.	N.D.
Coefficient Of Curvature, Cc	0.77	1.04	0.93	N.D.	N.D.
Specific Gravity, Gs	2.72	2.65	2.77	2.65	2.65
Minimum Index Density (pcf)	86.1	90.8	63.5	N.D.	N.D.
Maximum Index Void Ratio, e max	0.972	0.821	1.723	N.D.	N.D.
Maximum Index Density (pcf)	102.5	101.4	100.4	N.D.	N.D.
Minimum Index Void Ratio , e min	0.653	0.631	0.727	N.D.	N.D.
Liquid Limit (%)	0	0	0	58	385
Plastic Limit (%)	0	0	0	27	41
Plasticity Index (%)	0	0	0	31	344

N.D. = Not Determined

Table 3-2: Index properties for mixtures of Yatesville sand with silt

Percent Silt (%)	4	7	12	17	26	37	50	75
USCS Classification Symbol	SP	SP-SM	SP-SM	SM	SM	SM	SM	ML
Median Grain Size, D_{50} (mm)	0.17	0.16	0.17	0.17	0.17	0.15	0.06	0.03
Coefficient Of Uniformity, Cu	2.6	2.6	3.5	4.5	6.6	8.7	13.3	7.2
Coefficient Of Curvature, Cc	0.78	0.75	0.91	1.10	1.10	0.62	0.80	0.87
Specific Gravity, Gs	2.72	2.72	2.73	2.73	2.73	2.74	2.75	2.76
Minimum Index Density (pcf)	88.8	90.7	92.3	94.1	98.6	101.4	84.3	74.3
Maximum Index Void Ratio, e max	0.911	0.872	0.840	0.802	0.728	0.684	1.031	1.314
Maximum Index Density (pcf)	105.4	111.0	111.9	113.2	116.2	122.1	120.6	108.2
Minimum Index Void Ratio , e min	0.609	0.528	0.517	0.502	0.483	0.397	0.423	0.591

Table 3-3: Index properties for mixtures of Monterey sand with silt

Percent Silt (%)	5	10	15	20	25	35	50	75
USCS Classification	SP	SP-SM	SP-SM	SM	SM	SM	SM	ML
Median Grain Size, D_{50} (mm)	0.43	0.43	0.43	0.43	0.40	0.36	0.07	0.05
Coefficient Of Uniformity, Cu	1.7	2.2	11.5	17.0	25.3	38.5	21.3	6.2
Coefficient Of Curvature, Cc	1.07	1.39	6.47	8.40	10.73	0.72	0.52	2.95
Specific Gravity, Gs	2.66	2.66	2.67	2.67	2.68	2.69	2.71	2.74
Minimum Index Density, Method B (pcf)	94.5	97.6	99.7	102.6	103.0	99.4	85.7	79.9
Maximum Index Void Ratio, e max	0.755	0.702	0.670	0.627	0.625	0.690	0.973	1.139
Maximum Index Density (pcf)	94.5	113.0	119.6	124.2	125.3	121.9	121.4	109.9
Minimum Index Void Ratio (Method B), e min	0.566	0.470	0.419	0.344	0.335	0.378	0.418	0.579

Table 3-4: Atterberg limit data for mixtures of Yatesville sand with silt and clay

Percent Fines	Percent Silt	Percent Kaolinite	Percent Bentonite	LL (%)	PL (%)	PI (%)	Activity
4	0	4	0	17	0	0	0.00
4	2	2	0	20	0	0	0.00
7	0	7	0	19	0	0	0.00
7	3.5	3.5	0	19	0	0	0.00
12	0	0	12	48	28	20	1.67
12	0	12	0	17	0	0	0.00
12	6	6	0	17	0	0	0.00
17	0	0	17	61	26	35	2.06
17	0	8.5	8.5	41	22	19	1.12
17	0	17	0	18	0	0	0.00
17	5.7	5.7	5.7	31	24	7	0.62
17	8.5	8.5	0	15	0	0	0.00
26	0	13	13	55	20	35	1.35
26	0	26	0	20	13	7	0.27
26	13	13	0	15	14	1	0.08
37	0	37	0	21	13	8	0.22

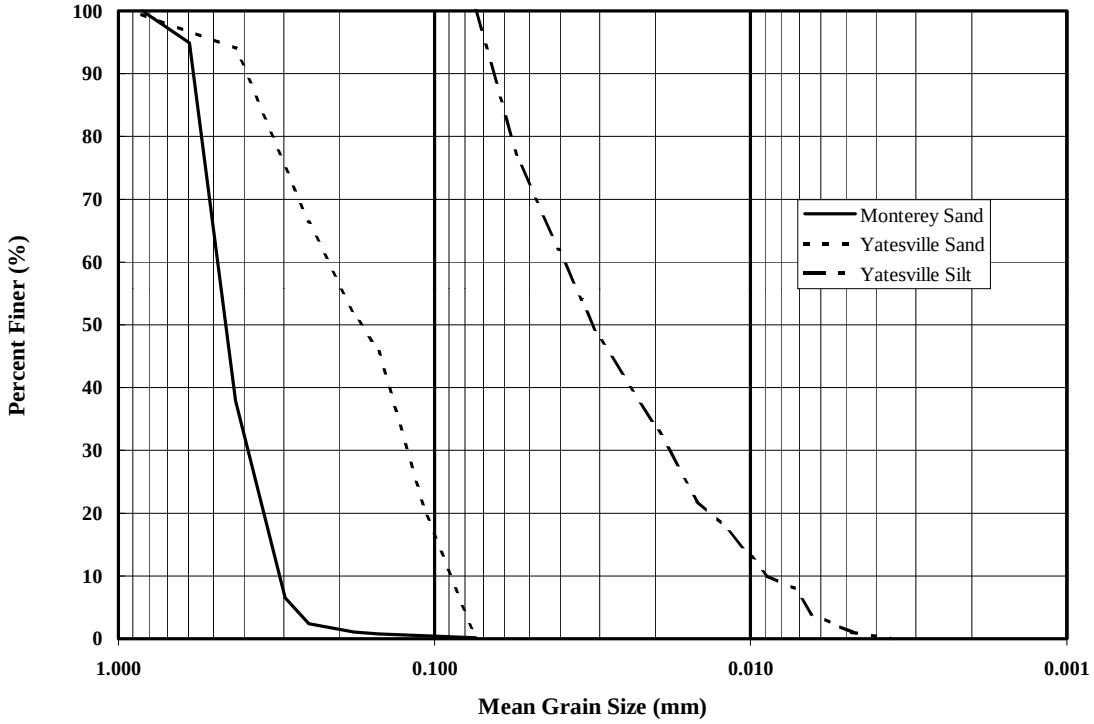

Figure 3-1: Grain size distributions for component soils

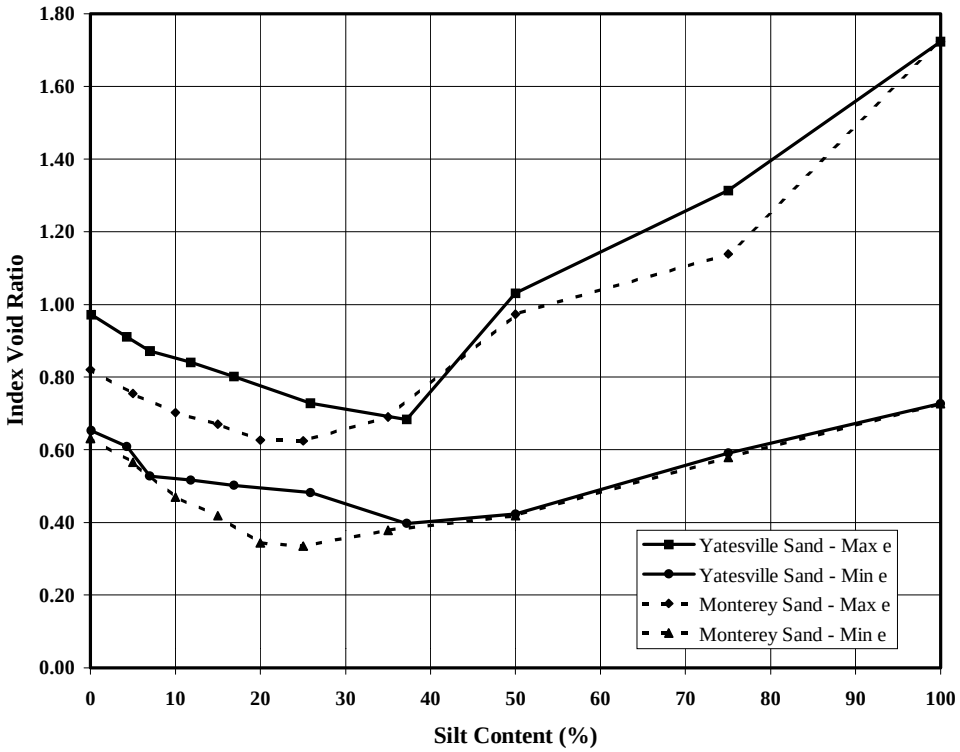

Figure 3-2: Variation in index void ratios with silt content

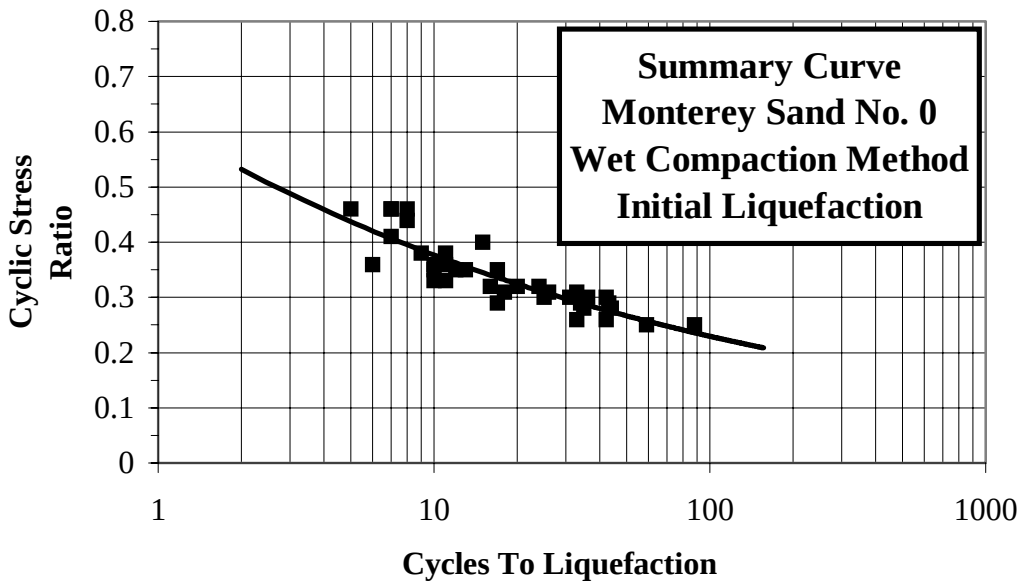

Figure 3-3: A cyclic resistance curve (After Silver et al. (1976))

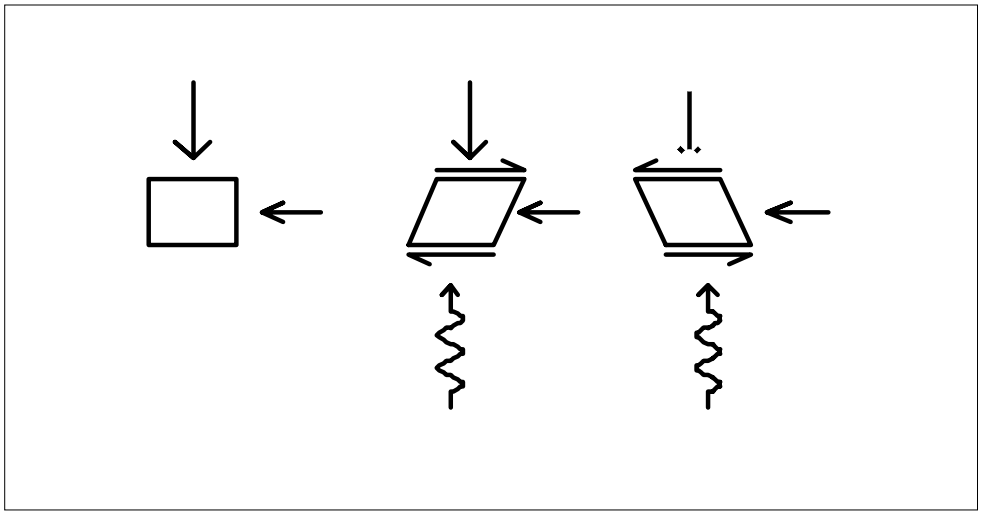

Figure 3-4: Idealized stresses induced by seismic shaking on a soil element under level ground (After Seed and Lee (1966))

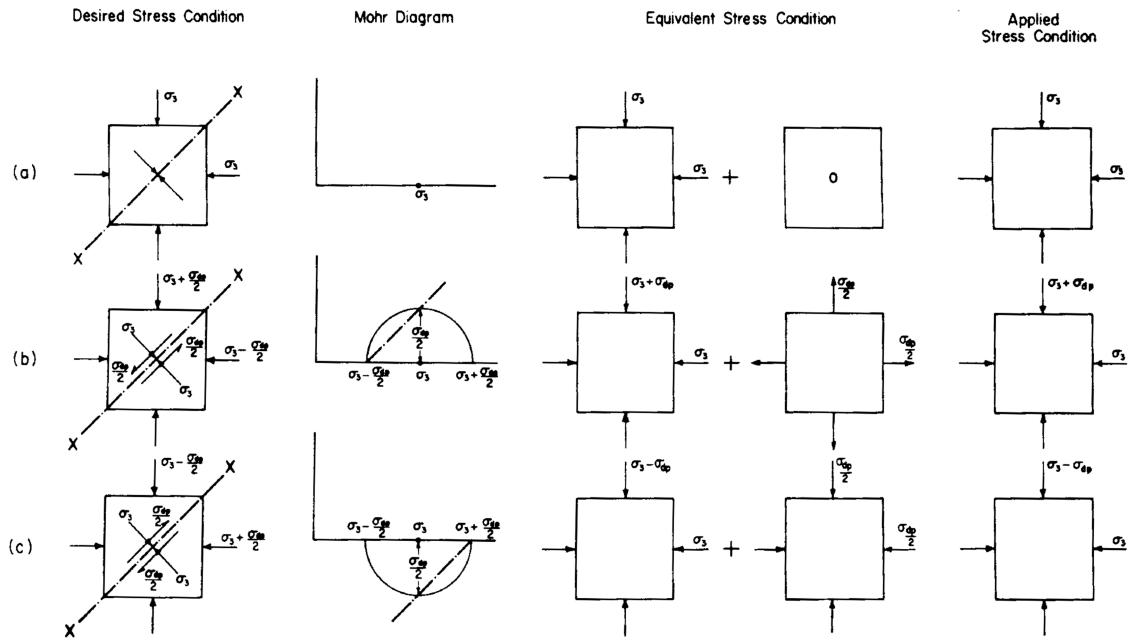

Figure 3-5: Stress conditions during cyclic triaxial test (From Seed and Lee (1966))

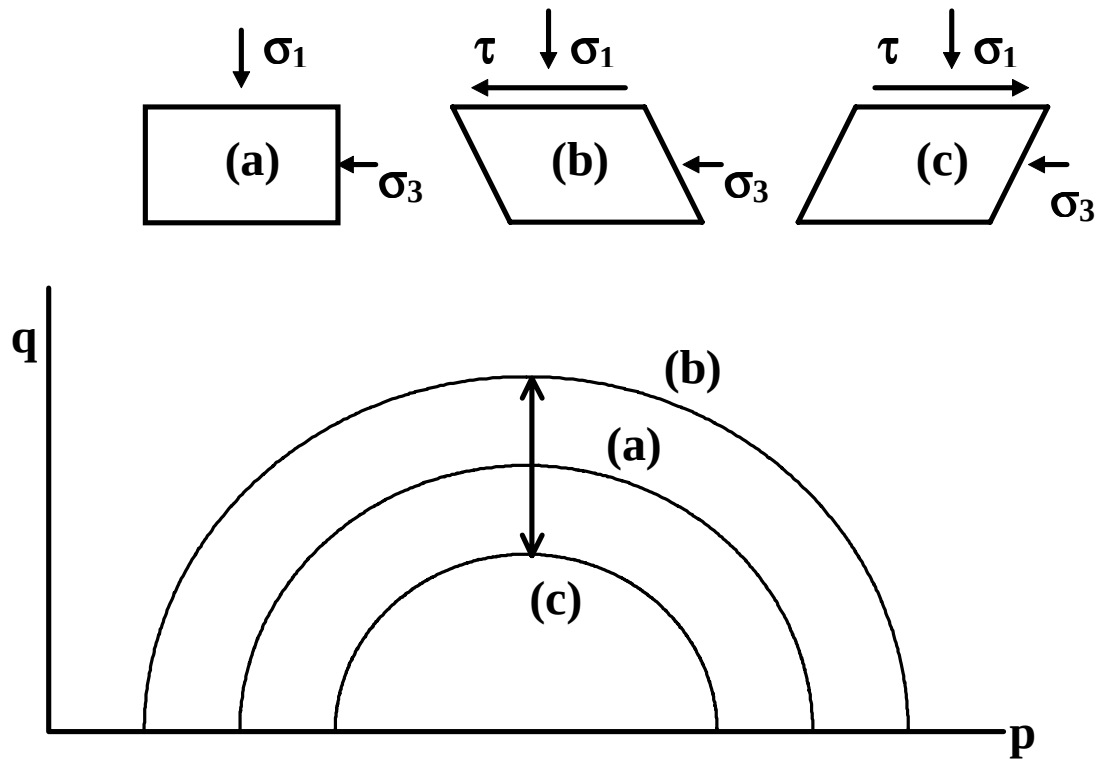

Figure 3-6: Applied stresses and stress path for a soil element in the field

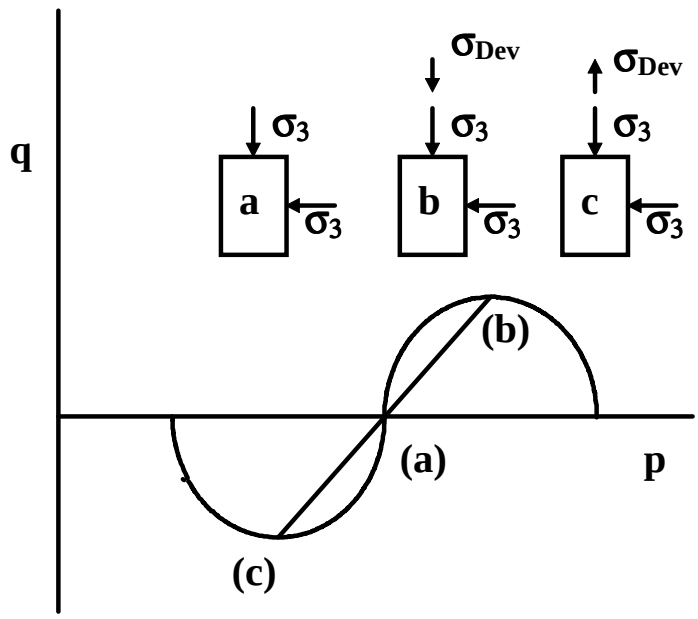

Figure 3-7: Applied stresses and stress path for a soil element in a cyclic triaxial test

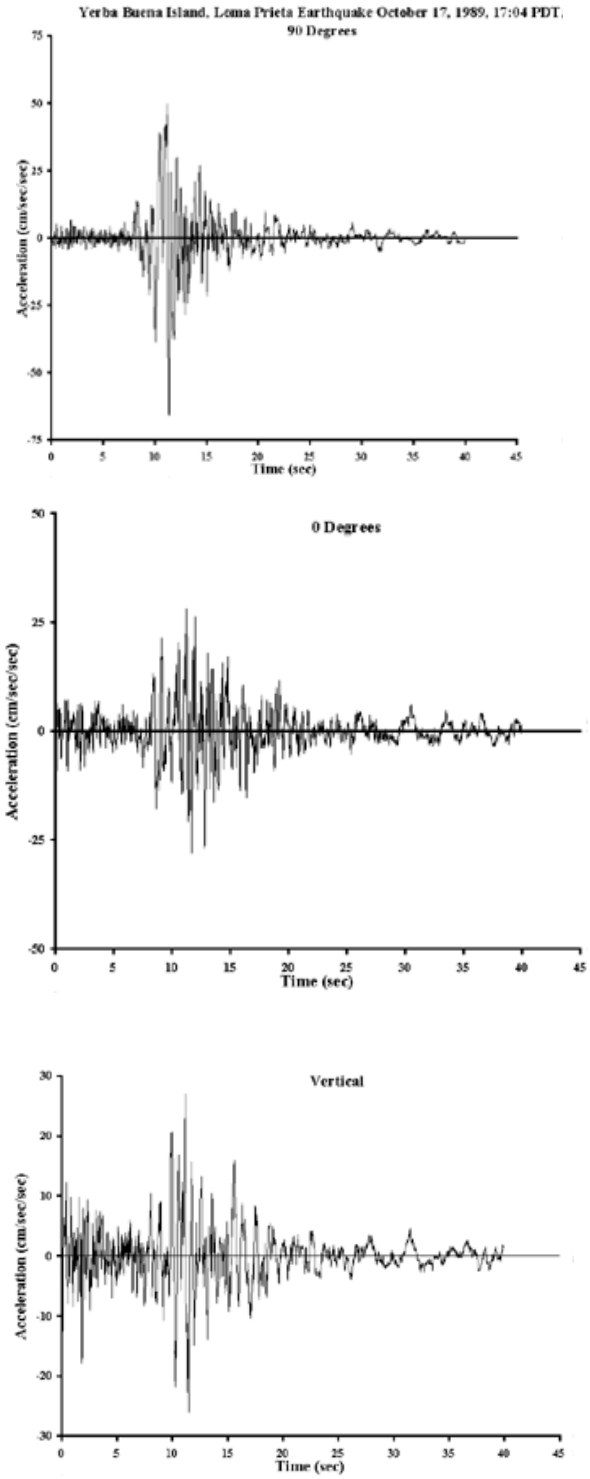

Figure 3-8: Multiple components of earthquake motions

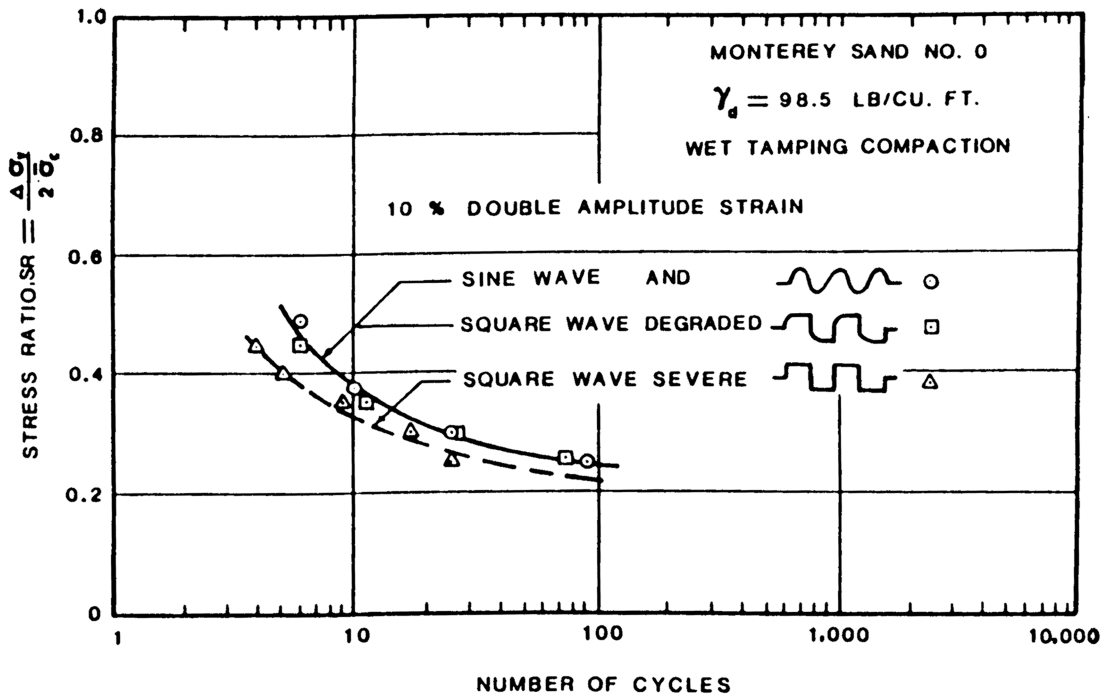

Figure 3-9: The effect of wave shape on liquefaction resistance (From Silver et al. (1976))

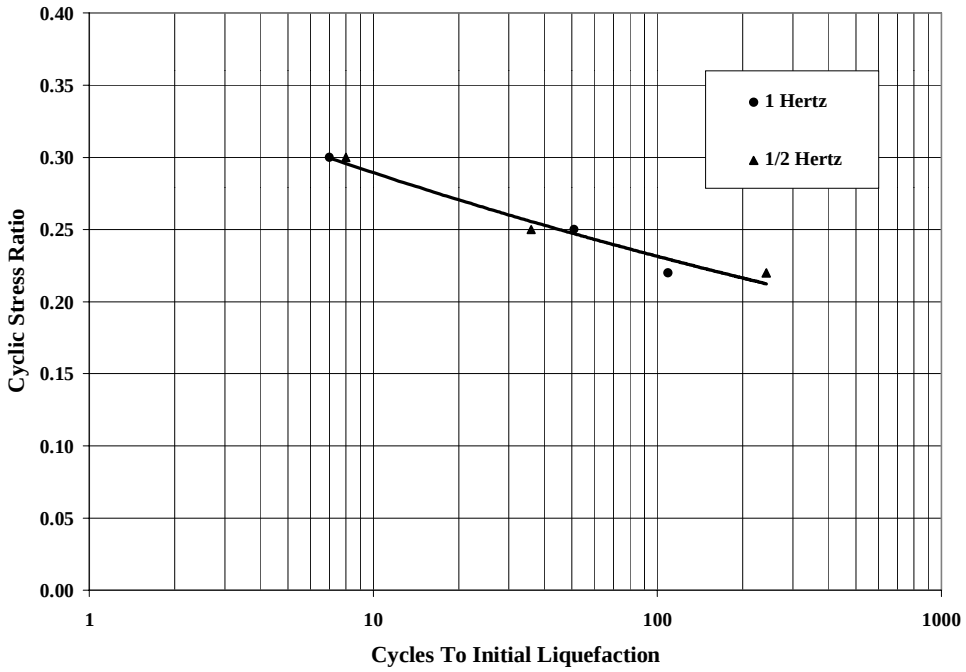

Figure 3-10: Effects of different frequencies of loading on Monterey sand

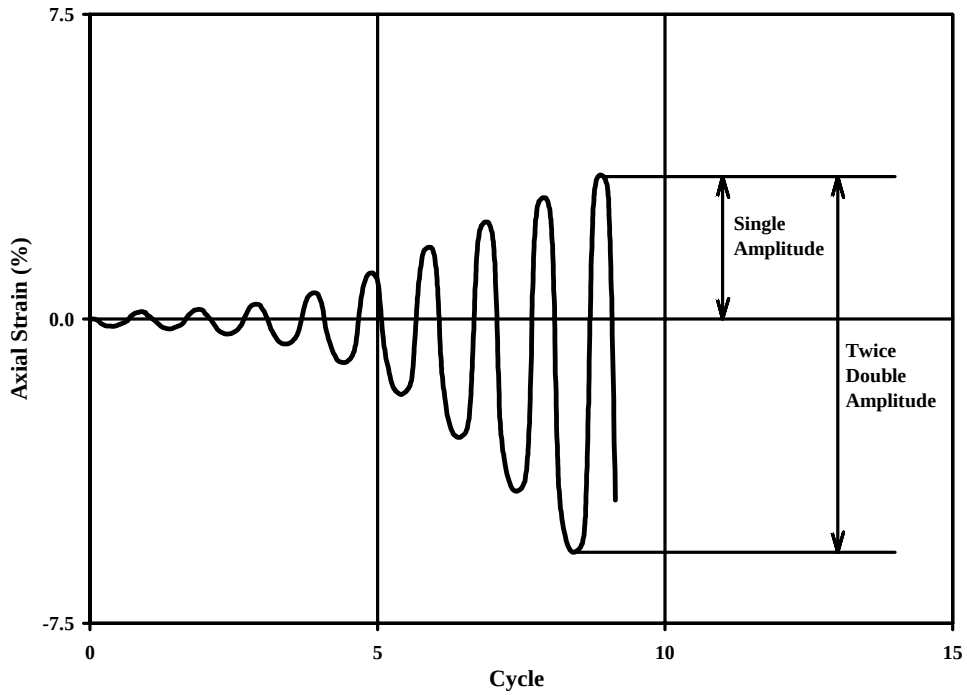

Figure 3-11: Definitions of single and double amplitude shear strain

Cyclic Triaxial Specimen Data Sheet

Date: 1/27/98

Soil Type: Yatesville Sand with 0% Silt
Goal e: 0.76
Goal Gammad: 15.16 kn/m^3
Goal Dr: 67.0 %

Proposed CSR: 0.33
Actual e: 0.760
Actual Gammad: 15.16 kn/m^3
Actual Dr: 67.1 %

Sample Prep:

Max Void Ratio: 0.972
Min Void Ratio: 0.656
Gs: 2.72

Layer:	Wt Soil (gm)	Wt Water (gm)
1	134.3	19.0
2	134.6	19.0
3	134.8	18.9
4	135.1	18.9
5	135.3	18.8
6	135.6	18.8
7	135.8	18.7

Water Content = 14.0 %
Initial Saturation = 48.7 %

Sample Data

Sample Height: 6.06 inches
Sample Volume: 611.48 cc
Wt Dry Soil: 945.5 gm
Gammad: 1.5462 gm/cc

Membrane Thickness = 0.025 inches
Sample Diameter: 2.825 inches
2.825 inches
2.825 inches
Diameter - Membrane: 2.800 inches

Saturate with CO2 And Deaired Water

Binitial= 0.82 Initial LVDT = 0.000 Bfinal= 0.97
CPfinal= 450 Kpa BPfinal= 400 Kpa

Consolidation Stress: 100 Kpa **Consolidation Time:** 2 minutes

LVDT initial = -0.084 volts
LVDT final = -0.104 volts
Final Sample Volume = 37.09 cu in
Final Sample Height = 6.048 inches
Final Sample Area = 6.1332 sq inches

Vol Trans initial = 1.426 volts
Vol Trans final = 1.613 volts
Change in Sample Volume = -3.61 cc
Post Consol e = 0.749
Post Consol Dr = 70.6

Shear at +/-: 66 Kpa CSR = 0.33

CPfinal= 500 Kpa BPfinal= 400 Kpa

Data File: y0e76c33

Failed in 12 Cycles

Comments:

Figure 3-12: Typical cyclic triaxial data sheet

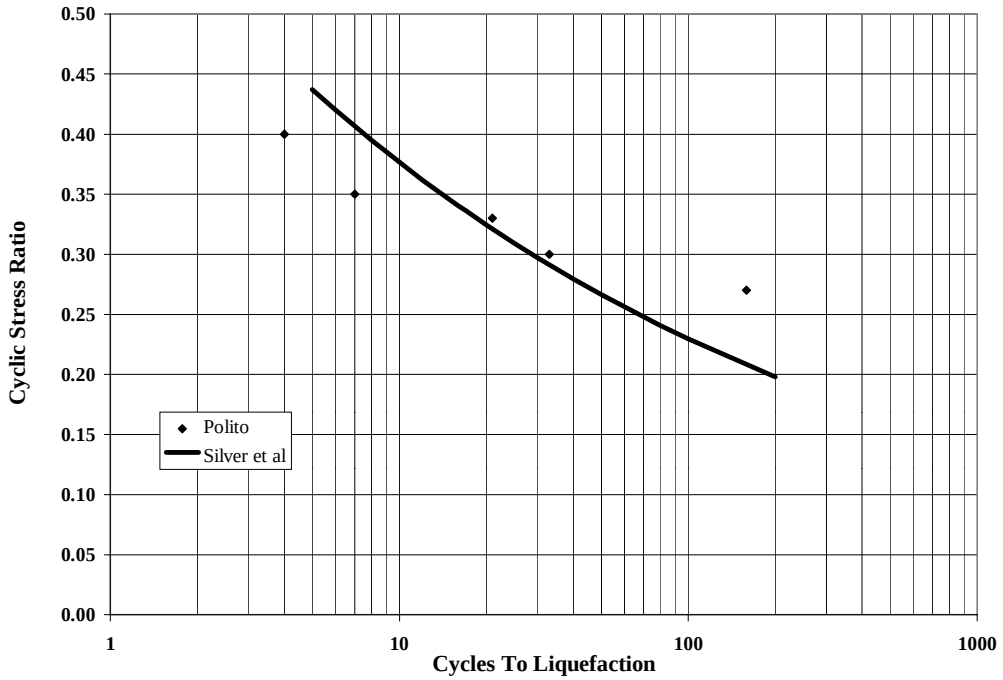

Figure 3-13: Calibration of test methodology

Chapter 4: The Effects of Non-Plastic Fines

During the past 40 years the liquefaction of clean sands under seismic loads has been studied and a sound understanding of its mechanisms and the parameters which affect it has been developed. Unfortunately, the understanding of the liquefaction of sands containing fine-grained material is less complete. A review of the literature shows that there is no clear consensus as to what effect an increase in non-plastic silt content has upon the liquefaction resistance of a sand.

This chapter is concerned with the effects of non-plastic, i.e. silty, fines content on the liquefaction resistance of sands. The current study was undertaken in an attempt to clarify the discrepancies found in the literature, and to find a single parameter which controls the liquefaction resistance of silty sands and sandy silts. In order to achieve this goal, nearly 300 cyclic triaxial tests were run using two base sands with non-plastic silt contents ranging from 0 to 100 percent. Tests were performed and evaluated using gross void ratio, gross relative density, soil specific relative density, and sand skeleton void ratio as measures of density.

Both clean sands and sands containing silt have been shown to be liquefiable in the field (Mogami and Kubo, 1953; Seed and Lee, 1966; Youd and Bennett, 1983) and in the laboratory (Lee and Seed, 1967a; Casagrande, 1975; Koester, 1994). Non-plastic silts, most notably mine tailings, have also been found to be susceptible to liquefaction (Dobry and Alvarez, 1967; Okusa et al., 1980; Garga and McKay; 1984). Numerous laboratory studies have been performed, and have produced what appear to be conflicting results. These studies report that increasing silt content in a sand will either increase the liquefaction resistance of the sand (Chang et al., 1982; Dezfulian, 1982), decrease the liquefaction resistance of the sand (Shen et al., 1977; Tronsco and Verdugo, 1985; Finn et al., 1994; and Vaid, 1994), or decrease the liquefaction resistance until some limiting

silt content is reached, and then increase its resistance (Law and Ling, 1992; Koester, 1994). Additionally, several studies (Shen et al., 1977; Tronsco and Verdugo, 1985; Kuerbis et al., 1988; and Vaid, 1994) have shown that the liquefaction resistance of a silty sand is more closely related to its sand skeleton void ratio than to its silt content.

The results of the current study will be presented in terms of the various measures of density examined. First, the results of the tests evaluated in terms of gross void ratio and gross relative density will be presented, followed by the results of the tests evaluated in terms of sand skeleton void ratio. Next, the results of the tests evaluated in terms of soil specific relative density will be presented. Following the presentation of the test results, a discussion on flow liquefaction and cyclic mobility will be presented. Lastly, the results of the current study will be compared to the data presented in the literature and used to explain and reconcile the seemingly differing trends.

4.1 Findings of the Current Study

Several measures of density were used to examine the results of the testing program. They include gross void ratio, gross relative density, sand skeleton void ratio, and soil specific relative density.

4.1.1 Tests Evaluated In Terms Of Gross Void Ratio And Gross Relative Density

The cyclic resistances measured were first examined in terms of their gross void ratios. The gross void ratio of a specimen is simply the ratio of the volume of the voids in the specimen to the volume of the solids, and is a function of the dry density of the specimen and specific gravity of the soil. Except for the small effect that the amount of silt present has on the specific gravity of the soil mixture, the gross void ratio of a specimen is dependent solely upon the weight of soil used and the volume of the specimen, and is independent of silt content.

The effects of holding the gross void ratio of the specimens constant while altering their silt content was examined. For the Monterey sand, a void ratio of 0.68, which corresponds to a relative density of 74 percent in the clean sand, was used. For the Yatesville sand, a void ratio of 0.76, which corresponds to a relative density of 68 percent in the clean sand, was used. These void ratios were chosen because they resulted in soil specific relative densities that allowed specimens to be formed over the entire range of silt contents investigated.

The results of the tests on Monterey and Yatesville sands prepared to a constant gross void ratio are presented in Figures 4-1 and 4-2, respectively. The cyclic resistance can be seen to decrease with increasing silt content until a minimum is reached between silt contents of 35 and 50 percent at which time the cyclic resistance begins increasing as the silt content continues to increase.

The cyclic resistance for each sand and silt combination tested during this study have been plotted as a function of gross void ratio in Figures 4-3 and 4-4. These plots represent the Monterey sand and Yatesville sand mixtures, respectively.

As can be seen in Figure 4-3, for the Monterey sand and silt mixtures there is no clear correlation between gross void ratio and cyclic resistance. For example, at a gross void ratio of 0.68 (i.e. the tests plotted in Figure 4-1), the cyclic resistance varies from a low of 0.14 for a mixture of 50 percent sand and 50 percent silt to a high of 0.40 for the both clean sand and the pure silt. For a given silt content, however, there is a decrease in cyclic resistance as the gross void ratio increases and the soil specific relative density decreases.

Figure 4-4 shows that for the Yatesville sand and silt mixtures there is, again, little correlation between gross void ratio and cyclic resistance. Similar to the Monterey sand

specimens, within each silt content there is a decrease in cyclic resistance with an increase in gross void ratio and corresponding decreases in soil specific relative density.

As might be expected, the behavior of these soils are the same when examined in terms of gross relative density as they are when examined in terms of gross void ratio. This is because the gross relative density for a specimen is based upon the gross void ratio of the specimen and the maximum and minimum index void ratios for the clean base sand. Because the maximum and minimum index void ratios used do not vary with silt content, the only term in the gross relative density equation which changes from specimen to specimen is the gross void ratio. As a result, the changes in gross relative density simply mirror the changes in gross void ratio.

In summary, there does not appear to be a unique relationship between silt content, gross void ratio, and cyclic resistance for the mixtures of sand and silt. Instead, for specimens prepared to a constant gross void ratio, cyclic resistance first decreases and then increases after some limiting silt content is reached. As density is varied within a given silt content, there is a decrease in cyclic resistance with an increase in gross void ratio and a corresponding decrease in soil specific relative density. This decrease in cyclic resistance with an increase in gross void ratio was observed in both the Monterey and Yatesville sand mixtures and is similar to the behavior of clean sands. Similar trends were found when the data was evaluated in terms of gross relative density due to its direct dependence on gross void ratio.

4.1.2 Tests Evaluated In Terms Of Sand Skeleton Void Ratio

The cyclic resistances measured were next evaluated in terms of their sand skeleton void ratios. Sand skeleton void ratio is the void ratio that would exist in the soil if all of the silt and clay particles were removed, leaving only the sand grains to form the soil skeleton. Several investigators (Shen et al., 1977; Tronsco and Verdugo, 1985; Kuerbis et al., 1988;

and Vaid, 1994) have suggested that as long as there is sufficient room for the silt grains within the voids produced by the sand, the controlling factor in the cyclic resistance of a sand-silt mixture is the sand skeleton void ratio.

The maximum amount of silt that can be contained in the voids created by the sand without deforming the sand skeleton is referred to as the limiting silt content. The limiting silt content is the transition point below which the soil structure is primarily one of silt contained in a sand matrix and above which it is predominately sand grains suspended in a silt matrix with little sand grain to sand grain contact.

The proposed idea that cyclic resistance is solely a function of sand skeleton void ratio implies that, as long as the voids created by the sand skeleton are large enough to accommodate all of the silt particles present, the amount of silt, and thus the gross void ratio, has no effect on the cyclic resistance of the soil.

The limiting silt content was calculated for both the Monterey and Yatesville sands using the method presented by Mitchell (1993). It was assumed in this analysis that the silt in the voids was at its maximum index void ratio. This assumption provides an lower bound value of limiting silt content. For the Monterey sand, the limit is approximately 32 percent silt by weight and for the Yatesville sand it is approximately 36 percent. These silt contents were also found to correspond to the transition points at which the cyclic resistance of the soils in this study became independent of the sand skeleton void ratio.

Below the limiting silt content, Monterey sand was found to maintain a nearly constant cyclic resistance with increasing silt contents for specimens prepared to a constant sand skeleton void ratio. This is shown in Figure 4-5. Conversely, below the limiting silt content, Yatesville sand was found to increase in cyclic resistance with increasing silt contents for specimens prepared to constant sand skeleton void ratios. This is shown in Figure 4-6. The cyclic resistances for each sand and silt combination tested during this

study have been replotted to show cyclic resistance as a function of sand skeleton void ratio. These are shown for the Monterey and Yatesville base sands in Figures 4-7 and 4-8, respectively.

For the Monterey sand and silt mixtures there is a strong correlation between sand skeleton void ratio and cyclic resistance for silt contents of 35 percent and less. Cyclic resistance decreases linearly with increasing sand skeleton void ratio. For silt contents of 50 percent and above, (that is, well above the limiting silt content) the cyclic resistance of the soil is nearly constant, varying little with either increasing sand skeleton void ratio or increasing silt content.

The Yatesville sand and silt mixtures exhibit a similar relationship between sand skeleton void ratio and cyclic resistance to that found for the Monterey sand specimens although there is considerably more scatter in the Yatesville sand data. Cyclic resistance again decreases with increasing sand skeleton void ratio for silt contents 37 percent and less. For silt contents of 50 percent and above, (that is, well above the limiting silt content), the cyclic resistance of the soil is nearly constant, varying little with either increasing sand skeleton void ratio or increasing silt content.

In summary, for soils with silt contents below the limiting silt content there is a relationship between sand skeleton void ratio and cyclic resistance, with cyclic resistance increasing with decreasing sand skeleton void ratio. Above the limiting silt content, the cyclic resistance is relatively constant and is independent of both the silt content and the sand skeleton void ratio.

4.1.3 Tests Evaluated In Terms Of Constant Soil Specific Relative Density

Finally, the data were evaluated in terms of their soil specific relative densities. The soil specific relative density is the relative density of the specimen based upon its gross void ratio and the maximum and minimum index void ratios for that particular mixture of sand and silt. When the cyclic resistances are plotted in terms of the soil specific relative density, clear patterns emerge. For specimens with silt contents below the limiting silt, there is a linear trend between increasing soil specific relative density and increasing cyclic resistance. These plots are presented for the Monterey and Yatesville base sands in Figures 4-9 and 4-10, respectively.

For the soils with silt contents greater than the limiting silt content, there is a second behavioral trend. For these soils the cyclic resistance is controlled by the silt fraction of the soil and is lower, at the same soil specific relative density, than the cyclic resistance of soils below the limiting silt content. Figure 4-11 plots the cyclic resistance of these soils against the silt fraction void ratio of the specimen. The cyclic resistance can be seen to vary with the silt fraction void ratio. Because cyclic resistance does not vary as a function of the percentages of sand and silt in the mixture, it is independent of the amount of sand, if any, which is present in the specimen. This indicates that as there is little sand grain to sand grain contact in these specimens, with the sand present making little or no contribution to the specimen's ability to resist liquefaction.

The change in cyclic resistance which occurs when the soil changes from a sand controlled matrix to a silt controlled matrix can be clearly seen in Figure 4-12 which presents a plot of cyclic resistance versus silt content for specimens of Yatesville sand and silt adjusted to 25 percent soil specific relative density. While a transition zone appears to exist between silt contents of approximately 35 and 55 percent, the decrease in

cyclic resistance that occurs when the silt content reaches a level greater than the limiting silt content can be clearly seen in this figure.

In summary, for soils with silt contents below the limiting silt content there is a nearly linear relationship between soil specific relative density and cyclic resistance. Below the limiting silt content, the cyclic resistance of the soil is independent of silt content, increasing with increasing soil specific relative density. Above the limiting silt level, the cyclic resistance varies with the silt fraction void ratio, increasing with decreasing silt fraction void ratio, and is independent of the amount of sand present.

4.2 Flow Liquefaction and Cyclic Mobility

When a soil specimen is subjected to cyclic loading and reaches a state of zero effective stress, it is often said to have liquefied. However, its post-liquefaction behavior can be classified as one of two conditions, flow liquefaction or cyclic mobility. Correspondingly, soil deposits subjected to earthquake loadings are also capable of these two behaviors, flow liquefaction and cyclic mobility (Casagrande, 1975). These behaviors are best considered in terms of steady-state soil mechanics (Casagrande, 1975; Castro and Poulos, 1977). While a detailed essay on steady-state soil mechanics is beyond the scope of this document, a brief summary is provided below.

When a soil is sheared to a large strain, often upwards of 25 percent, it approaches a constant condition. This condition is referred to as the steady-state condition and is characterized by displacement at a constant effective stress, a constant velocity, a constant volume, and an absence of particle breakage and rearrangement. In order for this to occur, the soil must be subjected to some driving shear stress such as a slope or a building. In the field, a soil at steady-state will flow until the driving shear stresses become less than the steady-state shear strength.

The steady-state strength of a soil is controlled by a combination of its critical void ratio and critical effective confining stress. A plot of these critical void ratios and confining stresses is referred to as the steady-state line. A sample steady-state line is shown in Figure 4-13. Soils that plot above the steady-state line, such as Point A, are contractive when sheared, and are susceptible to flow liquefaction. Soils that plot below the line, such as Point B, are dilative when sheared, and are susceptible to cyclic mobility.

4.2.1 Flow Liquefaction

Flow liquefaction occurs in soils that are at void ratios larger than the critical void ratio corresponding to the effective confining stress acting on the soil. Point A in Figure 4-13 represents such a condition. When sheared undrained, the soil will develop positive pore pressures which reduce the effective stress on the specimen, which in turn decrease the strength of the specimen. This lowered strength leads to increased strain that leads to increased pore pressures, which results in lower strength. This self-perpetuating loss of strength continues until a steady-state condition is achieved. Very large strains or displacements typically accompany flow liquefaction.

A typical strain versus cycles of loading curve from a cyclic triaxial test performed on a specimen exhibiting flow liquefaction is presented in Figure 4-14. Its behavior in characterized by the very small strains throughout the loading until just before the on-set of initial liquefaction at which time a large, sudden, compressive strain occurs. The magnitude of the strain achieved is limited by the mechanisms of the test, not by the behavior of the soil. If allowed, it would continue to deform indefinitely. If one were to examine the specimen following the test, it would be found to bear no resemblance its shape prior to loading. It would be found to be a soft mass of soil with very little strength, with only the rubber membrane preventing it from forming a shallow pile in the floor of the cell.

4.2.2 Cyclic Mobility

Cyclic mobility occurs in soils which are at void ratios which are smaller than the critical void ratio corresponding to the effective confining stress acting on the soil. Point B in Figure 4-13 represents such a condition. When sheared undrained, the soil will initially attempt to compress, thus developing positive pore pressures which reduce the effective stress on the specimen, but as it continues to strain, the soil becomes dilative, the pore pressures decrease, the effective stress increases, and the strength of the soil increases. This dilative behavior and resulting strength gain prevents large displacements from occurring.

A typical strain versus cycles of loading curve from a cyclic triaxial test performed on a specimen exhibiting cyclic mobility is presented in Figure 4-15. Its behavior is characterized by the nearly uniform development of bi-axial strains throughout the course of loading and a return to essentially zero strain at the end of loading. Because the specimen is weaker in “tension” (when minor principal effective stress acts in the vertical direction) than in compression, there is a tendency for slightly greater strains to develop when the minor principal effective stress acts in the vertical direction. If one were to examine the specimen following the test, it would be found to be essentially the same as it was prior to loading, that is, a rigid, uniform cylinder.

Due to limited test data it is difficult to determine at exactly what void ratio or relative density the behavior of the soils in this study changed from flow liquefaction to cyclic mobility. A possible future course of study would be to determine if the density required to change the behavior of the soil from flow liquefaction to cyclic mobility is a function of silt content.

4.3 Evaluation And Reconciliation Of The Results Of Published Studies

Numerous laboratory studies which have been reported in the literature have produced what appear to be conflicting results. These studies report that increasing the silt content in a sand will either increase the liquefaction resistance of the sand (Chang et al., 1982; Dezfulian, 1982), decrease the liquefaction resistance of the sand (Shen et al., 1977; Tronsco and Verdugo, 1985; Finn et al., 1994; and Vaid, 1994), or decrease the liquefaction resistance until some limiting silt content is reached, and then increase its resistance (Law and Ling, 1992; Koester, 1994). Additionally, several studies (Shen et al., 1977; Tronsco and Verdugo, 1985; Kuerbis et al., 1988; and Vaid, 1994) have shown that the liquefaction resistance of a silty sand is more closely related to its sand skeleton void ratio than to its silt content.

It was found that the majority of these seemingly contradictory trends reported in the literature could be explained in light of the findings of this study. Following the introduction of the concept of normalized cyclic resistance, the changes in soil specific relative density which occur for specimens prepared to a constant gross void as silt content increases will be discussed. Next, the results of the study will be used to explain the behaviors identified in the literature. First, the trend of decreasing cyclic resistance with increasing silt content will be explained, followed by the trend of decreasing and then increasing cyclic resistance with increasing silt content. Next, the findings of constant or increasing cyclic resistance with constant sand skeleton void ratio will be examined. Lastly, the trend of increasing cyclic resistance with increasing silt content will be discussed.

4.3.1 Normalized Cyclic Resistance

The majority of the published laboratory studies into the effects of silt content on liquefaction resistance have been performed using cyclic triaxial tests. Unfortunately,

several factors make comparisons of cyclic triaxial data from different investigators difficult. The first of these difficulties is that the results reported in these studies are from tests performed on a variety of sands with differing silt contents, densities, confining stresses, and cyclic loading levels. Secondly, different investigators have used different criteria to define liquefaction. Liquefaction is often defined as occurring when the pore water pressure in the specimen first equals the total confining stress and the effective stress becomes equal to zero (i.e. initial liquefaction). Liquefaction, however, has also been defined as occurring when some level of single or double amplitude axial strain, such as 2.5 or 5 percent, is reached in the specimen. Lastly, cyclic resistance, which is a measure of a soil's ability to resist liquefaction, is defined as the cyclic stress ratio required to achieve the defined liquefaction condition in a given number of cycles. Different investigators have chosen different numbers of cycles, often 10, 15, or 30, in defining their cyclic resistance.

The differences in soil type, testing conditions, and definitions of liquefaction and cyclic resistance require that the data be normalized in some manner before a comparison of the data from different studies can be made. In order to standardize both the published data and the data from the current study, and to evaluate them on a consistent basis, the concept of normalized cyclic resistance was created. To obtain the normalized cyclic resistance of a soil at a given silt content, the cyclic resistance of that soil is divided by the cyclic resistance of the clean base sand measured under the same conditions of density, effective confining stress, and failure criteria. In this manner, the variation of cyclic resistance with silt content within any study can be compared to the results of other studies. In this report, when the results of multiple authors are plotted on a single graph, the normalized cyclic resistance approach will be used.

4.3.2 Changes In Soil Specific Relative Density With Increasing Silt Content

The maximum and minimum index densities of a sand and silt mixture, which are an indication of the range of densities it can achieve, vary with silt content. When the silt content of a sand is increased, the maximum and minimum void ratios initially decrease as the soil becomes more well-graded. When the soil reaches the limiting silt content, the maximum and minimum void ratios reach their lowest values. As the silt content of the soil continues to increase, the maximum and minimum void ratios increase as the grain size distribution of the soil becomes more uniform.

By holding the gross void ratio of the specimens constant as the silt content increases, the soil specific relative density of the specimens first decreases and then increases as a result of the corresponding changes in the maximum and minimum void ratios. This behavior is shown in Figure 4-16, which plots the minimum and maximum void ratios for the Yatesville sand and silt mixtures and the soil specific relative density for specimens prepared to a gross preconsolidation void ratio of 0.76.

Because a specimen's ability to resist liquefaction is directly related to its soil specific relative density, a decrease in soil specific relative density produces a decrease in cyclic resistance and an increase in soil specific relative density leads to an increase in cyclic resistance. This pattern is shown for the Monterey and Yatesville base sands in Figures 4-17 and 4-18, which plot cyclic resistance and soil specific relative density against silt content for the tests performed at constant gross void ratio.

It should be noted that the lowest cyclic resistance does not correspond precisely with the limiting silt content or the lowest soil specific relative density. This occurs because, as the silt content of the soil is increased, the soil transforms from a sand-controlled matrix to a silt-controlled matrix. However, in the range of silt contents immediately above the

limiting silt content, the sand grains are still closely situated and exert an influence upon one another. Due to the inherent difference in cyclic resistance between the sand and the silt, as silt content increases, cyclic resistance continues to decrease, although the soil specific relative density is actually increasing.

This inherent difference in cyclic resistance between the sand and silt can be seen in Figure 4-16, which shows the curves of cyclic stress ratio versus the number of cycles to initial liquefaction for both clean Yatesville sand and pure Yatesville silt corrected to 50 percent soil specific relative density. Despite having the same soil specific relative density, the silt can be seen to have a much lower cyclic resistance. For cyclic resistance define as initial liquefaction in 10 cycles, the cyclic resistances of the Yatesville sand and the silt shown in Figure 4-19 are 0.24 and 0.07, respectively.

4.3.3 Decreasing Cyclic Resistance With Increasing Silt Content

Numerous authors have reported a decrease in cyclic resistance with increasing silt content. Shen et al. (1977), Tronsco and Verdugo (1985), and Vaid (1994) have all reported this trend for specimens prepared either to a constant gross void ratio or a constant dry density. The decreases in cyclic resistance were marked, decreasing as much as 60 percent from their clean sand values for an increase in silt content of 30 percent (Tronsco and Verdugo, 1985).

The decrease of cyclic resistance with increasing silt content can be explained in terms of the soil specific relative density. The specimens in these studies were all prepared to constant gross void ratios or densities, and were at silt contents that were likely below the limiting silt content. Because the specimens were prepared to constant gross void ratios, their soil specific relative densities decreased with increasing silt contents, leading to a decrease in cyclic resistance. Because the specimens were only tested at silt contents

below their limiting silt contents, the increase in cyclic resistance which would have occurred at the higher silt contents were not detected.

Figure 4-20 presents the normalized cyclic resistances for both the Monterey and Yatesville sands prepared to constant gross voids ratio at silt contents below their limiting silt contents and for the studies which reported a decrease in cyclic resistance with increasing silt content. These curves all show similar trends, confirming that this behavior can be explained in terms of the soil specific relative density.

4.3.4 Decreasing And Then Increasing Cyclic Resistance With Increasing Silt Content

Rather than a simple decrease in cyclic resistance with increasing fines contents, several investigators have reported a decrease and then an increase in cyclic resistance with increasing silt content. Koester (1994) and Law and Ling (1992) found that, for specimens prepared to a constant gross void ratio, as silt content increased the cyclic resistance of a soil first decreased until some limiting silt content was reached, at which point the cyclic resistance began increasing as silt content continued to increase. Koester (1994) reported a decrease in cyclic resistance to less than one-quarter of the clean sand cyclic resistance at a silt content of 20 percent, then an increase in cyclic resistance to 32 percent of the clean sand value at a silt content of 60 percent. Unlike Chang et al. (1982), and Dezfulian (1982), neither Koester (1994) nor Law and Ling (1992) reported increases in cyclic resistance to levels greater than those of the clean sand.

The decrease and then increase of cyclic resistance with increasing silt content can also be explained in terms of the soil specific relative density. The specimens tested in these studies were all prepared to constant gross void ratios, again creating soil specific relative densities that vary with varying silt content. As the silt content of the specimens increases, their soil specific relative density first decreases and then increases as the

maximum and minimum index void ratios vary. This variation in soil specific relative density creates the reported variations in cyclic resistance.

Figure 4-21 presents the normalized cyclic resistances for both the Monterey and Yatesville sand specimens prepared to constant gross void ratios and for the studies which reported a decrease followed by an increase in cyclic resistance with increasing silt content. These curves all show similar trends, confirming that this behavior can be explained in terms of the soil specific relative density.

For specimens prepared to an approximately constant soil specific relative density, Singh (1994) found that cyclic resistance underwent only small changes with increasing silt content. Although a small decrease in cyclic resistance followed by a slight increase in cyclic resistance with increasing silt content was measured, the changes in cyclic resistance are much smaller than those occurring in the studies by Koester (1994) and Law and Ling (1992). This difference in the magnitude of the changes in cyclic resistance between the various studies can be seen in Figure 4-21. This nearly constant cyclic resistance for samples prepared to different silt contents at a constant soil specific relative density agrees well with the findings of the current study.

4.3.5 Cyclic Resistance With Constant Sand Skeleton Void Ratio

Several studies have shown that cyclic resistance is more closely related to sand skeleton void ratio than it is to gross void ratio, gross relative density, or fines content. Tronsco and Verdugo (1985) found that at a constant sand skeleton void ratio, the cyclic resistance of a sand is constant with increasing silt content. Shen et al., (1977), Kuerbis et al., (1988), and Vaid, (1994) however, have shown that at a constant sand skeleton void ratio, the cyclic resistance of a sand does not remain constant but increases with increasing silt content. Both of these trends were noted in this study and can each be explained in terms of soil specific relative density.

Sand skeleton void ratio is the void ratio that would exist in the soil if all of the silt and clay particles were removed, leaving only the sand grains to form the soil skeleton. Finn et al. (1994) report that as long as there is room for the silt within the voids produced by the sand, the liquefaction resistance is independent of silt content.

As stated by Finn et al. (1994), the relationship between cyclic resistance and sand skeleton void ratio is only applicable to soils which have sufficient room in the voids created by the sand skeleton to accommodate all of the silt present: that is, soils below their limiting silt contents. Above the limiting silt content, it has been seen that cyclic resistance is controlled by the silt fraction and is independent of the sand skeleton. For many sands, below the limiting silt content, the maximum and minimum index void ratios decrease as silt is added not because the sand skeleton is altered but because the voids in the sand skeleton are being filled, thereby decreasing the gross void ratio of the soil. This behavior occurs in the Monterey sand, which has a nearly linear decrease in index void ratio with an increase in silt content. This behavior may be seen in Figure 4-22.

Also shown in Figure 4-22 is a curve representing the gross void ratios of several Monterey sand and silt mixtures that correspond to a sand skeleton void ratio of 0.75. For silt contents below 20 percent, the gross void ratio curve approximately parallels the curves for both the maximum and minimum index void ratios. This parallel decrease occurs because, for this soil, as the voids in the sand skeleton are filled, the gross void ratio decreases at the same rate as the maximum and minimum void ratios. Because the three different void ratios all decrease at approximately the same rate over this range of silt contents, the soil specific relative density remains nearly constant for a given sand skeleton void ratio, which in turn produces nearly constant cyclic resistance.

The cyclic resistances of the Monterey sand specimens prepared at various silt contents to a constant sand skeleton void ratio of 0.75 are shown in Figure 4-23. As expected for soils with nearly constant soil specific relative densities, the cyclic resistances are essentially constant with increasing silt content. This shows that for soils in which constant sand skeleton void ratios produce constant soil specific relative densities, the cyclic resistance remains constant. This matches the behavior reported by Tronsco and Verdugo (1985).

Not all soils however, exhibit a constant cyclic resistance with for a constant sand skeleton void ratio. Shen et al. (1977), Kuerbis et al. (1988), and Vaid (1994) have shown that for specimens prepared to a constant sand skeleton void ratio, the cyclic resistance of a sand does not remain constant, but increases with increasing silt content. The soil mixtures formed using Yatesville sand exhibited this behavior. Figure 4-24 shows the increase in cyclic resistance with increasing silt content measured for Yatesville sand specimens prepared to various silt contents at constant sand skeleton void ratios of 0.90 and 1.00.

The increase in cyclic resistance measured for these soils at a constant sand skeleton void ratio occurs because as their silt content increases, their maximum and minimum index void ratios decrease less rapidly than their gross void ratios. As a result, although the sand skeleton void ratio remains constant, the soil specific relative densities increases with increasing silt content.

Figure 4-25 shows that the gross void ratio of specimens of Yatesville sand prepared to a constant sand skeleton void ratio of 0.90 decreases more rapidly with increasing silt content than do the index void ratios. This results in an increase in soil specific relative density with increasing silt content which, in turn, causes an increase in cyclic resistance. This is confirmed in Figure 4-26 where the results presented in Figure 4-24 are replotted in terms of their soil specific relative densities. These specimens of Yatesville sand,

despite having different sand skeleton void ratios and silt contents, exhibit the same variation in cyclic resistance when examined in terms of their soil specific relative densities.

From the studies of specimens prepared to a constant sand skeleton void ratios it can be seen that some soils such as Monterey sand have a very strong link between their sand skeleton void ratios and their cyclic resistances, while other soils such as Yatesville sand are not as well correlated. Soils, such as Monterey sand, which exhibit constant cyclic resistances with constant sand skeleton void ratio appear to do so because their soil specific relative densities remain relatively constant with increasing silt content. Other soils, such as Yatesville sand, which exhibit an increase in cyclic resistances with constant sand skeleton void ratio appear to do so because their soil specific relative densities increase with increasing silt content. Whether the cyclic resistance remains constant or increases when the sand skeleton void ratio is held constant depends the rate at which the maximum and minimum index void ratios change relative to the gross void ratio as silt content increases.

4.3.6 Increasing Cyclic Resistance With Increasing Silt Content

Several investigators have found that cyclic resistance increases with increasing silt content. For specimens prepared to a constant gross void ratio, Chang, et al. (1982), found that after a small initial drop, cyclic resistance increased dramatically with increasing silt content. The silt used in the study was slightly plastic, having a plasticity index of 5. Results of these tests are presented in Figure 4-27 where the dramatic increase in cyclic resistance can be seen. The normalized cyclic resistance increased nearly linearly with silt content until a silt content of 60 percent is reached, increasing to a cyclic resistance between 50 and 60 percent greater than that of the clean sand. Similarly, Dezfulian (1982) also reported a trend of increasing cyclic resistance with

increasing silt content. Both studies used silts with some small level of plasticity or a measurable clay fraction.

No evidence was found during the current study to either support or explain these large increases in cyclic resistance with increasing silt content. One clue to the reason for this behavior lies in the fact that it was only reported for studies that used silts with some plasticity. As has been reported in the literature (Lee and Fitton, 1968; Ishihara and Koeski, 1989; and Yasuda, et al.; 1994), an increase in the plasticity of the fine-grained fraction will increase the cyclic resistance of a sand.

While it is possible that the increase in cyclic resistance reported by Chang, et al. (1982) and Dezfulian (1982) is due to the plastic nature of the fine grained material, it should be noted that Shen et al. (1977) studied soils with a higher plasticity index than those reported by Chang, et al. (1982), yet reported no increase in cyclic resistance due to the plasticity of the fines.

4.4 Conclusions

Numerous studies reported in the literature have produced varying answers to the question of what effect increasing non-plastic silt content has upon the liquefaction resistance of a sand. In order to answer this question and to reconcile the results of these studies, a study utilizing nearly 300 cyclic triaxial tests was performed using two base sands and 19 combinations of sand and non-plastic silt. Tests were run at varying silt contents and the results interpreted in terms of gross void ratio, gross relative density, and skeleton void ratio, and soil specific relative density. Several conclusions regarding the effects of non-plastic fines on the liquefaction susceptibility of sandy soils were drawn from this study.

From this study three distinct behavioral patterns were found for the cyclic resistance of soils composed of sand and non-plastic silt. Which of these behaviors controls is determined by whether there is sufficient room in the voids created by the sand skeleton to contain the silt present without disturbing the sand structure. This silt content has been called the limiting silt content and occurred between 30 and 40 percent for the sands used in this study.

If the silt content of the soil is below the limiting silt content, there is sufficient room in the voids created by the sand skeleton to contain the silt, and the soil can be described as having silt contained in a sand matrix. The cyclic resistance of the soil is then controlled by the soil specific relative density of the specimen, where the soil specific relative density is calculated using the gross void ratio of the specimen and the maximum and minimum index void ratios for that particular mixture of sand and silt. The cyclic resistance for these soils is independent of silt content. Increasing the soil specific relative density increases the soil's cyclic resistance.

If the silt content is greater than the limiting silt content, the specimen's structure consists predominately of sand grains suspended within a silt matrix with little sand grain to sand grain contact. Above the limiting silt content, the amount of sand present in the soil and its soil specific relative density have little effect on its cyclic resistance. The cyclic resistance for these soils is controlled by the silt fraction void ratio of the soil. Decreasing the silt fraction void ratio increases the soil's cyclic resistance.

There is a transition zone consisting of soils with silt contents at or slightly above the limiting silt content. This zone occurs as a result of the structure of the soil changing from a predominately sand controlled fabric to a predominately silt controlled fabric as silt content increases. Soils in this zone possess cyclic resistances intermediate to those with greater or lesser silt contents.

The seemingly contradictory reports concerning the effects of non-plastic fines content on the liquefaction resistance of sands that have appeared in the literature were reconciled in light of the behavioral patterns found in this study. These behaviors were found to depend primarily upon the limiting silt content of the soil and the soil specific relative density of the specimen. It was shown that the trends of decreasing cyclic resistance or decreasing and then increasing cyclic resistance with increases in silt content reported in the literature could be explained in view of the results of this study. The concept that cyclic resistance is controlled by the sand skeleton void ratio of the soil was also reconciled with the results of this study. The trend of increasing cyclic resistance with increasing silt content which has been reported in the literature does not appear to occur in non-plastic silts and is likely due to the plasticity of the fines used in those studies.

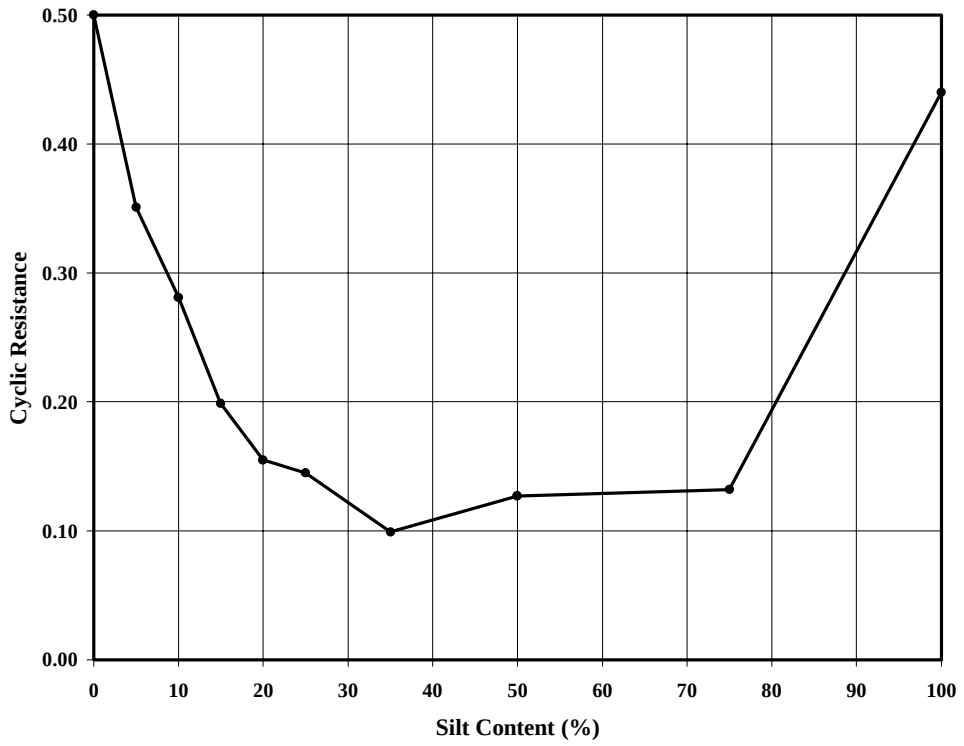

Figure 4-1: Variation in cyclic resistance with silt content for Monterey sand at constant gross void ratio of 0.68

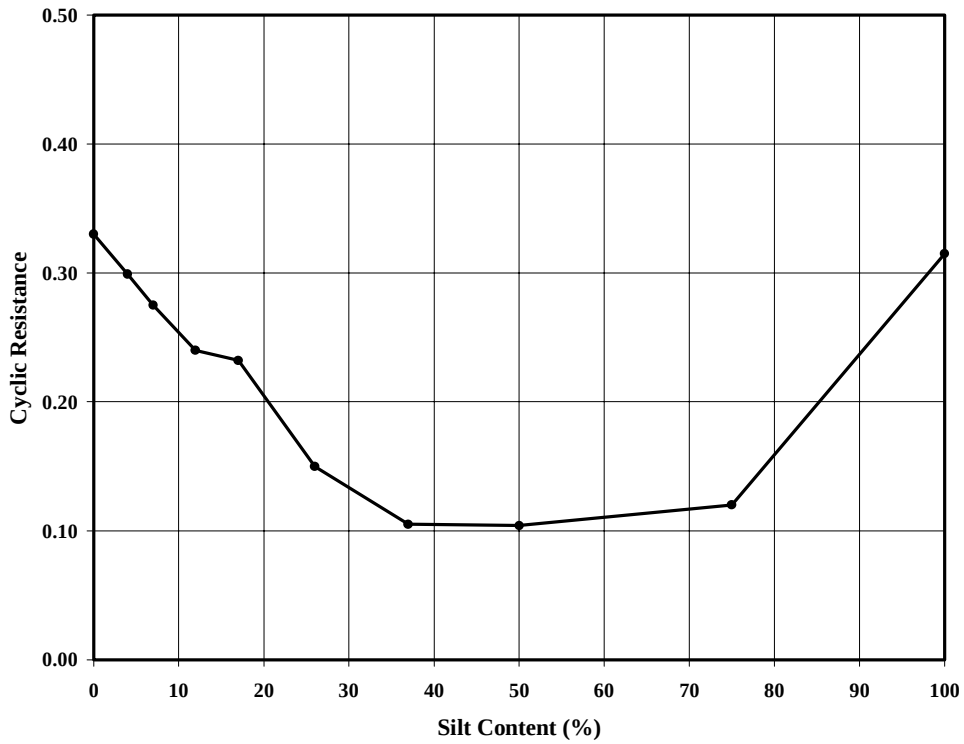

Figure 4-2: Variation in cyclic resistance with silt content for Yatesville sand at constant gross void ratio of 0.76

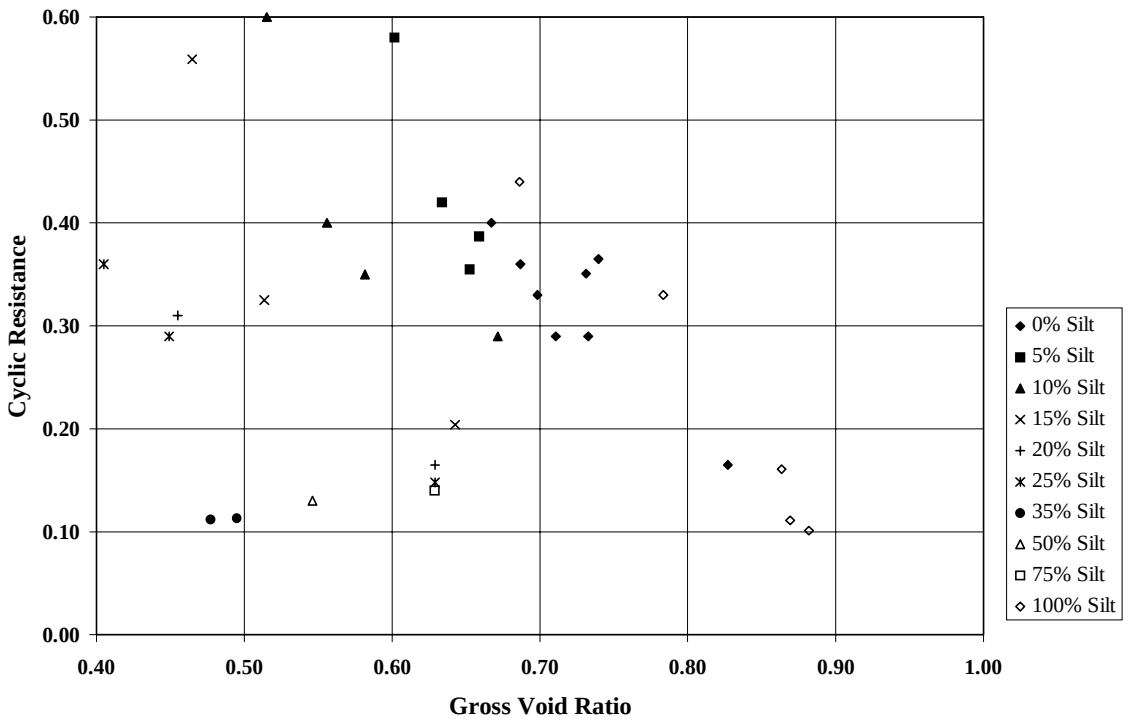

Figure 4-3: Variation in cyclic resistance with gross void ratio for Monterey sand

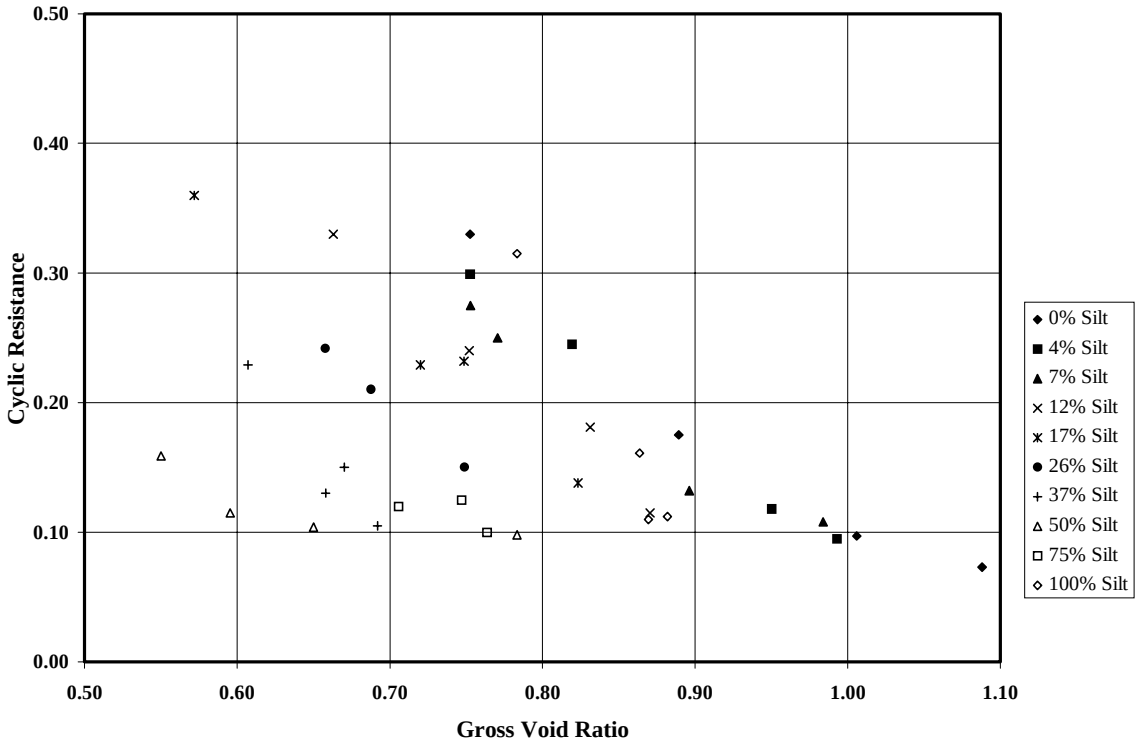

Figure 4-4: Variation in cyclic resistance with gross void ratio for Yatesville sand

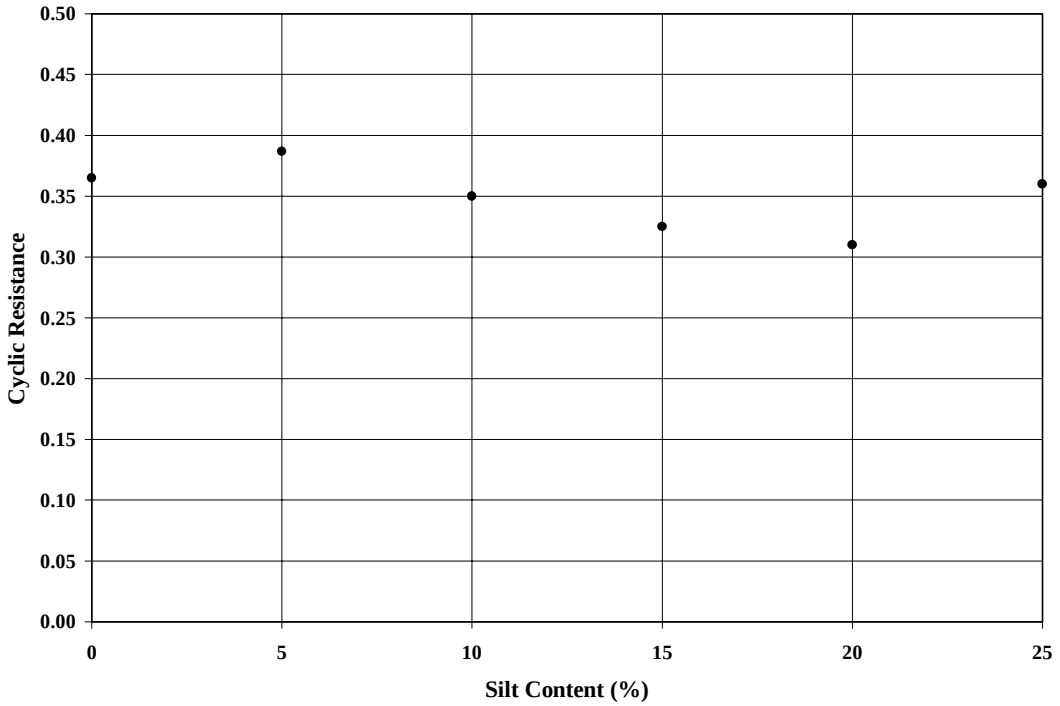

Figure 4-5: Variation in cyclic resistance for Monterey sand specimens prepared to a constant sand skeleton void ratio of 0.75

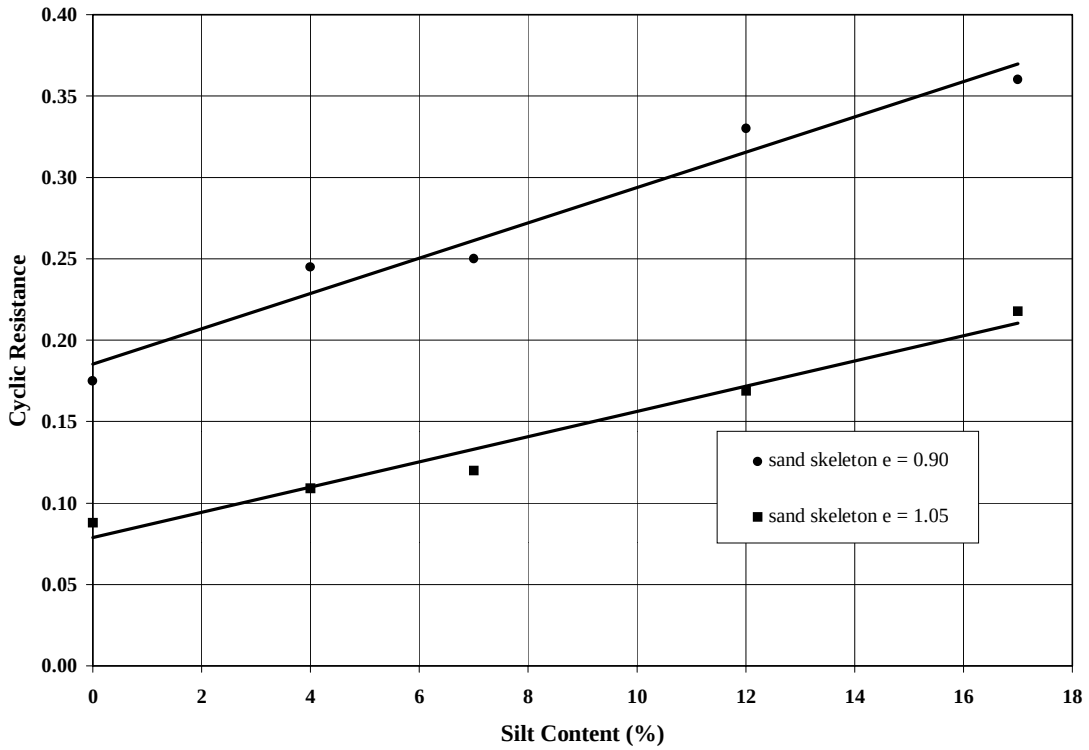

Figure 4-6: Variation in cyclic resistance for Yatesville sand specimens prepared to constant sand skeleton void ratios

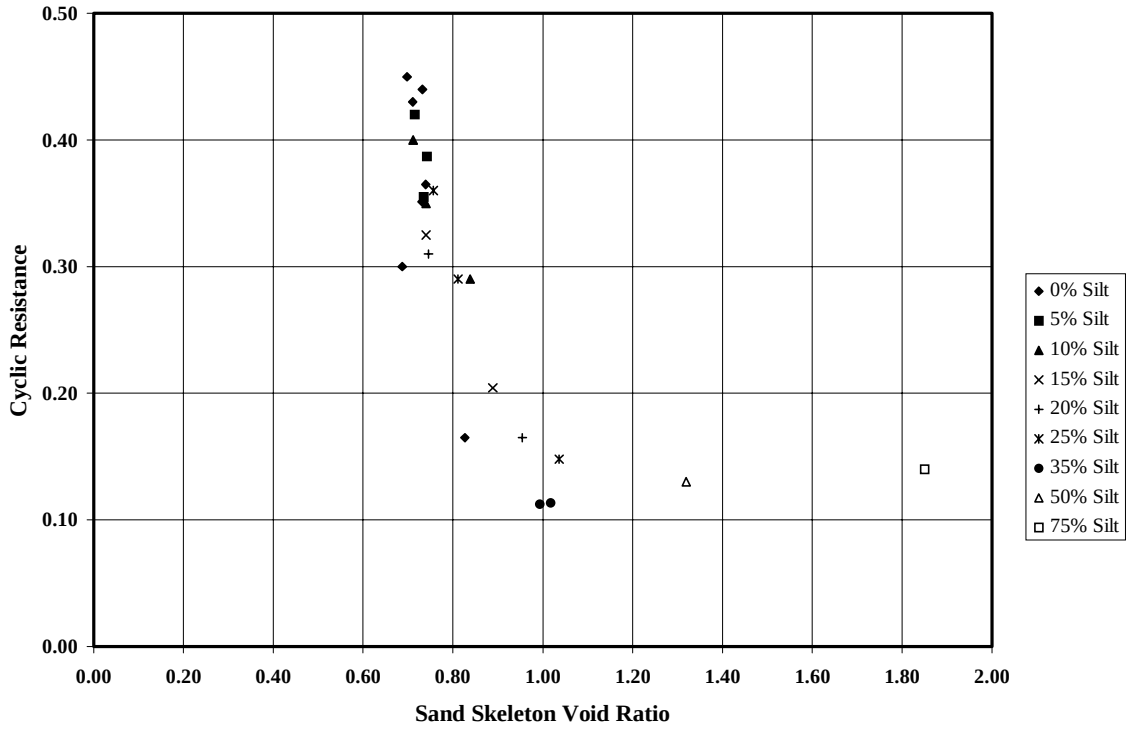

Figure 4-7: Variation in cyclic resistance with sand skeleton void ratio for Monterey sand

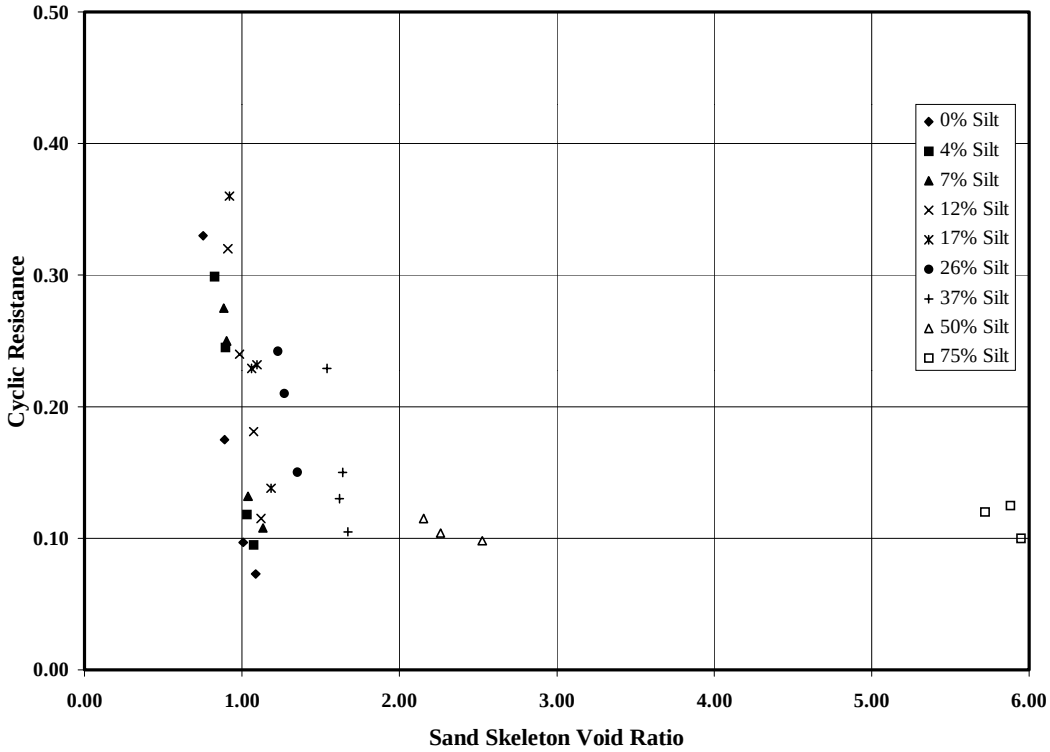

Figure 4-8: Variation in cyclic resistance with sand skeleton void ratio for Yatesville sand

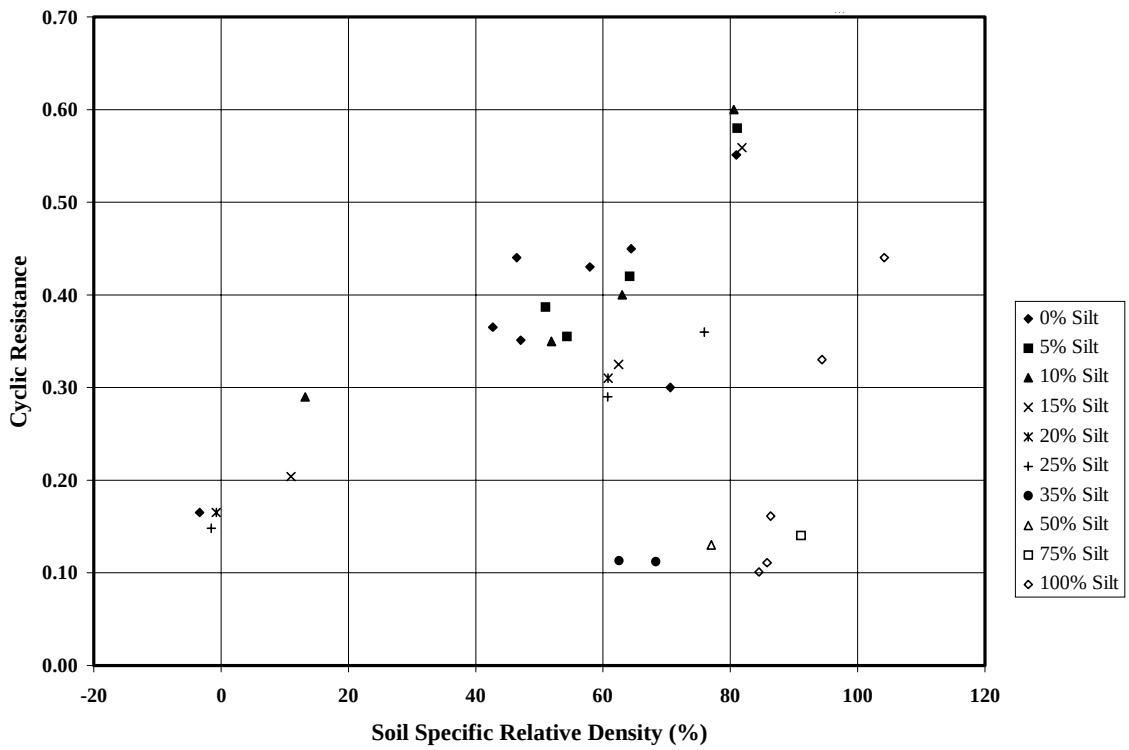

Figure 4-9: Variation in cyclic resistance with soil specific relative density for Monterey sand

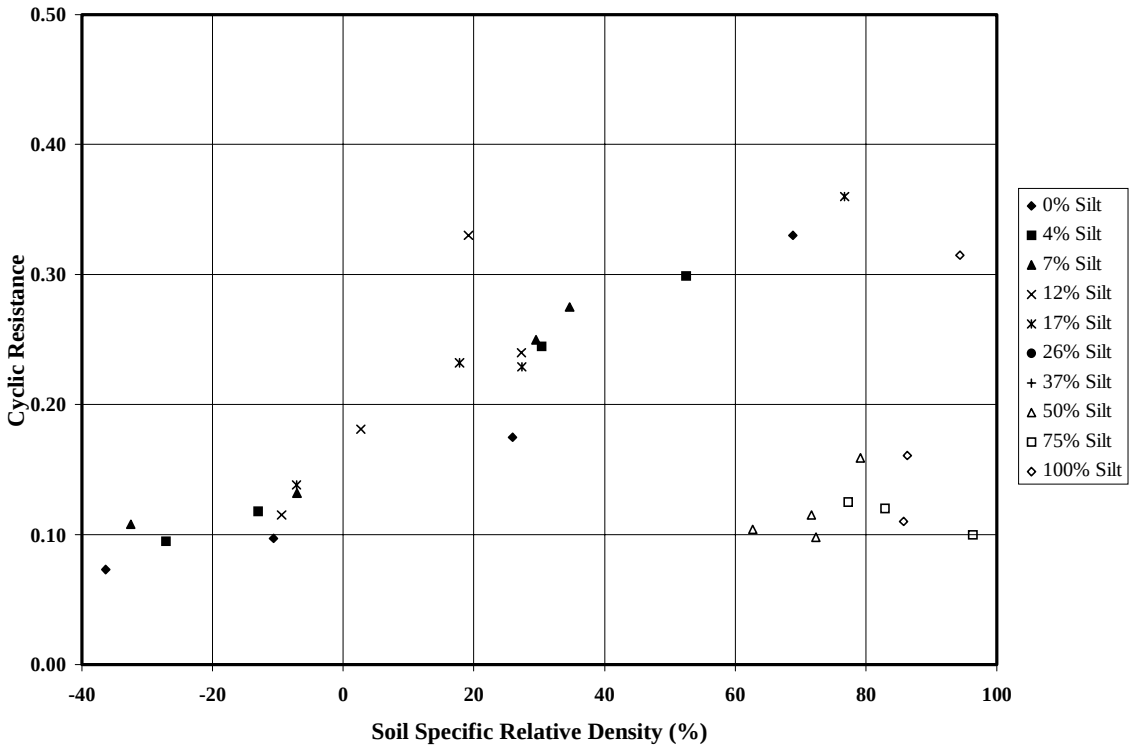

Figure 4-10 - Variation in cyclic resistance with soil specific relative density for Yatesville sand

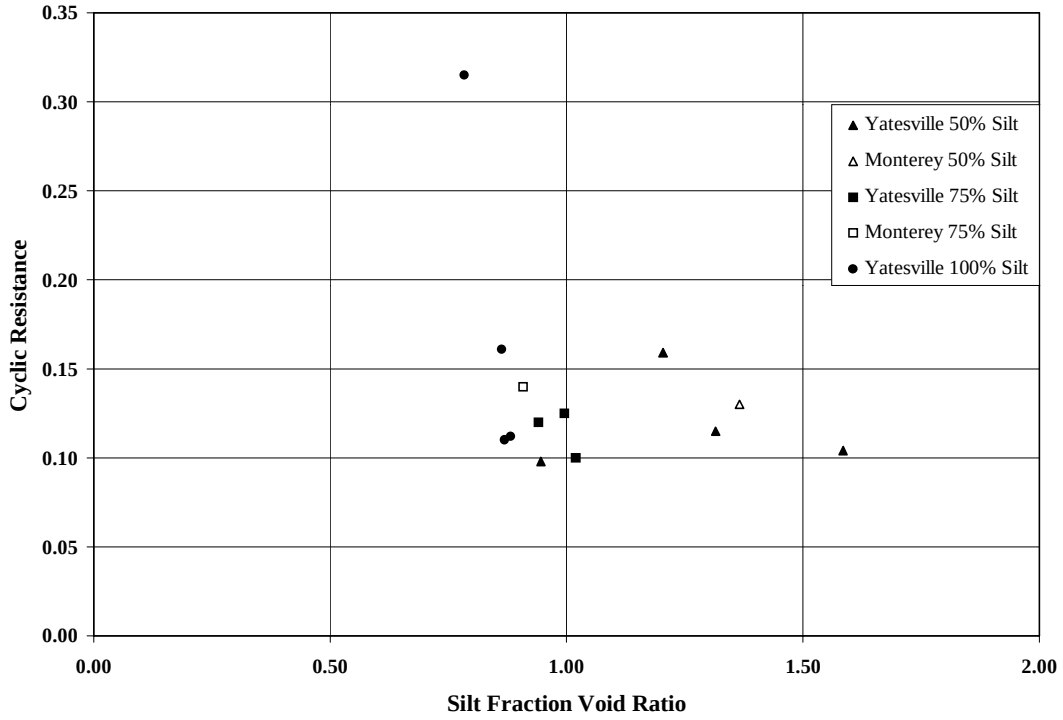

Figure 4-11 - Variation in cyclic resistance with silt fraction void ratio for soils with silt contents above the limiting silt content

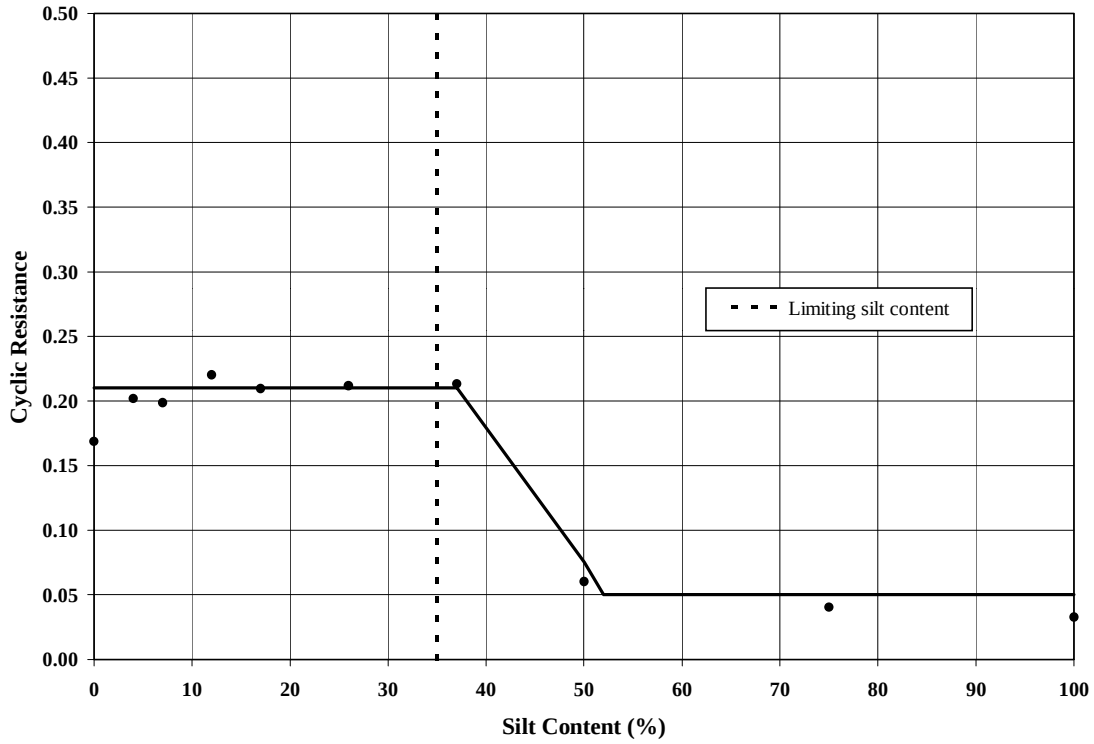

Figure 4-12: Variation in cyclic resistance with silt content for Yatesville sand specimens adjusted to 25% soil specific relative density

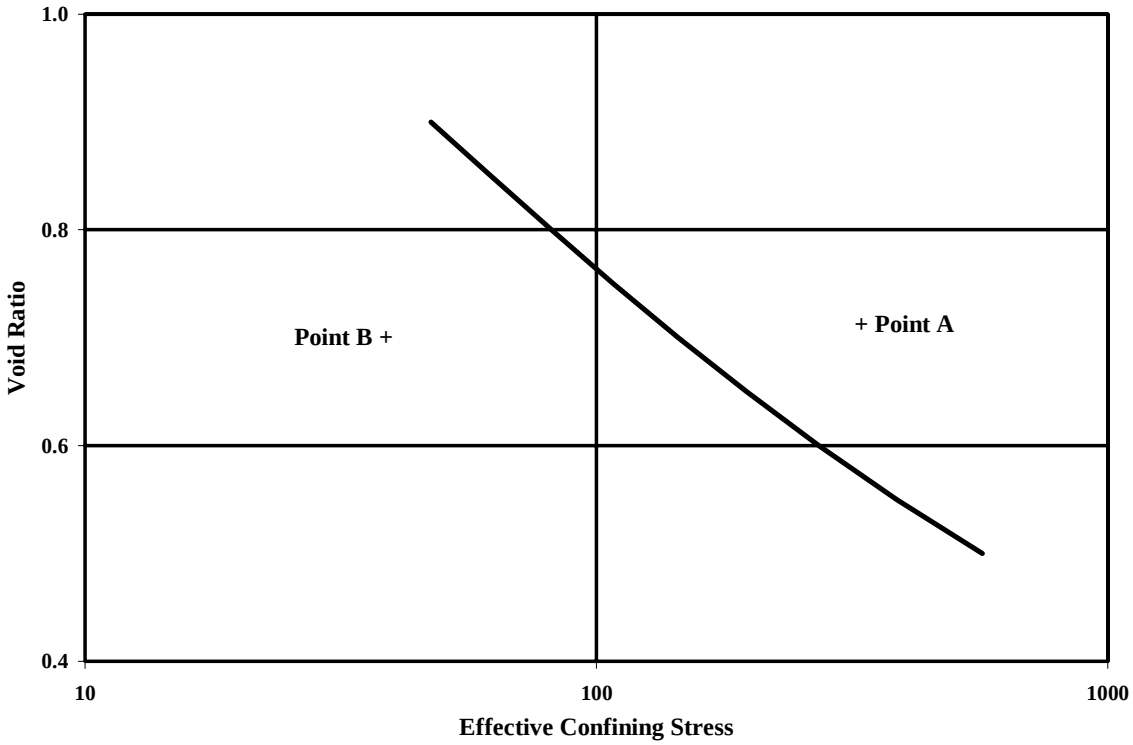

Figure 4-13: Example of a steady-state line

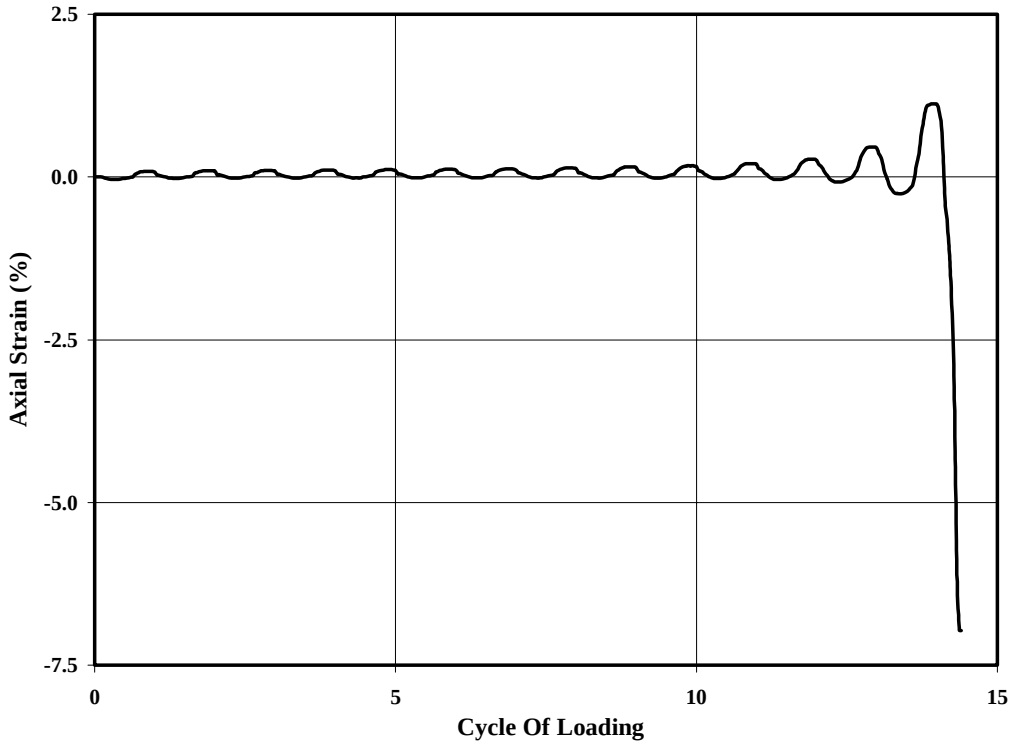

Figure 4-14: Typical strain behavior for a specimen susceptible to flow liquefaction

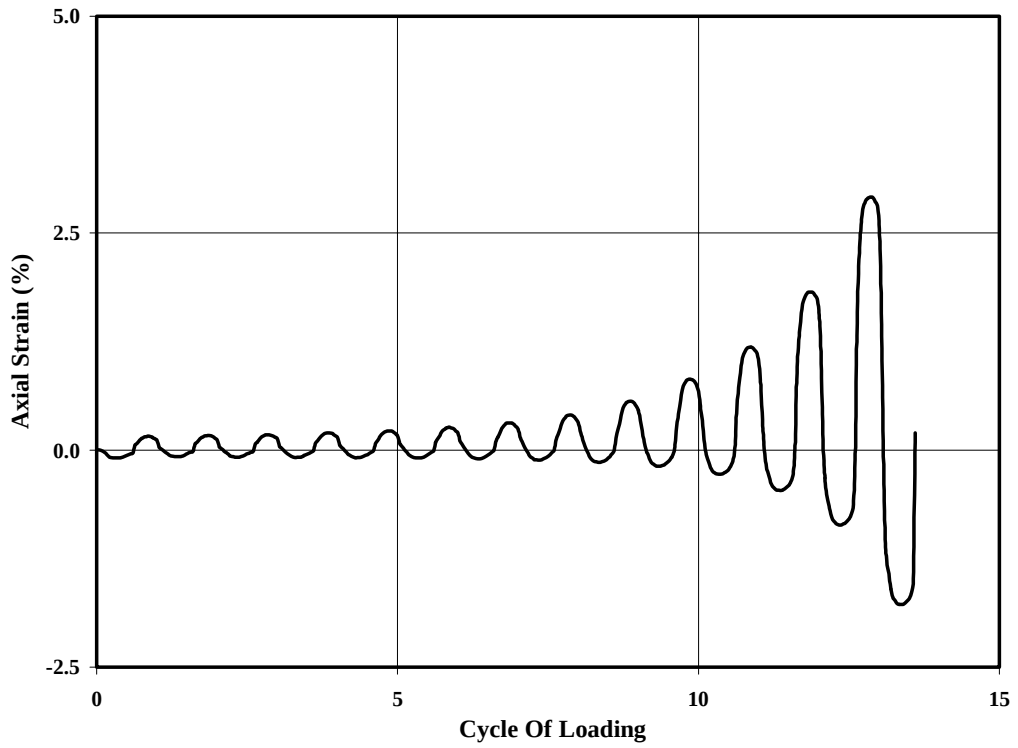

Figure 4-15: Typical strain behavior for a specimen susceptible to cyclic mobility

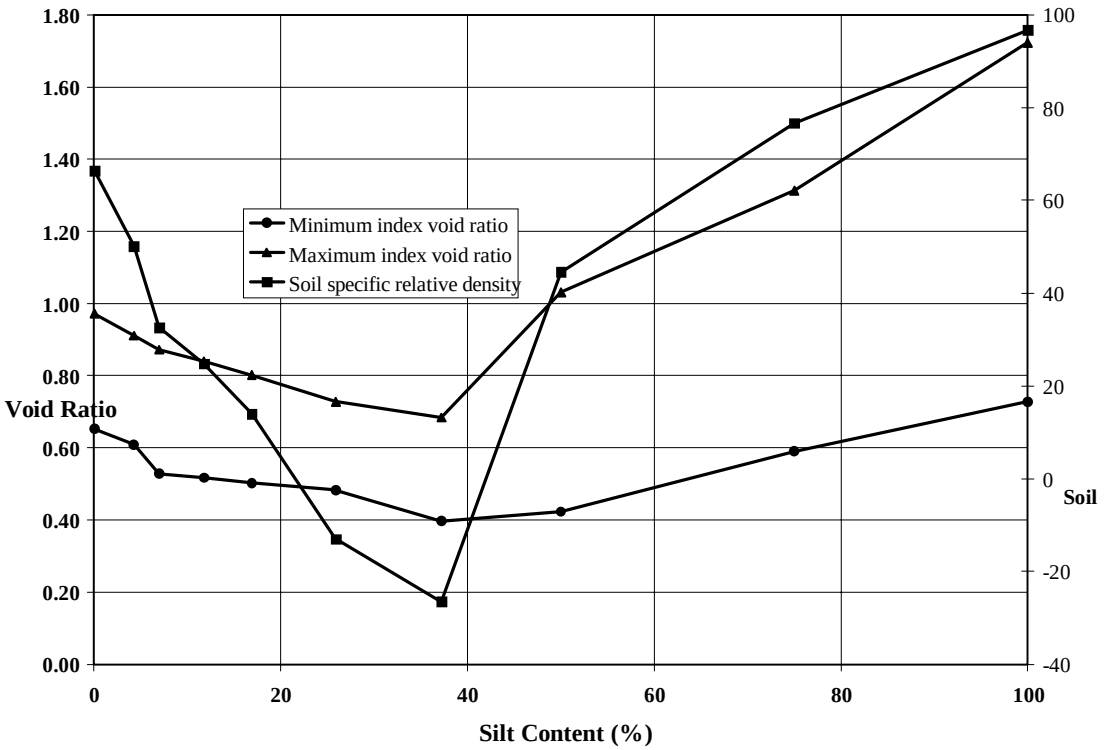

Figure 4-16: Variation in index void ratios and soil specific relative density for Yatesville sand specimens prepared to a constant gross void ratio of 0.76

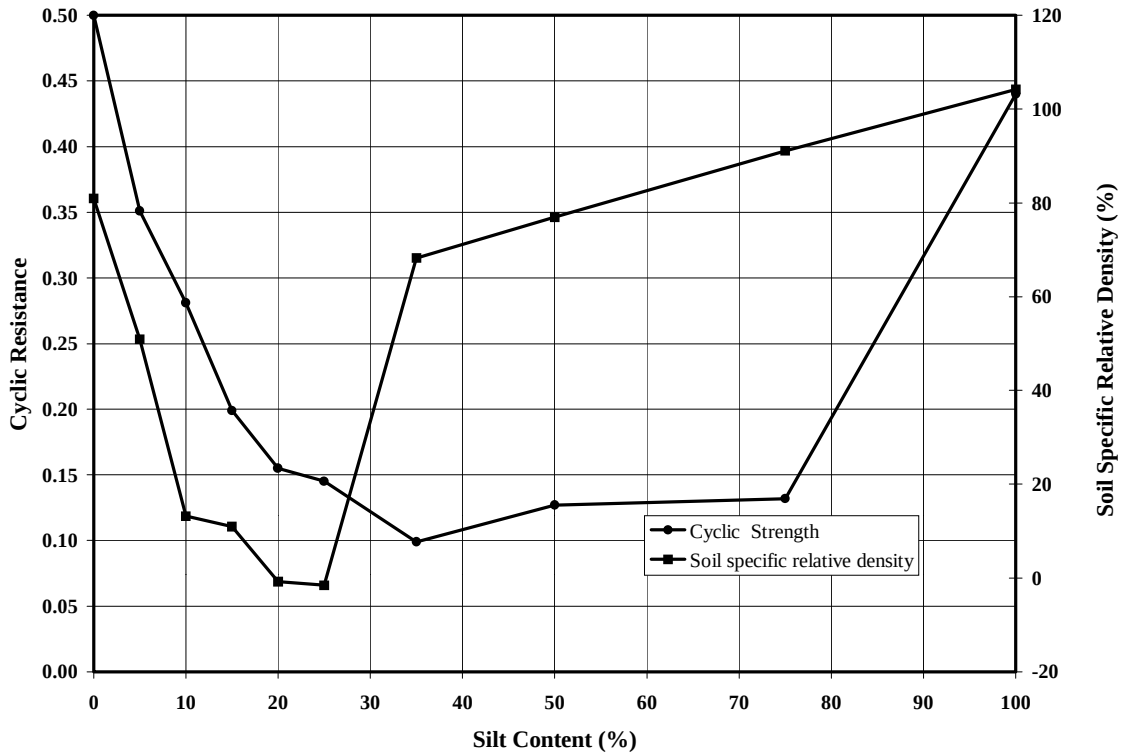

Figure 4-17: Variation in cyclic resistance and soil specific relative density for Monterey sand specimens prepared to a constant gross void ratio of 0.68

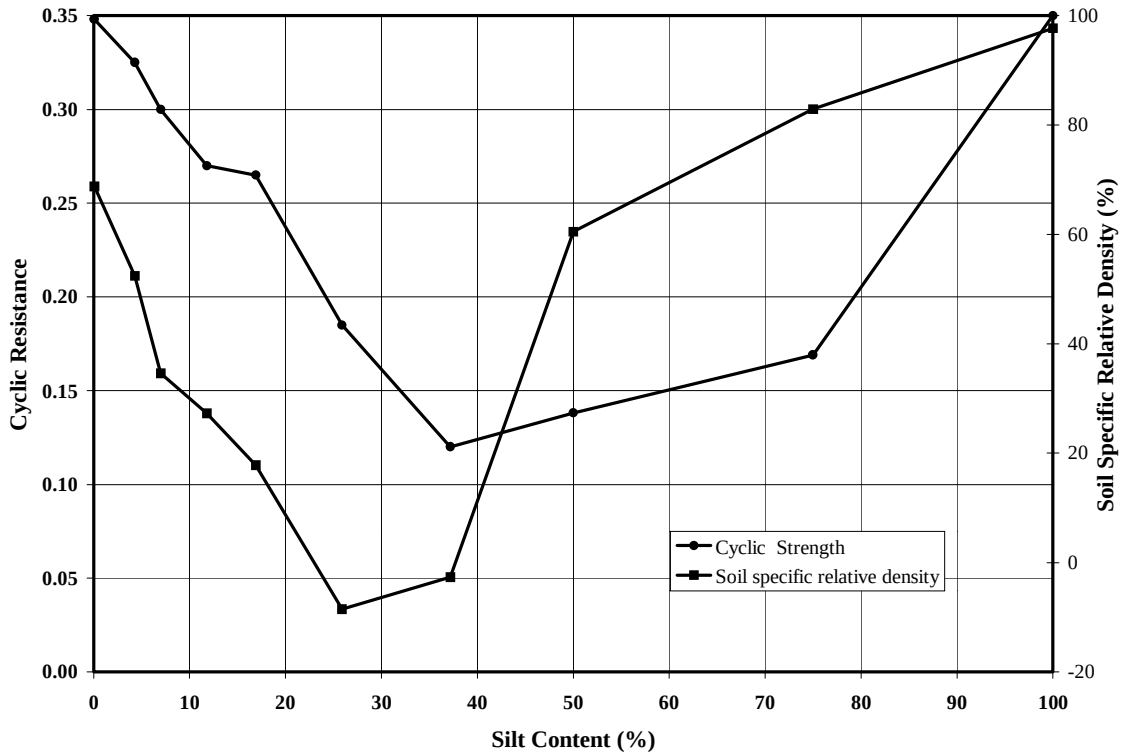

Figure 4-18: Variation in cyclic resistance and soil specific relative density for Yatesville sand specimens prepared to a constant gross void ratio of 0.76

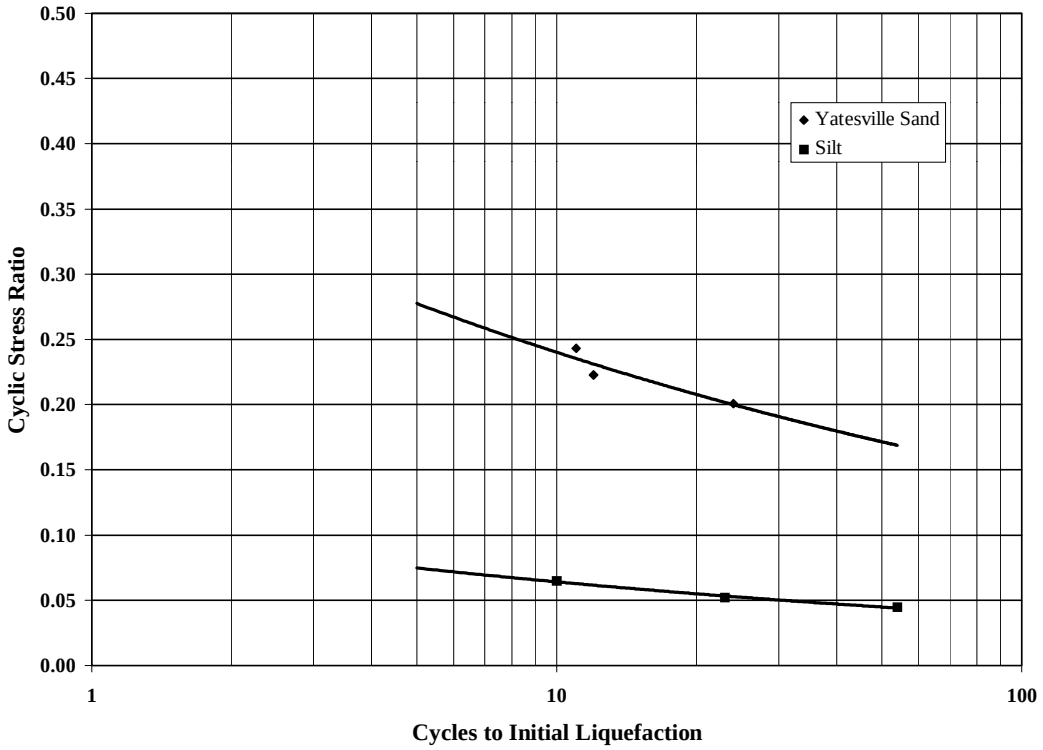

Figure 4-19: Number of cycles to initial liquefaction versus cyclic stress ratio for Yatesville sand and silt at 50 percent soil specific relative density

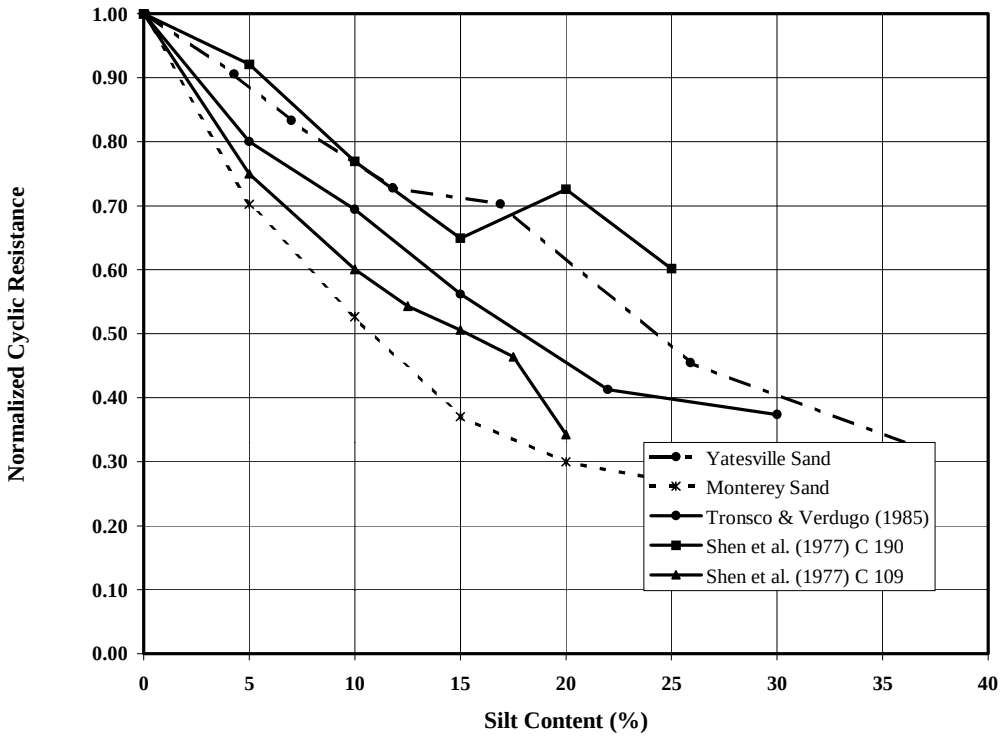

Figure 4-20: Comparison of variations in normalized cyclic resistance between data from the current and published studies

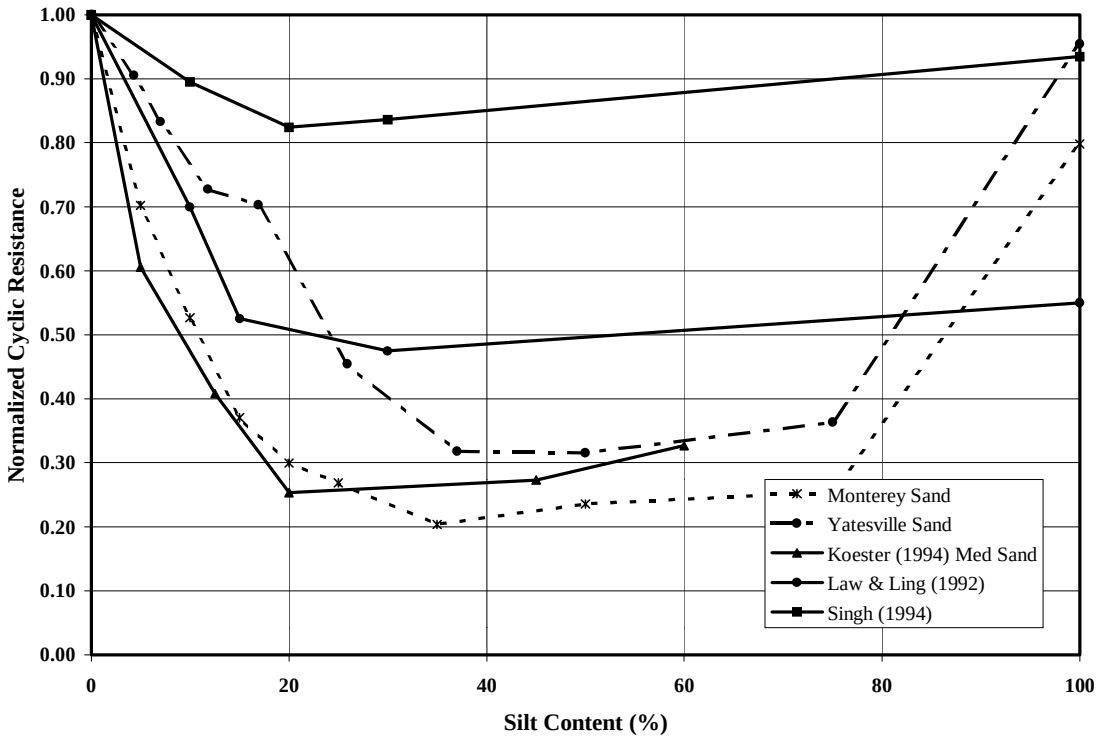

Figure 4-21: Comparison of variations in normalized cyclic resistance between data from the current and published studies

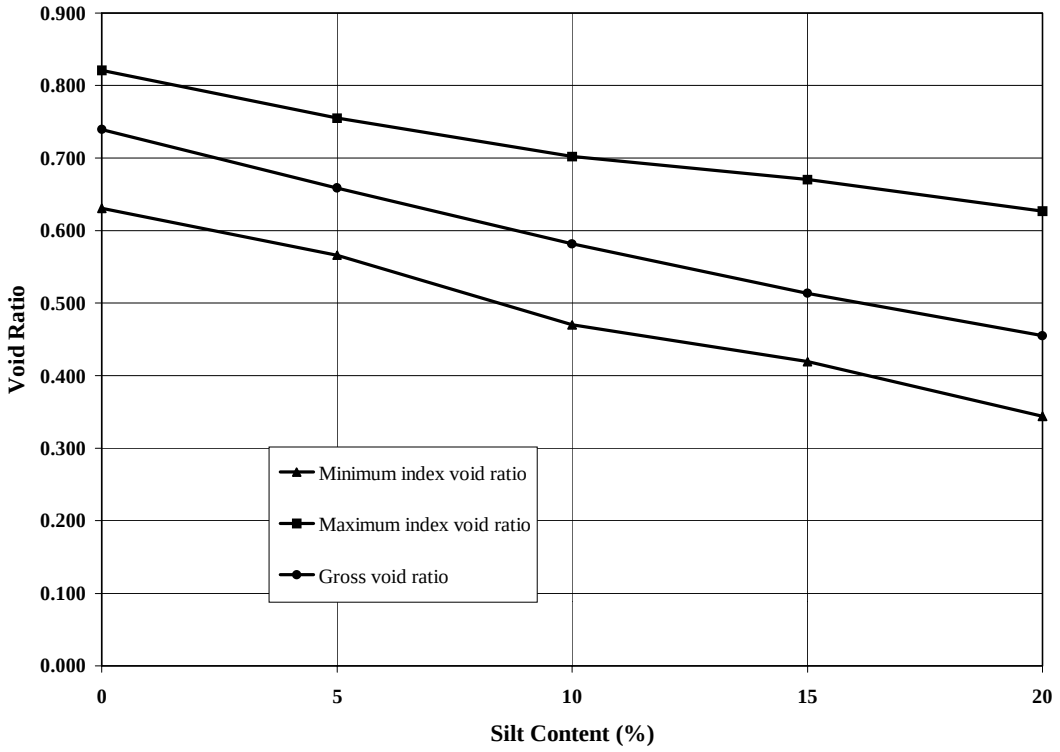

Figure 4-22: Variation in index void ratios and gross void ratio for Monterey sand specimens prepared to a constant sand skeleton void ratio of 0.75

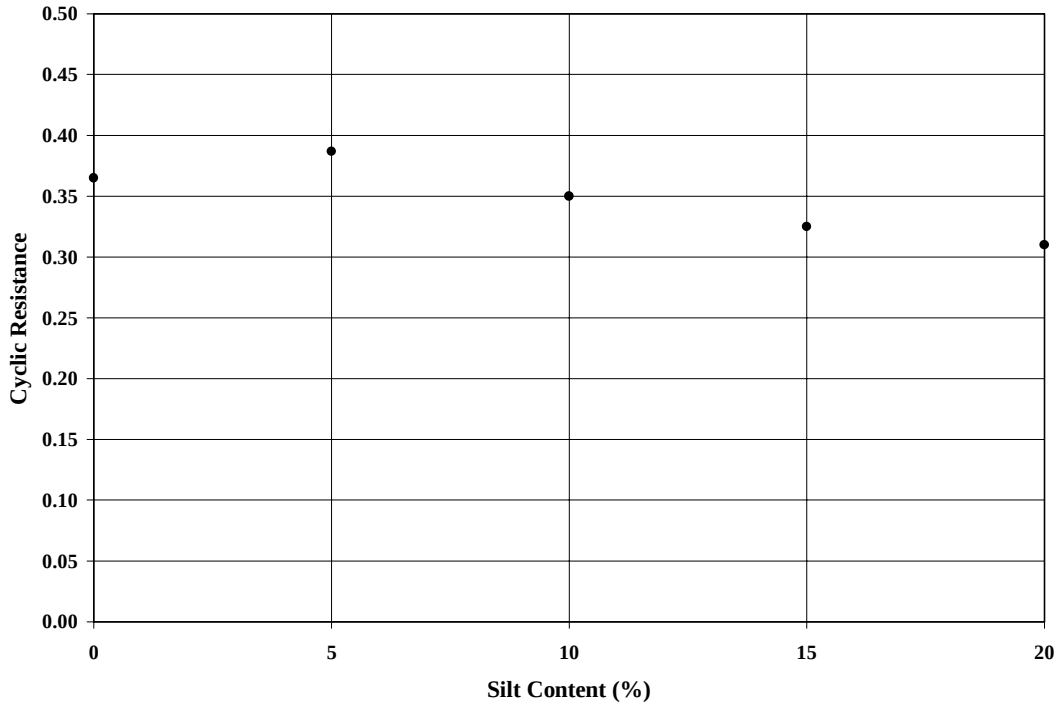

Figure 4-23: Variation in cyclic resistance for Monterey sand specimens prepared to a constant sand skeleton void ratio of 0.75

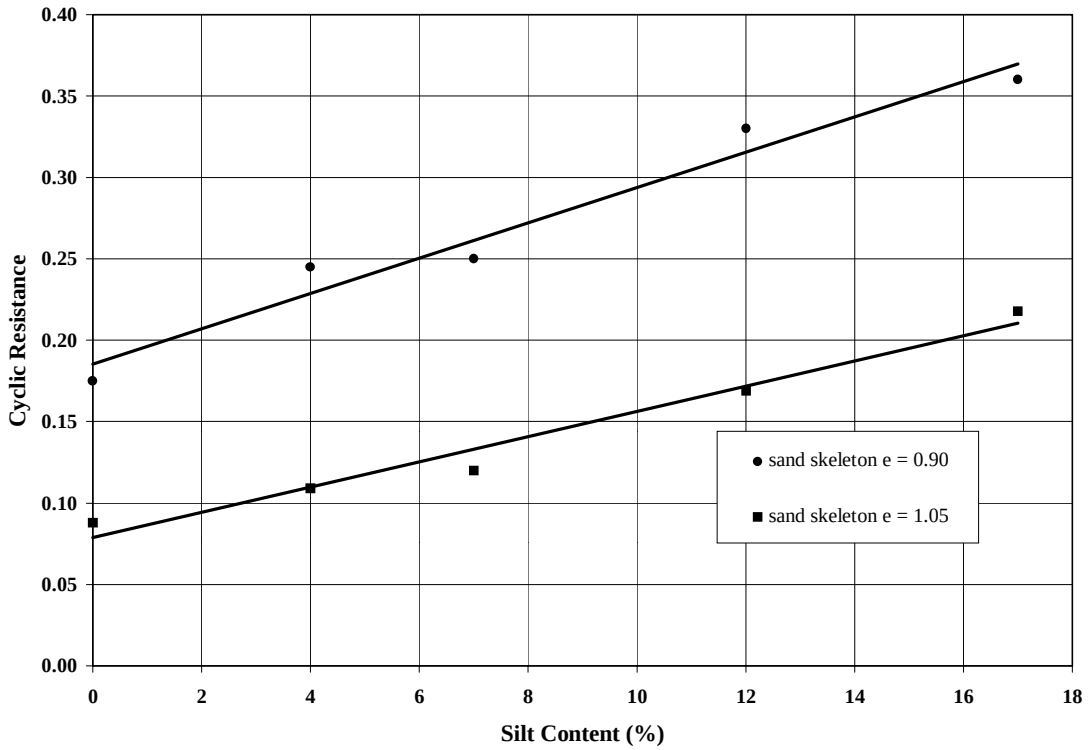

Figure 4-24: Variation in cyclic resistance for Yatesville sand specimens prepared to constant sand skeleton void ratios

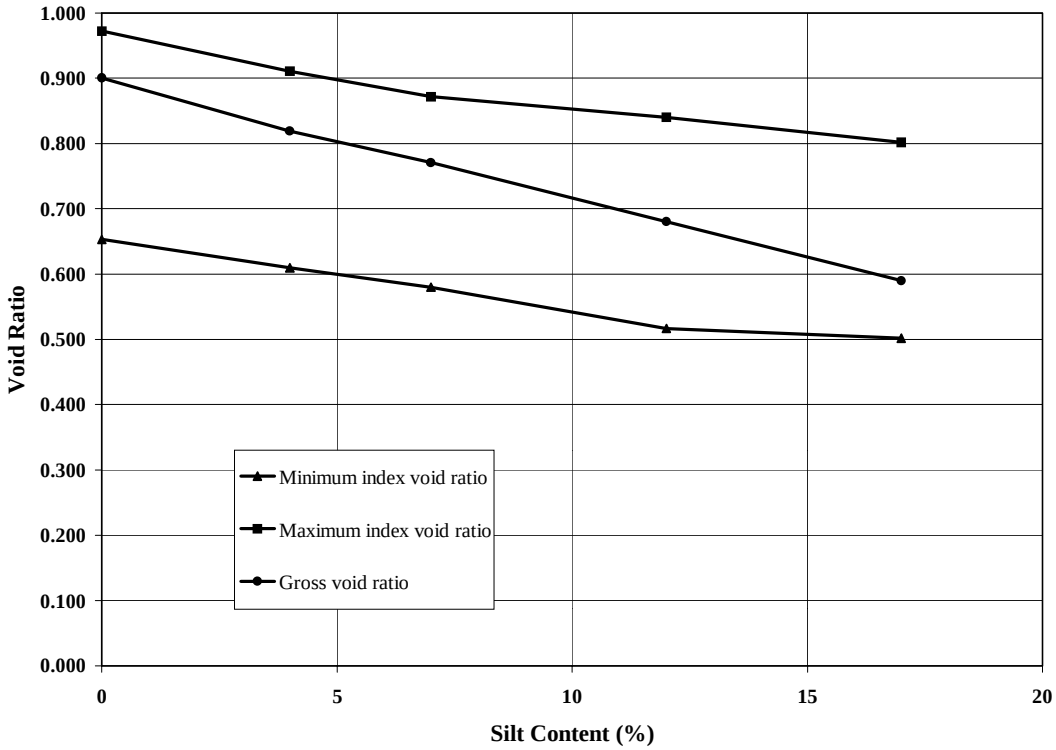

Figure 4-25: Variation in index void ratios and gross void ratio for Yatesville sand specimens prepared to a constant sand skeleton void ratio of 0.90

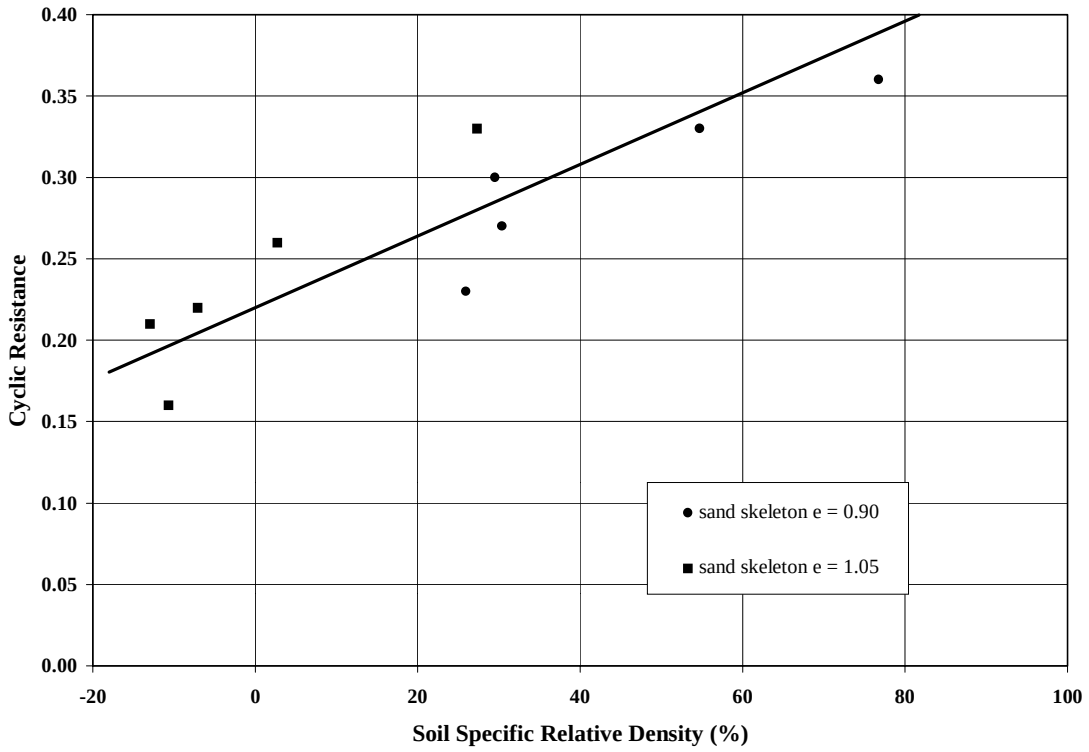

Figure 4-26: Variation in cyclic resistance with soil specific relative density for Yatesville sand specimens prepared to constant sand skeleton void ratios

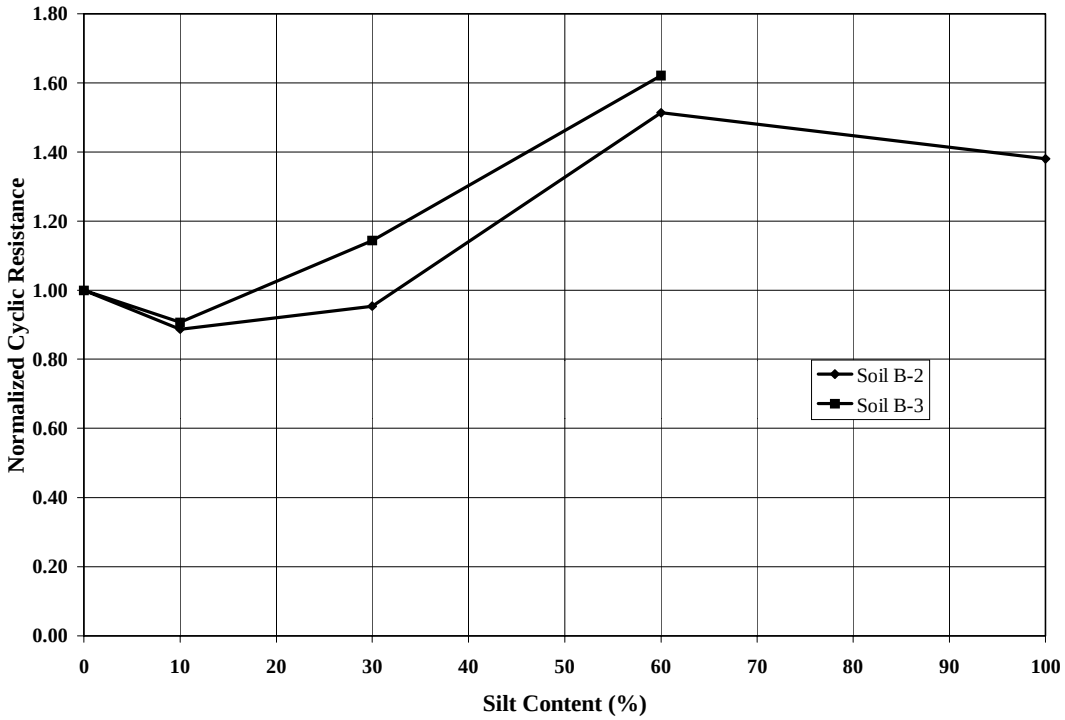

Figure 4-27: Increase in normalized cyclic resistance with increasing silt content

Chapter 5: The Effects Of Plastic Fines And Plasticity Based Liquefaction Criteria

A series of cyclic triaxial tests were run to determine the effects which the percentage and plasticity of the fine-grained material have upon the cyclic resistance of clayey sands. The specimens were prepared at a constant soil specific relative density at fines contents below the limiting silt content. The fine-grained material consisted of various combinations of Yatesville silt, kaolinite, and bentonite.

While the majority of the data in the literature would predict an increase in cyclic resistance as the clay content of the specimen increased, the study found a different behavior to occur. As the clay content increased, the cyclic resistance (as based upon initial liquefaction) was found to decrease until some percentage of clay was present, at which point the cyclic resistance began slowly increasing. Even at clay contents well above those which previous studies would seem to indicate should make the soil non-liquefiable, the soils was found to have a lower cyclic resistance than a comparable specimen with non-plastic fines.

Following a brief discussion of the results found in the literature and the testing program, this chapter will examine the effects which increasing clay content and plasticity have on cyclic resistance when measured by several different parameters. Next, the results of an evaluation of plasticity based liquefaction criteria, such as the Chinese criteria, are discussed. Lastly, a possible explanation for the observed behaviors is presented.

5.1 Previous Studies

The addition of plastic or clayey fines is widely thought to increase the liquefaction resistance of a sand. While the addition of even small amounts of fines is believed to increases the liquefaction resistance of a sand, the addition of sufficient clayey fines is

even thought to make a soil non-liquefiable. Field studies by Tokimatsu and Yoshimi, (1983), indicates that sands with more than 20 percent clay will not liquefy. Seed et al. (1983) reached a similar conclusion. Ishihara (1993) reports two Japanese studies that predict that soils with greater than 10 percent clay are non-liquefiable. Additionally, laboratory studies by Ishihara and Koseki (1989) and Yasuda et al. (1994) found a strong correlation between increased plasticity of the fines and increased cyclic resistance. Lee and Fitton (1968) concluded that the addition of clay fines may increase the cyclic resistance of a soil considerably.

5.2 Testing Program

A series of cyclic triaxial tests were performed in order to determine the effects which increased plastic fines content and plasticity have upon the liquefaction resistance of soils. During testing it was found that in order to obtain meaningful results the soil must be examined in terms of its overall plasticity, not just the plasticity of the fine-grained fraction.

5.2.1 Soils Tested

The testing program performed to examine the effects which the addition of plastic fines have upon the liquefaction resistance of sand is similar to the testing program used to study the effects of non-plastic fines. Approximately 45 cyclic triaxial tests were run on 14 combinations and percentages of kaolinite, bentonite, and Yatesville silt. Additionally, the results of the tests performed on specimens prepared with non-plastic fines at similar fines contents and densities were included in the evaluations.

As similar trends where found for the effects of non-plastic fines on both the Monterey No. 0/30 sand and the Yatesville sand, it was decided to perform the investigation into the effects of plastic fines using only the Yatesville sand.

All tests were run at soil specific relative densities of approximately 25 percent. This density matches that used for a series of tests run with non-plastic fines, the results of which were presented in Figure 4-12. Fines contents in this portion of the study varied from 4 to 37 percent, with clay contents varying from 2 to 37 percent.

As in the study on non-plastic fines, a minimum of three tests were performed at varying cyclic stress ratios for each combination of fines. The number of cycles to initial liquefaction was then plotted against the applied CSR's and the cyclic resistance of the soil determined. Again, the cyclic resistance was defined as the cyclic stress ratio required to cause initial liquefaction in 10 cycles.

5.2.2 Measurement of Plasticity

The plasticity of the soil mixtures were determined by Atterberg limits tests on the portion of the soil passing the Number 40 sieve as specified by ASTM D 4318 Standard Test Method For Liquid Limit, Plastic Limit, And Plasticity Index Of Soils. The plasticity data reported herein is based on this test methodology.

Numerous studies performed in the past have analyzed the effects of plastic fines based on the plasticity of the fine-grained material alone. This may lead to erroneous results as it neglects the effects that the percentage of fines in the soil has upon the Atterberg Limits of that soil. For example, the pure kaolinite used in this study has a liquid limit of 58 and a plasticity index of 31, but Yatesville sand with 26 percent kaolinite has a liquid limit of 20 and a plasticity index of 7. In a more extreme example, Yatesville sand with 12 percent kaolinite classifies as non-plastic with a liquid limit of 17 and no plastic limit. These three soils have activities of 0.31, 0.27, and 0 respectively.

5.3 Results Of Testing

The findings of the laboratory study performed during this research do not support the conclusions of the studies found in the literature. The current study does not indicate that adding small amounts of clay to a sand increases its resistance to liquefaction, in fact, it indicates that the addition of small amounts of clay to a sand actually decreases the sand's ability to withstand liquefaction. It is not until a significant amount of highly plastic fines are added that the soil becomes less liquefiable than either the clean sand or a sand with a similar amount of non-plastic fines. These trends may be seen in Figure 5-1 which plots the cyclic resistances of soils prepared to a constant soil specific relative density as a function of their fines content and composition.

In order to determine what effects of fines content and type have on cyclic resistance of the soil, the cyclic resistances determined for each soil mixture were plotted against various parameters. This was done for all of the specimens from a clean sand to a sand with 37 percent fines. The fines ranged from pure silt to pure bentonite and pure kaolinite. The parameters examined include fines content, clay content, liquid limit, plasticity index, activity, void ratio, water content, and liquidity index.

Next, in order to isolate the effects of fines plasticity, cyclic resistances were plotted for specimens prepared to a constant fines content of 17 percent with varying fines composition. The fines ranged from non-plastic silt to a combination of half kaolinite and half bentonite. The parameters examined include clay content, liquid limit, plasticity index, activity, water content, and liquidity index.

5.3.1 Results Of All Tests Performed

The data from the entire plastic fines portion of the testing program and the results of selected tests with non-plastic fines were used in order to determine the combined effects of fines content and plasticity on the cyclic resistance of sandy soils. These effects were evaluated using common soil parameters.

5.3.1.1 The Effect Of Fines Content

Based upon the current study there is little correlation between the fines content of a sand and its cyclic resistance. As may be seen from Figure 5-2 there is little correlation between the quantity of fines present and the cyclic resistance of the soil. At a fines content of 17 percent for example, the cyclic resistance varies from a low of 0.11 to a high of 0.23, an increase of more than 100 percent.

This lack of correlation between fines content and cyclic resistance agrees with the findings of Ishihara and Koeseki (1989).

5.3.1.2 The Effect Of Clay Content

Similar to the effect of fines content, no clear correlation may be drawn regarding the effect of clay content on liquefaction resistance. As may be seen in Figure 5-3, for clay contents below 20 percent, there is a weak trend of decreasing cyclic resistance with increasing clay contents. For specimens prepared to a constant soil specific relative density of approximately 25 percent, it is interesting to note that specimens containing non-plastic fines have the highest cyclic resistance. These cyclic resistances are higher even than the specimen prepared with 37 percent kaolinite, when cyclic resistance is measured in terms of initial liquefaction.

While this lack of correlation between clay content and cyclic resistance is also in agreement with the findings of Ishihara and Koeseki (1989), it is in direct contrast with

the field studies. Several authors have reported that during earthquakes, soils with more than 10 percent clay do not liquefy, clearly however from Figure 5-3, it may be seen that sands with more than 30 percent clay would appear to be liquefiable under relatively low cyclic stress ratios.

5.3.1.3 The Effect Of Liquid Limit

Unlike the effect of fines content and clay content, there appears to be some correlation between liquid limit and liquefaction resistance. As may be seen in Figure 5-4, cyclic resistance decreases as liquid limit increases until a liquid limit of approximately 17 percent is reached. For specimens with liquid limits greater than 17 percent there is a trend of increasing cyclic resistance with increasing liquid limit.

5.3.1.4 The Effect Of Plasticity Index

Based on the current study there is a correlation between the plasticity index of a sand and its cyclic resistance. As may be seen from Figure 5-5, there is a general trend of increasing cyclic resistance with increasing plasticity index.

This correlation between plasticity index and cyclic resistance agrees with the findings of Ishihara and Koeseki (1989), who found a strong agreement between these items.

5.3.1.5 The Effect Of Activity

Activity, which is the ratio of the plasticity index of a soil to its clay content, is a factor that accounts for both the quantity of clay present and the plasticity of that clay. In a sense it represents both the quantity and quality of the clay present.

For soils possessing a defined activity, there is a trend of increasing cyclic resistance with increasing activity. This trend is nearly linear as may be seen in Figure 5-6. This trend of increasing cyclic resistance with increasing activity is likely controlled by the plasticity index of the soil, as cyclic resistance has previously been shown to be relatively insensitive to clay content.

5.3.1.6 The Effect Of Void Ratio

Based upon the current study there is little correlation between the void ratio of a clayey sand and its cyclic resistance. As may be seen from Figure 5-7 there is little correlation between the void ratio of the soil and the cyclic resistance of the soil. This is because, as seen for soils with non-plastic fines, the cyclic resistance is controlled by the relative density of the soil rather than by its void ratio. The general clustering of the data points results from the fact that although at different void ratios, they are all at the same soil specific relative density. The scatter within each fines content would, however, seem to indicate some effect of the changing composition of the fines.

5.3.1.7 The Effect Of Water Content

As may be seen from Figure 5-8, the relationship between water content and cyclic resistance is very similar to that found between void ratio and cyclic resistance. This similarity is logical because, for soils with a limited range of specific gravities, the water content is nearly directly proportional to the void ratio.

5.3.1.8 The Effect of Liquidity Index

Liquidity index is a parameter used to express the water content of a soil relative to its Atterberg limits. It is the ratio of the soil's natural water content minus the plastic limit to the plasticity index of the soil. Liquidity index is analogous to relative density, with soils having liquidity indexes close to unity being near the liquid limit and those with liquidity indexes close to zero being near the plastic limit

From the current study, there appears to be a trend of decreasing cyclic resistance with increasing liquidity index, as may be seen in Figure 5-9. However, due to the relative small database, it is difficult to draw any firm conclusions.

5.3.2 Results Of Tests Performed At A Constant Fines Content

In order to isolate the effects of fines plasticity on the cyclic resistance of sandy soils, a series of tests were performed on specimens formed from soils with 17 percent fines. The composition of the fines was varied to allow for the evaluation of the effects of changing soil plasticity. These effects were again evaluated using common soil parameters.

5.3.2.1 The Effect Of Clay Content

For soils with 17 percent fine-grained material, no clear correlation may be drawn regarding the effect of clay content on liquefaction resistance. As may be seen in Figure 5-10, there is a weak trend of decreasing cyclic resistance with increasing clay contents. Although the scatter make it difficult to draw any firm conclusions, all of the soils tested with 17 percent fines and clay fractions between 8.5 and 17 percent had lower cyclic resistances than the soil with non-plastic fines at the same fines content and soil specific relative density.

5.3.2.2 The Effect Of Liquid Limit

For soils with 17 percent fine-grained material, a correlation may be drawn between the liquid limit of a soil and its cyclic resistance. As may be seen in Figure 5-11, cyclic resistance first decreases as liquid limit increases until a liquid limit of approximately 17 percent is reached. For soils with liquid limits greater than 17 percent cyclic resistance increases with increasing liquid limit. It should be noted that the soil with fines of 17 percent composed of half bentonite and half kaolinite, despite having a liquid limit of 41, still has a cyclic resistance of only approximately three-quarters that of the soil with 17 percent non-plastic silt.

5.3.2.3 The Effect Of Plasticity Index

While the small quantity of data make it difficult to draw firm conclusions about the effect of plasticity index on cyclic resistance, it may be seen in Figure 5-12 that increased plasticity index appears to result in an increase in cyclic resistance.

5.3.2.4 The Effect Of Activity

While the small quantity of data make it difficult to draw firm conclusions, increased activity appears to result in increased cyclic resistances for soils prepared to a constant fines content, as may be seen in Figure 5-13.

5.3.2.5 The Effect Of Water Content

There is little correlation between the water content of a soil and its cyclic resistance, as may be seen from Figure 5-14. For specimens with a nearly constant water content of approximately 24 percent, the cyclic resistance varies from a low of 0.11 to a high of 0.18, an increase of approximately 50 percent.

5.3.2.6 The Effect Of Liquidity Index

While due to a lack of data points it is impossible to draw firm conclusions, an increase in liquidity index does not appear to appreciably alter the cyclic resistance of a sandy soil prepared to a constant fines content. This may be seen in Figure 5-15.

5.4 Plasticity Based Liquefaction Criteria

Since the early 1970's, building codes in the People's Republic of China have included a listing of "thresholds to liquefaction" used to separate soils which are considered liquefiable from those considered non-liquefiable (Jennings, 1980). This criteria, commonly referred to as the Chinese criteria, is based on the observed behavior of soils during several major earthquakes in the P.R.C. Two of the key focuses of the criteria are the percentage of "clay" (smaller than 0.005 mm) present and the plasticity index of the

soil. A summary of the thresholds to liquefaction as presented by Jennings (1980) is presented in Table 5-1.

Based upon further field experiences and differences in testing methodologies, several modifications have been proposed to the Chinese criteria (Seed et al., 1973; Finn, Ledbetter, Wu, 1994; and Koester, 1994). A summary of these proposed modifications is presented in Table 5-2.

In order to evaluate the applicability of these criteria, and to investigate the mechanisms governing the behavior of these soils, the tests performed using Yatesville sand and the various combinations of plastic fines were evaluated in terms of each of these criteria. The applicable factors in each criteria were compared to the factors for the specimens tested. If all of the criteria showed the specimen to be liquefiable, it was considered to be liquefiable. If one or more of the criteria showed the specimen to be non-liquefiable, the specimen was declared non-liquefiable. In the laboratory, all of the specimens tested were found to be liquefiable in a reasonable numbers of cycles, at cyclic stress ratios likely to occur in the field. Therefore, any of the criteria predicting a specimen to be non-liquefiable would appear to be incorrect.

To further evaluate the inaccuracies in the criteria, the type of liquefaction that occurred in the lab, whether flow liquefaction or cyclic mobility, was considered. It is assumed that if cyclic mobility did develop in the field, the limited strains produced would do little damage and produce little evidence of occurrence. In contrast to flow liquefaction, the consequences of cyclic mobility may be considered minor enough to treat it as a non-liquefaction scenario for most design cases.

The results of the evaluations of the various criteria are summarized in Tables 5-3 through 5-6, and are discussed below.

5.4.1 The Chinese Criteria

Soils meeting these criteria are considered to be non-liquefiable and include those with plasticity indexes greater than 10, clay contents greater than 10 percent, mean grain sizes between 0.2 and 1 millimeters, relative densities greater than 75 percent, and void ratios less than 0.80.

Other criteria presented in the Chinese criteria include epicentral distance, intensity, the depth of the sand layer, and depth of the water table. As these are not properties inherent to the soil, they were not evaluated.

Of the fourteen soils tested, twelve would be considered non-liquefiable based upon the Chinese criteria. Of these twelve, seven were found to be susceptible to flow liquefaction. Thus the Chinese criteria would appears to be a poor predictor of liquefaction susceptibility. The details of the evaluation of this criteria are summarized in Table 5-3 and discussed below.

5.4.1.1 Plasticity Index

The requirement for plasticity indexes greater than 10 percent to be considered non-liquefiable under the Chinese criteria, appears reasonable. For the two soils tested which met this requirement, both underwent cyclic mobility failures. In fact, all of the soils with plasticity indexes of seven or greater were found to be susceptible to cyclic mobility rather than flow failures, although the soil containing 26 percent kaolinite, which has a plasticity index of 7, appears to be a borderline case.

5.4.1.2 Clay Content

The requirement for clay contents greater than 10 percent to be considered non-liquefiable under the Chinese criteria, does not appear reasonable. As discussed earlier in this chapter, there appears to be little correlation between liquefaction susceptibility and

clay content. Of the fourteen soils tested, ten would be considered non-liquefiable based upon this requirement. Of these ten, five were found to be susceptible to flow liquefaction.

5.4.1.3 Mean Grain Size

The requirement that requires that the mean grain size of the soil be between 0.2 and 1 millimeters in order to be liquefiable does not appear reasonable. All fourteen of the soils tested met this requirement, and thus would be considered liquefiable under the Chinese criteria. Of these, however, only nine were found to be susceptible to flow liquefaction.

5.4.1.4 Relative Density

All of the soils tested met this criteria, which requires that the relative density of the soil be less than 75 percent in order to be liquefiable. Of the fourteen soils tested, all would be considered liquefiable based upon the Chinese criteria. Of these, five were found to be susceptible to cyclic mobility.

5.4.1.5 Void Ratio

The requirement that soils with void ratios smaller than 0.8 may be considered non-liquefiable under the Chinese criteria does not appear valid. As discussed earlier in this chapter, there appears to be little correlation between liquefaction susceptibility and void ratio. Of the fourteen soils tested, twelve would be considered non-liquefiable based upon this requirement. Of these twelve, seven were found to be susceptible to flow liquefaction.

5.4.2 Seed et al.'s Criteria

Soils meeting the criteria suggested by Seed et al. (1983) include those with clay contents greater than 15 percent, liquid limits greater than 35 percent, and water contents less than 90 percent of their liquid limit. The details of the evaluation of this criteria are summarized in Table 5-4 and discussed below.

5.4.2.1 Clay Content

The requirement for soils with clay contents greater than 15 percent to be considered non-liquefiable under this criteria, does not appear infallible. Of the fourteen soils tested, five would be considered non-liquefiable based upon this requirement. Of these five, one was found to be susceptible to flow liquefaction, while another was a borderline case.

5.4.2.2 Liquid Limit

The requirement that a soil possess a liquid limit greater than 35 percent in order to be considered non-liquefiable under this criteria, appears conservative. For the two soils tested which met this requirement, both underwent cyclic mobility failures. In fact, all of the soils with liquid limit of 20 or greater were found to be susceptible to cyclic mobility, rather than flow failures. The soil prepared with 26 percent kaolinite, which has a liquid limit of 20, appears to be a borderline case.

5.4.2.3 Water Content

The criteria that soils with water contents less than 90 percent of their liquid limits may be considered non-liquefiable, appears to be valid. Of the fourteen soils tested, three would be considered non-liquefiable based upon this requirement. All three of these soils were found to be susceptible to cyclic mobility.

5.4.3 Finn, Ledbetter, and Wu's Criteria

Finn, Ledbetter, and Wu (1994) recommend that changes to be made to the Chinese criteria to account for uncertainty and differences in liquid limit determination between the ASTM and the Chinese standard. They recommend decreasing the fines content to 10 percent, the liquid limit to 34 percent, and the water content to 2 percent less than 90 percent of the liquid limit. The details of the evaluation of this criteria are summarized in Table 5-5 and discussed below.

5.4.3.1 Clay Content

As discussed for the Chinese criteria, the requirement that clay contents greater than 10 percent be considered non-liquefiable, does not appear reasonable. This is because there appears to be little correlation between liquefaction susceptibility and clay content. Of the fourteen soils tested, ten would be considered non-liquefiable based upon this requirement. Of these ten, five were found to be susceptible to flow liquefaction.

5.4.3.2 Liquid Limit

Similar to the findings for Seed et al.'s criteria, the requirement that a soil possess a liquid limit greater than 34 percent in order to be considered non-liquefiable appears conservative. All of the soils with liquid limits of 20 or greater were found to be susceptible to cyclic mobility, rather than flow liquefaction.

5.4.3.3 Water Content

The requirement that soils with water contents less than 2 percent below 90 percent of their liquid limits may be considered non-liquefiable, appears to be valid. Of the fourteen soils tested, three would be considered non-liquefiable based upon this requirement. All three of these soils were found to be susceptible to cyclic mobility.

5.4.4 Koester's Criteria

Koester (1994) recommend that a further change be made to Finn, Ledbetter, and Wu's criteria to account for differences in the determination of liquid limit between the ASTM and the Chinese standard. He recommended increasing the liquid limit to 36 percent. The details of the evaluation of this criteria are summarized in Table 5-6 and discussed below.

As has been shown for both Seed's and to Finn, Ledbetter, and Wu's criteria, the liquid limit criteria appears to be conservative regardless of whether it is 34, 35, or 36 percent, therefore this change would not appear to be of any major significance.

5.4.5 Implications

Three major implications may be made from this study of the effects of fines content and plasticity on the liquefaction resistance of sandy soils. The first is that the addition of plastic fines does not inherently increase the cyclic resistance of a sand. The second is that the most viable component of the plasticity based liquefaction criteria is the criteria involving the plasticity of the soil. Lastly, it may not be proper to evaluate the liquefaction susceptibility of soils with significant amounts of plastic fines using the commonly used definitions of liquefaction. Each of these implications is discussed further below.

5.4.5.1 Clayey Sands With Low Plasticity

Loose sandy soils which contain plastic fines but whose plasticity does not meet the requirements of the plasticity based liquefaction criteria (i.e. LL > 35 or PI >10) may have cyclic resistances much lower than soils with a comparable percentage of non-plastic (i.e. silty) fines. The addition of even a few percent of plastic fines mixed in with the silty fine-grained material may cause the cyclic resistance to decrease by almost two-thirds where cyclic resistance is measured as the cyclic stress ratio required to cause liquefaction in a given number of cycles. Thus the introduction of plastic fines may in fact increase the susceptibility of a sand to liquefaction.

These soils are subject to flow liquefaction. Under cyclic loading, these soils initially show very little straining, however as the pore pressure ratio approaches 100 percent (i.e. the effective stress approaches zero) these soils will undergo sudden, almost spontaneous, deformations which may be very large in magnitude.

5.4.5.2 Plasticity Criteria

Several sets of criteria have been proposed for separating liquefiable from non-liquefiable sands based upon the clay content, soil plasticity, and density of the soil. A review of

these criteria has shown that the one parameter which consistently separates soils susceptible to flow liquefaction from soils that tend to undergo cyclic mobility is the soil plasticity. Whether measured in terms of plasticity index or liquid limit, soils that meet some threshold level of plasticity tend to be safe from flow liquefaction failures.

While a threshold plasticity index of 10 seems to be appropriately conservative for separating soils susceptible to flow liquefaction from soils which tend to undergo cyclic mobility, the proposed threshold value of 35 for liquid limit may be overly conservative. Although more study is clearly necessary due to the limited size of the database, this investigation found that soils with liquid limits above 20 were not susceptible to flow liquefaction. This may be seen in Figure 5-16, which shows the separation that occurs between soils susceptible to flow liquefaction and soils susceptible to cyclic mobility when they are plotted in terms of their Atterberg limits.

Based upon Figure 5-16, a proposed zone of liquefiable soils is indicated on the plasticity chart shown in Figure 5-17, and includes soils with plasticity indexes less than 10 and liquid limits less than 35. Although the findings of this study would appear to indicate that soils with liquid limits greater than 20 percent are safe from flow liquefaction, the upper bound of the zone was set at a liquid limit of 35 percent for the sake of conservatism. Soils that plot in this region and contain plastic fines should be tested in the laboratory to determine their liquefaction susceptibility.

5.4.5.3 Evaluation Of Liquefaction

If evaluated using the commonly used definitions of liquefaction such as initial liquefaction or induced strain, soils containing plastic fines which meet the plasticity requirement appear to have cyclic resistances lower than soils with comparable percentages of non-plastic fines. However, this assessment is not necessarily meaningful, as these soils are not subject to flow liquefaction.

Under cyclic loading, soils meeting the plasticity requirement generate pore pressures, and thus decreases in effective stress, to the same extent as the soils not meeting the plasticity requirement, however their deformation characteristics are very different. Whereas soils not meeting the plasticity requirement are subject to spontaneous, very large deformations at large pore pressure ratios, soils meeting the plasticity requirement tend to deform in a gradual manner, straining in both compression and tension as the pore pressure ratio increases. They never undergo a rapid, large strain, flow condition and in fact do not undergo any deformation after the cessation of cyclic loading. This behavior is akin to what people often call "cyclic mobility".

For these reasons, it does not appear appropriate to define liquefaction for soils meeting the plasticity criteria in terms of zero effective stress or some level of strain measured in a lab test. Additionally, this behavior explains why field investigations have routinely reported these soils do not liquefy during earthquakes. While they may have generated pore pressures equal to their confining stresses (i.e. undergone initial liquefaction), because they did not undergo significant deformations or eject sand boils, they left no evidence of having liquefied.

5.5 A Behavioral Hypothesis

The decrease in cyclic resistance that occurs when small amounts of clay are added to a sand may have an explanation on a mechanical basis. The decrease in liquefaction resistance may result from the increased ease of movement among the sand grains brought on by the presence of the slicker clay particles offsetting the increased resistance to movement caused by the plasticity of the clay.

When small amounts of clay are added to the sand the clay particles may either adhere to the surface of the sand particles or be located between the normal sand grain to sand grain points of contact. Because the clay particles typically have lower friction angles than the sand, the presence of these particles may make it easier for the sand grains to slide past

each other, thus requiring less work to liquefy the soil. This increased ease of movement may not be compensated for until the soil has enough plasticity to prevent the sand grains from sliding. This would explain why plasticity based criteria such as the Modified Chinese criteria require relatively high levels of plasticity.

5.5 Conclusions

Several conclusions regarding the effects of plastic fines on the liquefaction of sandy soils and the use of plasticity based liquefaction criteria may be drawn from this study.

The addition of even a moderate amount of plastic fines (up to 25 percent) may decrease the liquefaction resistance of the soil if the fines are of low plasticity. The behavior of sands with plastic fines appears to be relatively independent of fines content, clay content, water content, and liquidity index. It is however dependent on the plasticity of the soil (not just the plasticity of the fines) whether that plasticity is measured in terms of liquid limit, plasticity index, or activity.

The key factor in the plasticity-based criteria is the requirement for the plasticity of the soil. If this requirement, whether measured in terms of liquid limit or plasticity index, is met, the soil does not appear to be susceptible to flow liquefaction. A plasticity index of 10 appears to be a reasonable threshold value, while the liquid limit values in the range of 34 to 36 which have been suggested, appear rather conservative.

The reason that the plasticity based criteria work is that they mirror a change in behavior in the soil following the onset of initial liquefaction. Soils meeting the criteria tend to undergo a cyclic mobility form of failure which results only in very small deformations of the soils mass, while those not meeting the criteria tend to undergo flow liquefaction which results in very large deformations.

This difference in behavior makes it inappropriate to evaluate liquefaction for soils meeting the plasticity requirements using either initial liquefaction or some level of strain measured in a lab test. Although these behaviors occur, the deformations are essentially elastic and the soil does not behave in any of the manners commonly associated with liquefaction, for example steady-state deformation or the emission of sand boils. Additionally, this behavior may explain why these soils have frequently been reported as being non-liquefiable during field studies. While these soils may have developed a condition of zero effective stress, their lack of deformation and the absence of sand boils tend to mask the occurrence of liquefaction.

Table 5-1:Thresholds to liquefaction (after Jennings, 1980)

Condition	Threshold
Maximum epicentral distance (km)	Log Dmax = 0.87M - 4.5
Minimum Intensity (Chinese scale)	6
Mean grain size (mm)	0.02 < D50 < 1.0
Clay particle content (percent)	10 <
Uniformity coefficient	10 <
Relative density (percent)	75 <
Void ratio	> 0.8
Plasticity index Ip	< 10
Depth to water table (m)	< 5
Depth to sand layer (m)	< 20

Table 5-2: Proposed modifications to the Chinese criteria

Author	Proposed Criteria/Modifications
Seed et al. (1983)	Percent finer than 0.005 mm < 15% Liquid Limit, LL < 35% Natural water content at least 90% of LL
Finn, Ledbetter, and Wu (1994)	Decrease fines content by 5% Decrease liquid limit by 2% Increase water content by 2%
Koester (1994)	Decrease fines content by 5% Increase liquid limit by 1% Decrease water content by 2%

Table 5-3: Evaluation of the Chinese criteria

Table 5-3: Evaluation of the Chinese Criteria

					Does The Soil Meet The						
Fines Content (%)	Fines Type	Clay Content (%)	PI (%)	Void Ratio	D50 Criteria (0.02 mm<D50) (D50<1.0 mm)	"Clay" Content Criteria (0.005 mm<10%)	PI Criteria (<10%)	Void Ratio Critera (>0.80)	Relative Density Critera (<75%)	Is The Soil Susceptible To Liquefaction?	Liquefaction Mode
4	M & K	2.0	0	0.80	Yes	Yes	Yes	Yes	Yes	Yes	FL
7	M & K	3.5	0	0.75	Yes	Yes	Yes	No	Yes	No	FL
12	M & K	6.0	0	0.72	Yes	No	Yes	No	Yes	No	FL
17	M & K	8.5	0	0.66	Yes	No	Yes	No	Yes	No	FL
26	M & K	13.0	1	0.58	Yes	No	Yes	No	Yes	No	FL
4	K	4.0	0	0.81	Yes	Yes	Yes	Yes	Yes	Yes	FL
7	K	7.0	0	0.75	Yes	Yes	Yes	No	Yes	No	FL
12	K	12.0	0	0.72	Yes	No	Yes	No	Yes	No	FL
17	K	17.0	0	0.66	Yes	No	Yes	No	Yes	No	FL
26	K	26.0	7	0.61	Yes	No	Yes	No	Yes	No	FL/CM
37	K	37.0	8	0.59	Yes	No	Yes	No	Yes	No	CM
17	K/B	17.0	19	0.67	Yes	No	No	No	Yes	No	CM
17	M/K/B	11.3	7	0.67	Yes	No	Yes	No	Yes	No	CM
12	B	12.0	20	0.74	Yes	No	No	No	Yes	No	CM

M = Yatesville Silt
K = Kaolinite
B = Bentonite
FL = Flow Liquefaction
CM = Cyclic Mobility

Table 5-4: Evaluation of the Seed et al’s criteria

Table 5-4: Evaluation of Seed et al.'s criteria

Fines Content (%)	Fines Type	Clay Content (%)	LL (%)	PI (%)	Void Ratio	Water Content (%)	0.9LL	Does The Soil Meet The "Clay" Content Criteria (0.005 mm<15%)	LL Criteria (<35%)	Water Content Critera (>0.9 LL)	Is The Soil Susceptible To Liquefaction?	Liquefaction Mode
4	M & K	2.0	20	0	0.80	29.3	18.0	Yes	Yes	Yes	Yes	FL
7	M & K	3.5	19	0	0.75	27.6	17.1	Yes	Yes	Yes	Yes	FL
12	M & K	6.0	17	0	0.72	26.5	15.3	Yes	Yes	Yes	Yes	FL
17	M & K	8.5	15	0	0.66	24.5	13.5	Yes	Yes	Yes	No	FL
26	M & K	13.0	15	1	0.58	21.5	13.5	Yes	Yes	Yes	No	FL
4	K	4.0	17	0	0.81	29.7	15.3	Yes	Yes	Yes	Yes	FL
7	K	7.0	19	0	0.75	27.6	17.1	Yes	Yes	Yes	Yes	FL
12	K	12.0	17	0	0.72	26.5	15.3	Yes	Yes	Yes	Yes	FL
17	K	17.0	18	0	0.66	24.4	16.2	No	Yes	Yes	No	FL
26	K	26.0	20	7	0.61	22.6	18.0	No	Yes	Yes	No	FL/CM
37	K	37.0	21	8	0.59	22.0	18.9	No	Yes	Yes	No	CM
17	K/B	17.0	41	19	0.67	24.8	36.9	No	No	No	No	CM
17	M/K/B	11.3	31	7	0.67	24.6	27.9	No	Yes	No	No	CM
12	B	12.0	48	20	0.74	27.4	43.2	Yes	No	No	No	CM

M = Yatesville Silt
K = Kaolinite
B = Bentonite
FL = Flow Liquefaction
CM = Cyclic Mobility

Table 5-5: Evaluation of the Finn et al's criteria

Table 5-5: Evaluation of Finn et al.'s criteria

Fines Content (%)	Fines Type	Clay Content (%)	LL (%)	PI (%)	Void Ratio	Water Content (%)	0.9LL+2	Does The Soil Meet The "Clay" Content Criteria (0.005 mm<10%)	Does The Soil Meet The LL Criteria (<33%)	Does The Soil Meet The Water Content Critera (>0.9 LL-2)	Is The Soil Susceptible To Liquefaction?	Liquefaction Mode
4	M & K	2.0	20	0	0.80	29.3	20.0	Yes	Yes	Yes	Yes	FL
7	M & K	3.5	19	0	0.75	27.6	19.1	Yes	Yes	Yes	Yes	FL
12	M & K	6.0	17	0	0.72	26.5	17.3	No	Yes	Yes	No	FL
17	M & K	8.5	15	0	0.66	24.5	15.5	No	Yes	Yes	No	FL
26	M & K	13.0	15	1	0.58	21.5	15.5	No	Yes	Yes	No	FL
4	K	4.0	17	0	0.81	29.7	17.3	Yes	Yes	Yes	Yes	FL
7	K	7.0	19	0	0.75	27.6	19.1	Yes	Yes	Yes	Yes	FL
12	K	12.0	17	0	0.72	26.5	17.3	No	Yes	Yes	No	FL
17	K	17.0	18	0	0.66	24.4	18.2	No	Yes	Yes	No	FL
26	K	26.0	20	7	0.61	22.6	20.0	No	Yes	Yes	No	FL/CM
37	K	37.0	21	8	0.59	22.0	20.9	No	Yes	Yes	No	CM
17	K/B	17.0	41	19	0.67	24.8	38.9	No	No	No	No	CM
17	M/K/B	11.3	31	7	0.67	24.6	29.9	No	Yes	No	No	CM
12	B	12.0	48	20	0.74	27.4	45.2	No	No	No	No	CM

M = Yatesville Silt
K = Kaolinite
B = Bentonite
FL = Flow Liquefaction
CM = Cyclic Mobility

Table 5-7: Evaluation of the Koester's criteria

Table 5-6: Evaluation of Koester's criteria

Fines Content (%)	Fines Type	Clay Content (%)	LL (%)	PI (%)	Void Ratio	Water Content (%)	0.9LL-2	Does The Soil Meet The "Clay" Content Criteria (0.005 mm<15%)	LL Criteria (<36%)	Water Content Critera (>0.9 LL-2)	Is The Soil Susceptible To Liquefaction?	Liquefaction Mode
4	M & K	2.0	20	0	0.80	29.3	16.0	Yes	Yes	Yes	Yes	FL
7	M & K	3.5	19	0	0.75	27.6	15.1	Yes	Yes	Yes	Yes	FL
12	M & K	6.0	17	0	0.72	26.5	13.3	No	Yes	Yes	No	FL
17	M & K	8.5	15	0	0.66	24.5	11.5	No	Yes	Yes	No	FL
26	M & K	13.0	15	1	0.58	21.5	11.5	No	Yes	Yes	No	FL
4	K	4.0	17	0	0.81	29.7	13.3	Yes	Yes	Yes	Yes	FL
7	K	7.0	19	0	0.75	27.6	15.1	Yes	Yes	Yes	Yes	FL
12	K	12.0	17	0	0.72	26.5	13.3	No	Yes	Yes	No	FL
17	K	17.0	18	0	0.66	24.4	14.2	No	Yes	Yes	No	FL
26	K	26.0	20	7	0.61	22.6	16.0	No	Yes	Yes	No	FL/CM
37	K	37.0	21	8	0.59	22.0	16.9	No	Yes	Yes	No	CM
17	K/B	17.0	41	19	0.67	24.8	34.9	No	No	No	No	CM
17	M/K/B	11.3	31	7	0.67	24.6	25.9	No	Yes	No	No	CM
12	B	12.0	48	20	0.74	27.4	41.2	No	No	No	No	CM

M = Yatesville Silt
K = Kaolinite
B = Bentonite
FL = Flow Liquefaction
CM = Cyclic Mobility

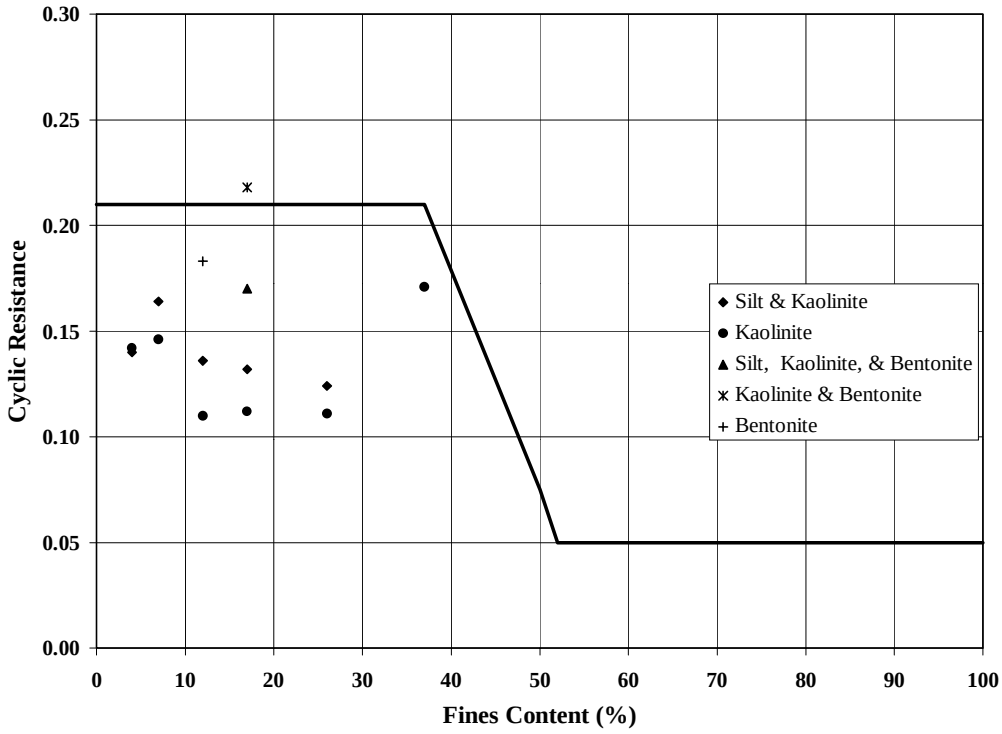

Figure 5-1: Variation in cyclic resistance with silty and clayey fines for specimens prepared to a constant soil specific relative density

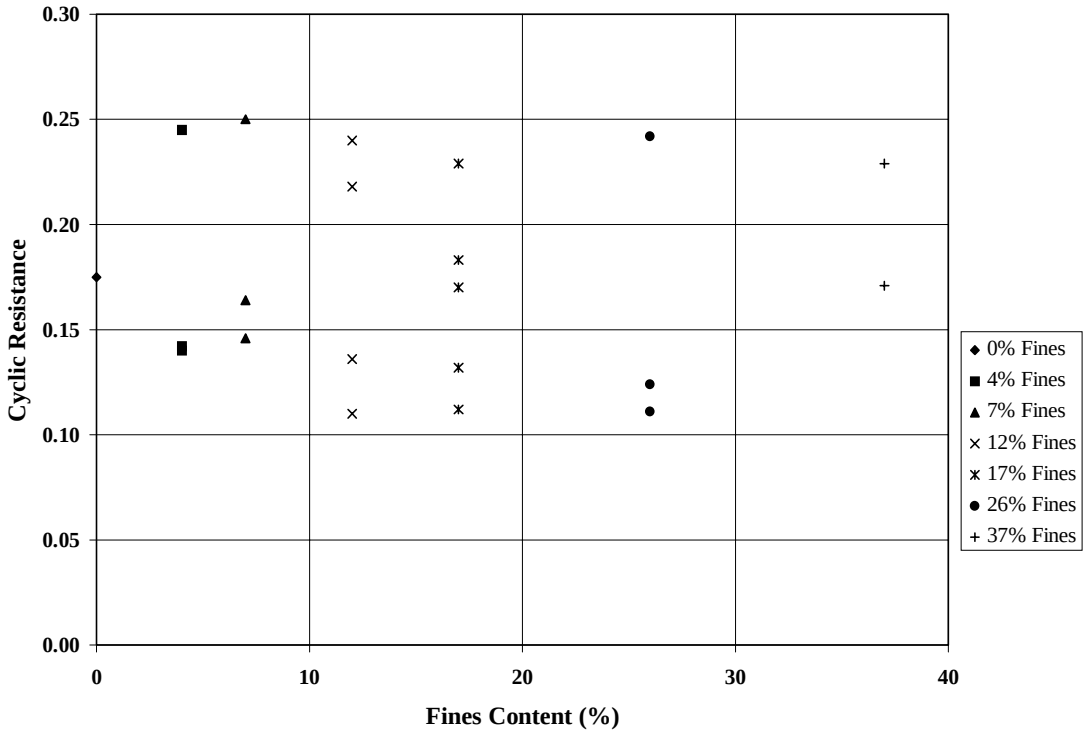

Figure 5-2: Variation in cyclic resistance with fines content for specimens prepared to a constant soil specific relative density

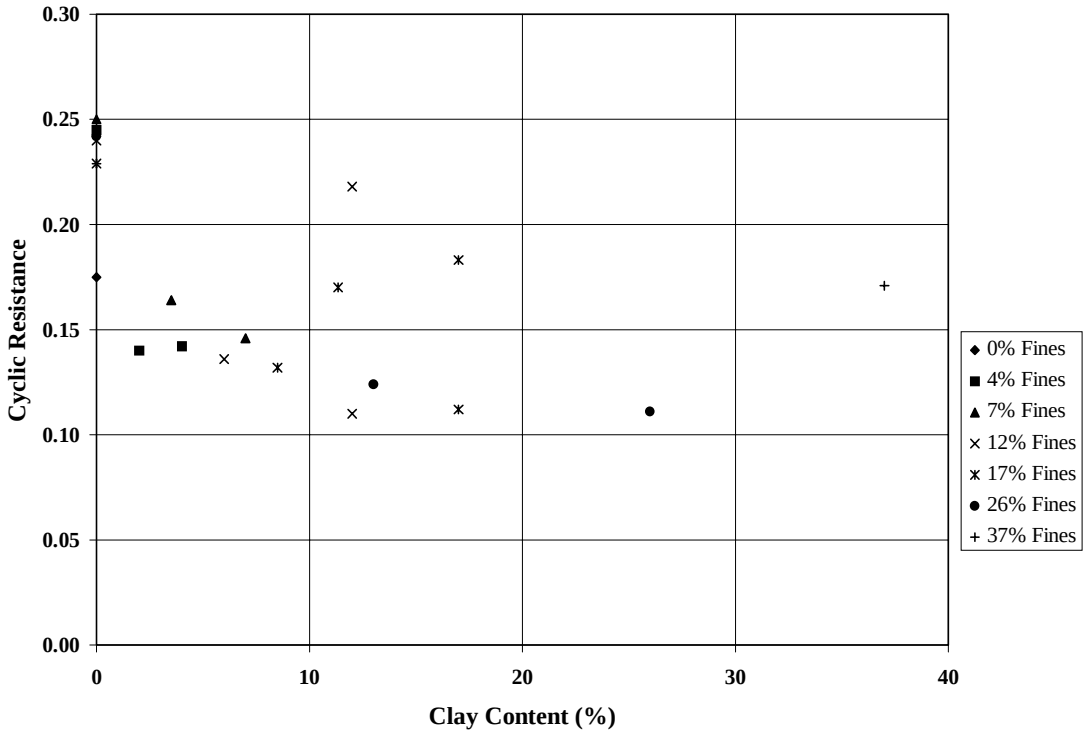

Figure 5-3: Variation in cyclic resistance with clay content for specimens prepared to a constant soil specific relative density

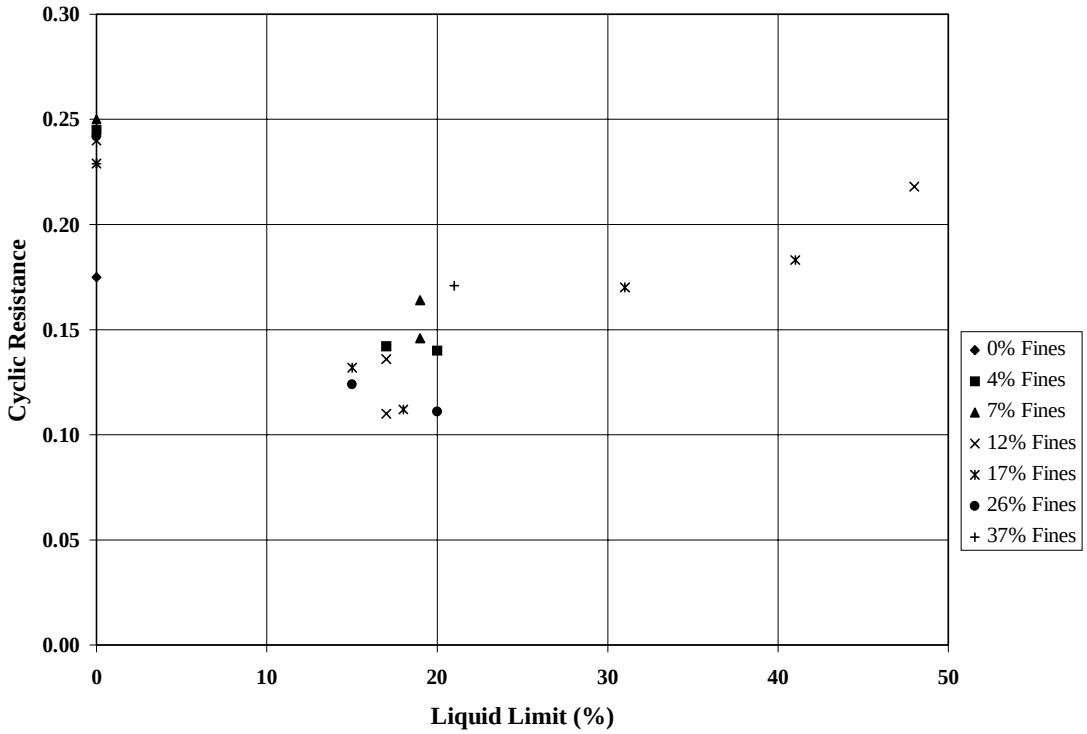

Figure 5-4: Variation in cyclic resistance with liquid limit for specimens prepared to a constant soil specific relative density

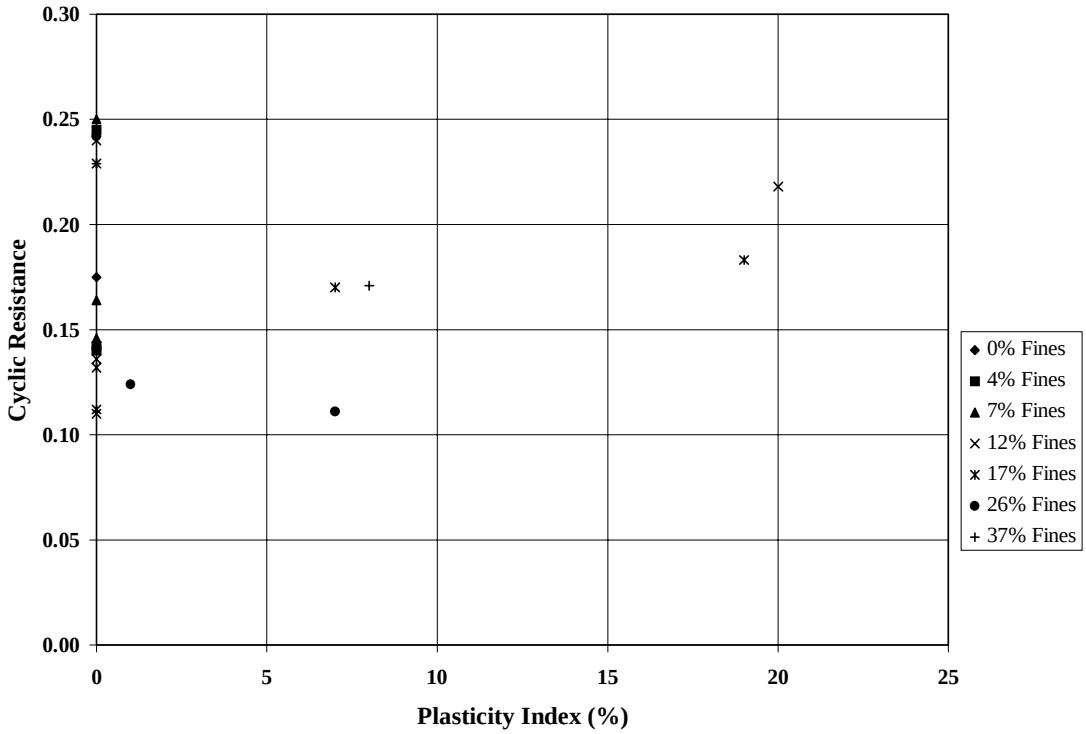

Figure 5-5: Variation in cyclic resistance with plasticity index for specimens prepared to a constant soil specific relative density

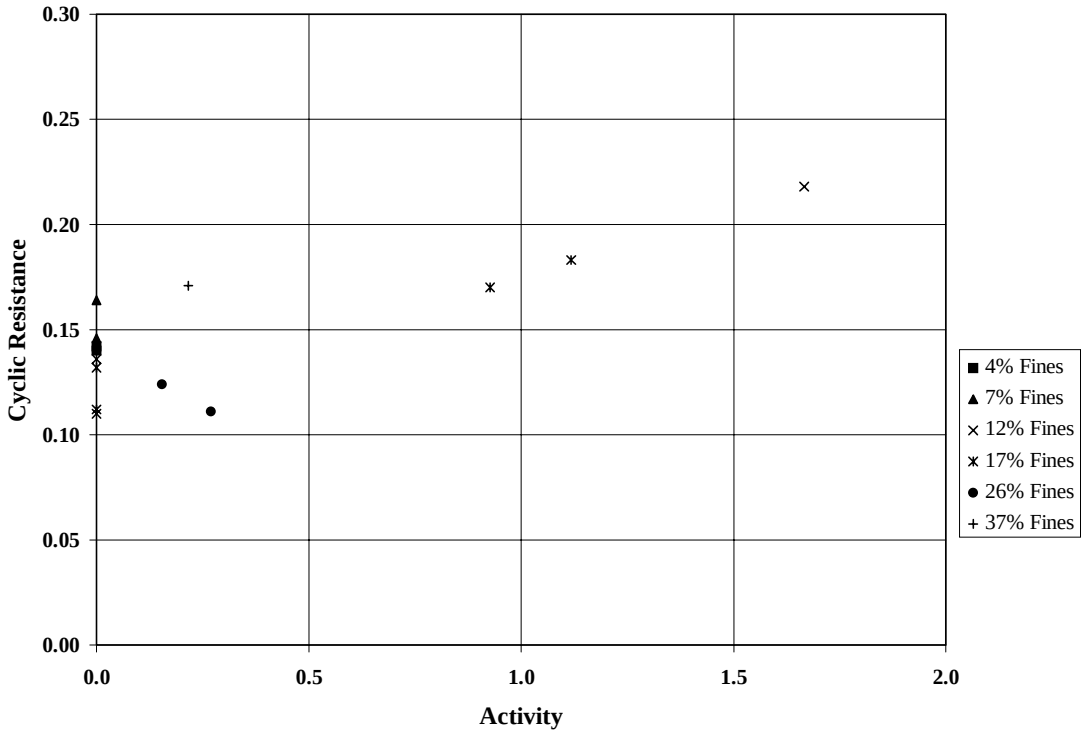

Figure 5-6: Variation in cyclic resistance with activity for specimens prepared to a constant soil specific relative density

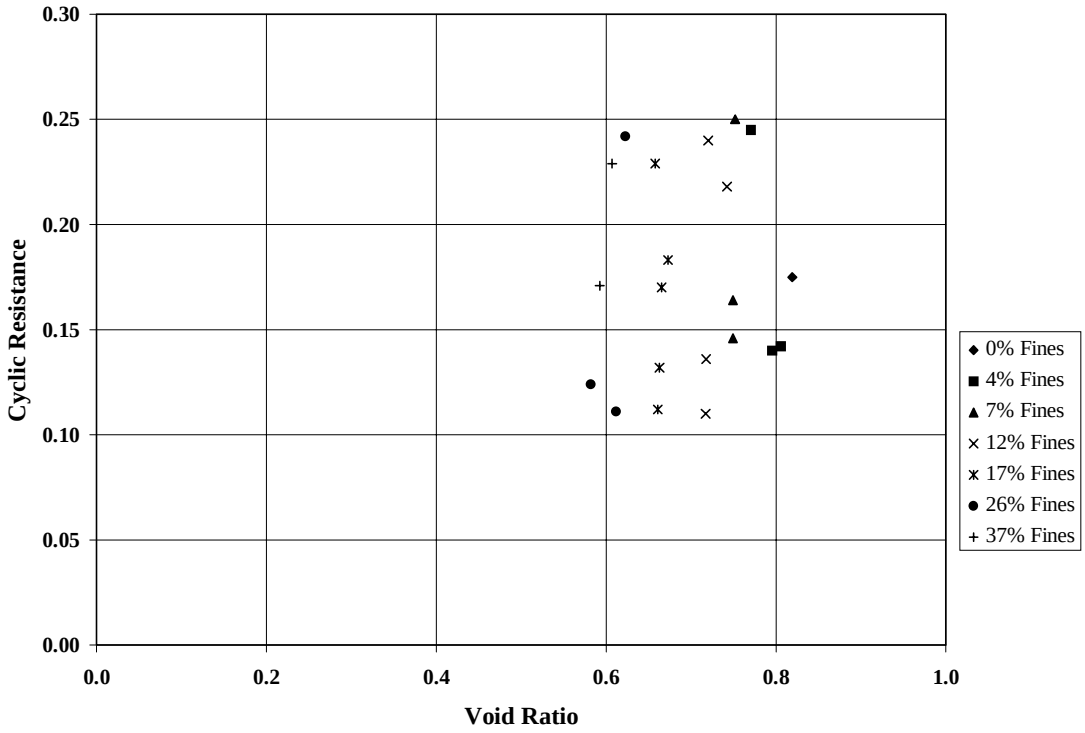

Figure 5-7: Variation in cyclic resistance with void ratio for specimens prepared to a constant soil specific relative density

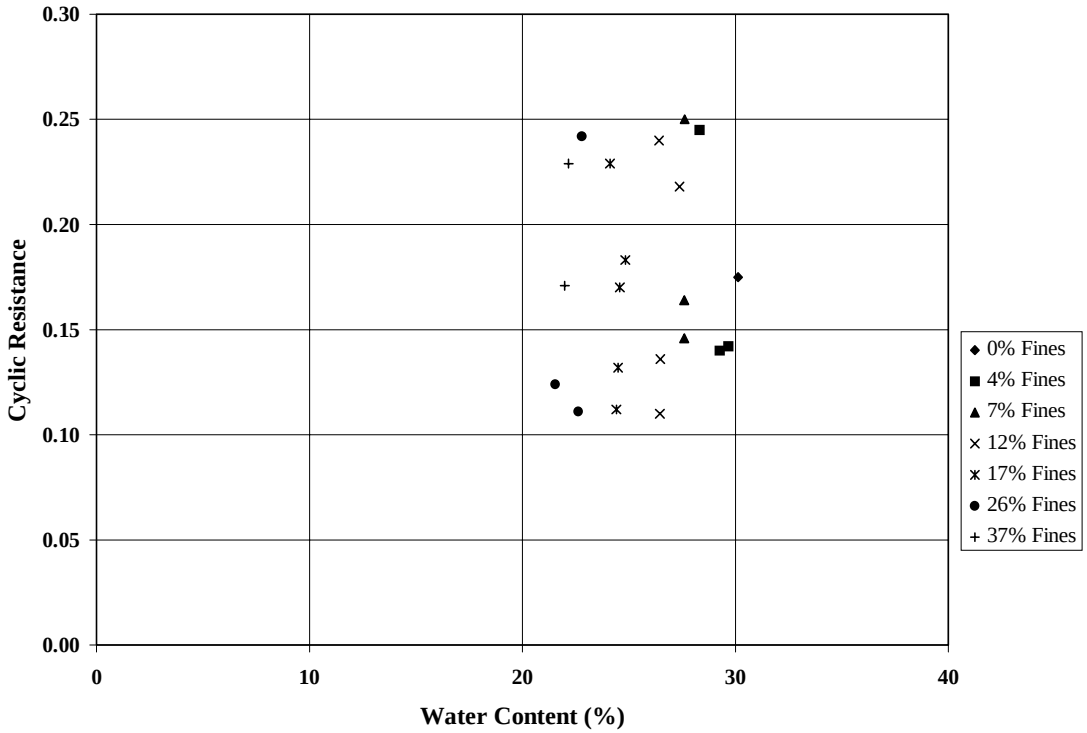

Figure 5-8: Variation in cyclic resistance with water content for specimens prepared to a constant soil specific relative density

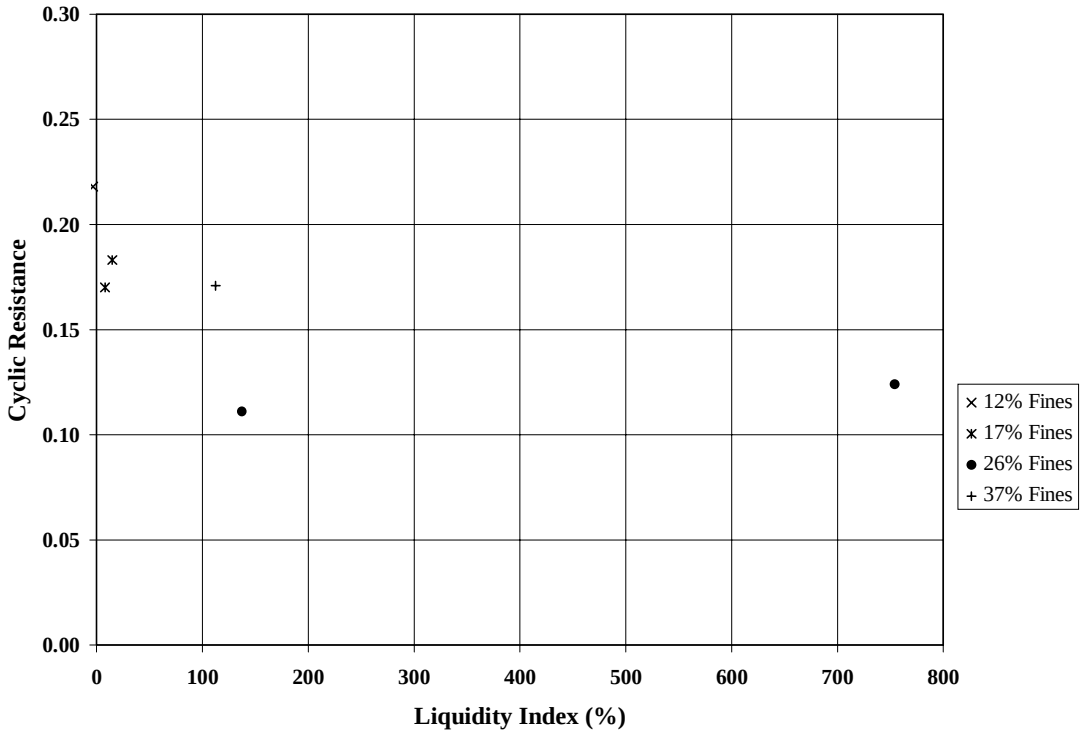

Figure 5-9: Variation in cyclic resistance with liquidity index for specimens prepared to a constant soil specific relative density

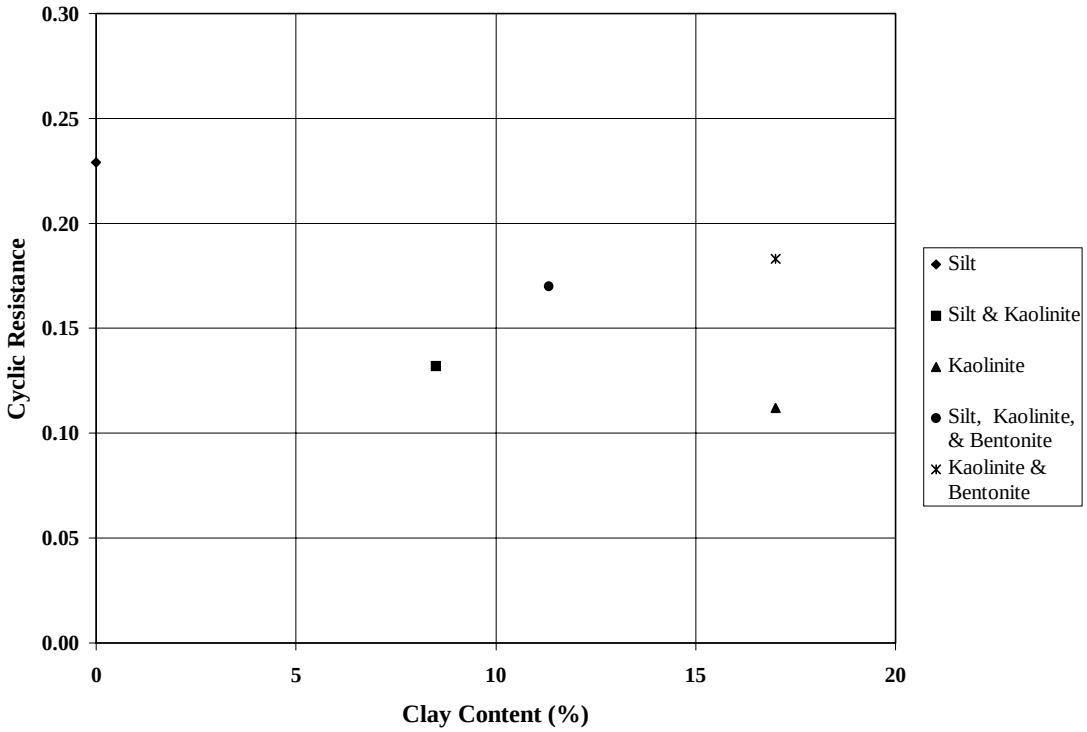

Figure 5-10: Variation in cyclic resistance with clay content for specimens prepared to a constant fines content

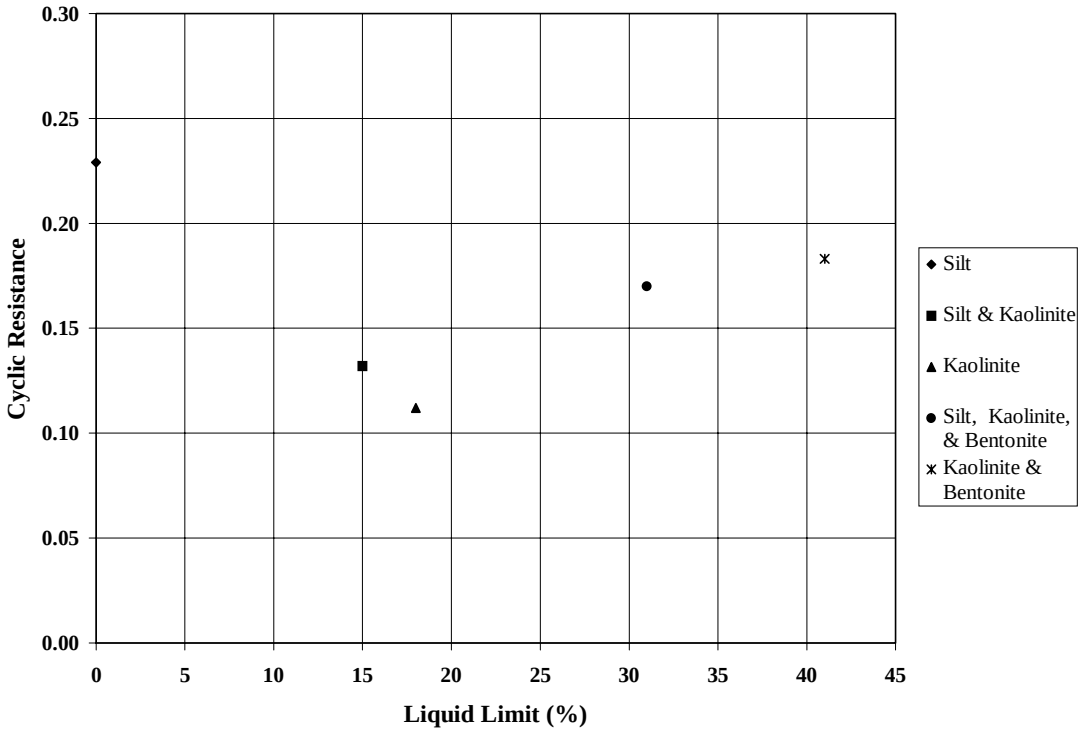

Figure 5-11: Variation in cyclic resistance with liquid limit for specimens prepared to a constant fines content

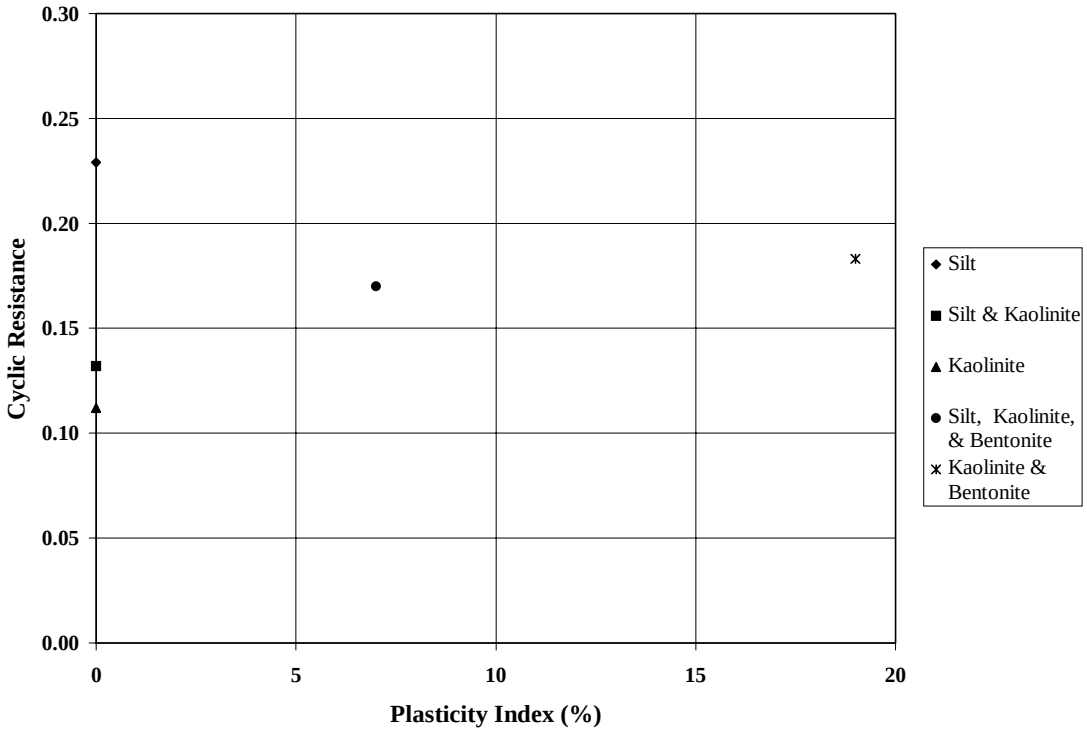

Figure 5-12: Variation in cyclic resistance with plasticity index for specimens prepared to a constant fines content

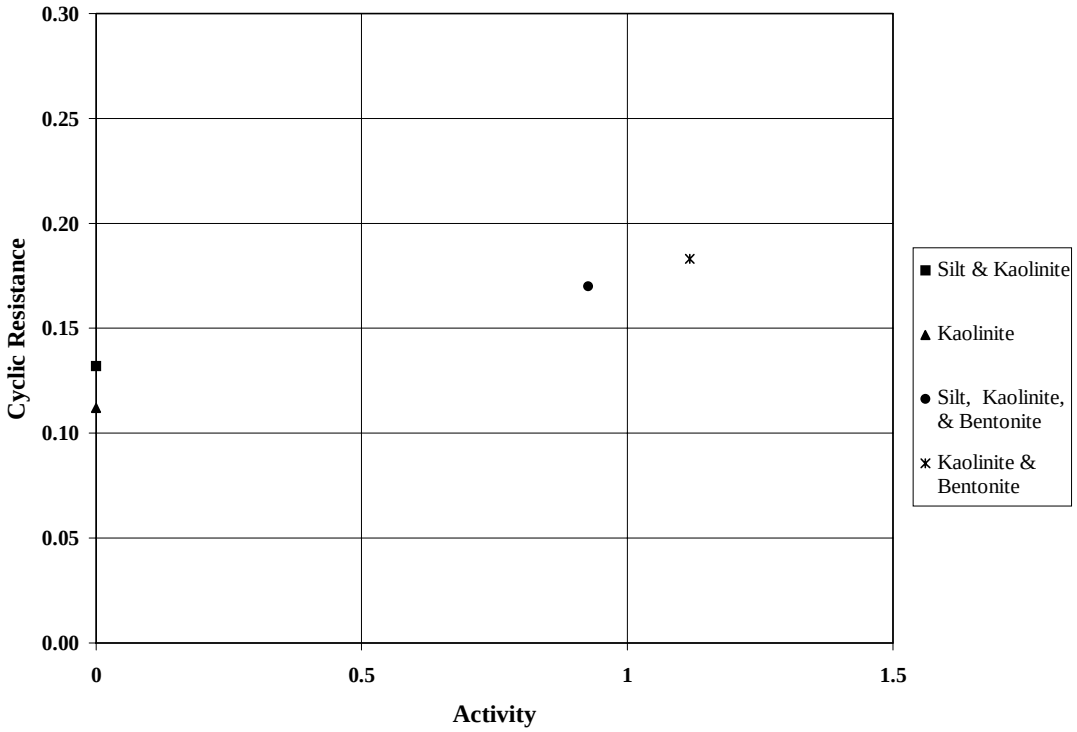

Figure 5-13: Variation in cyclic resistance with activity for specimens prepared to a constant fines content

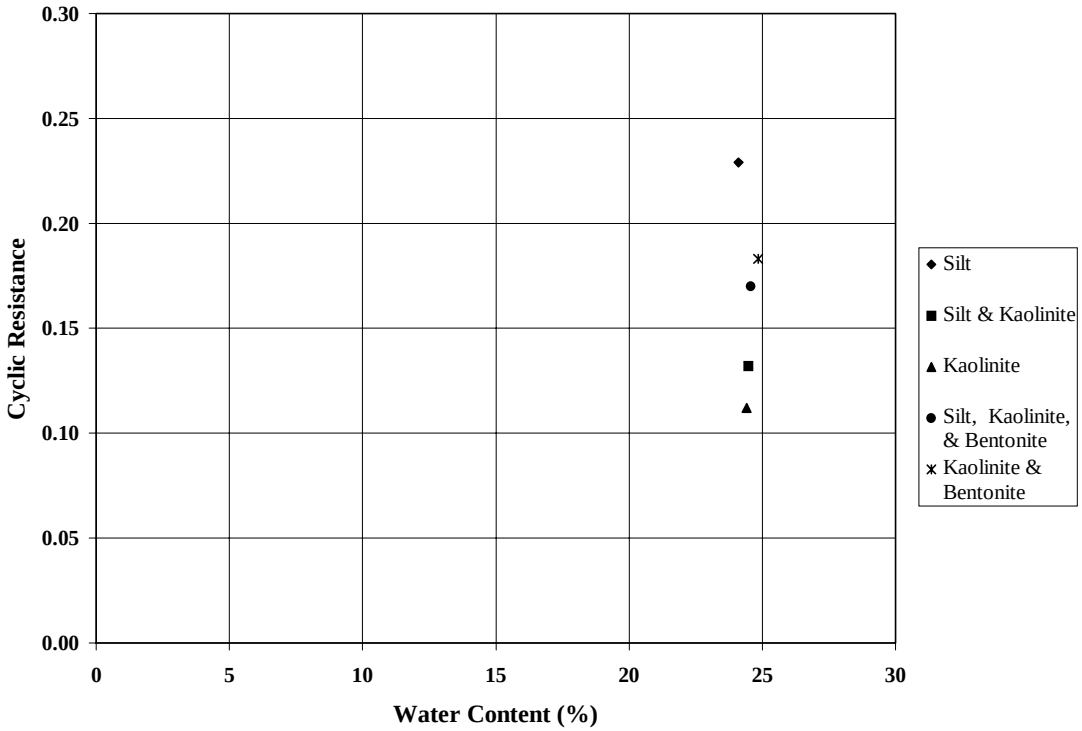

Figure 5-14: Variation in cyclic resistance with water content for specimens prepared to a constant fines content

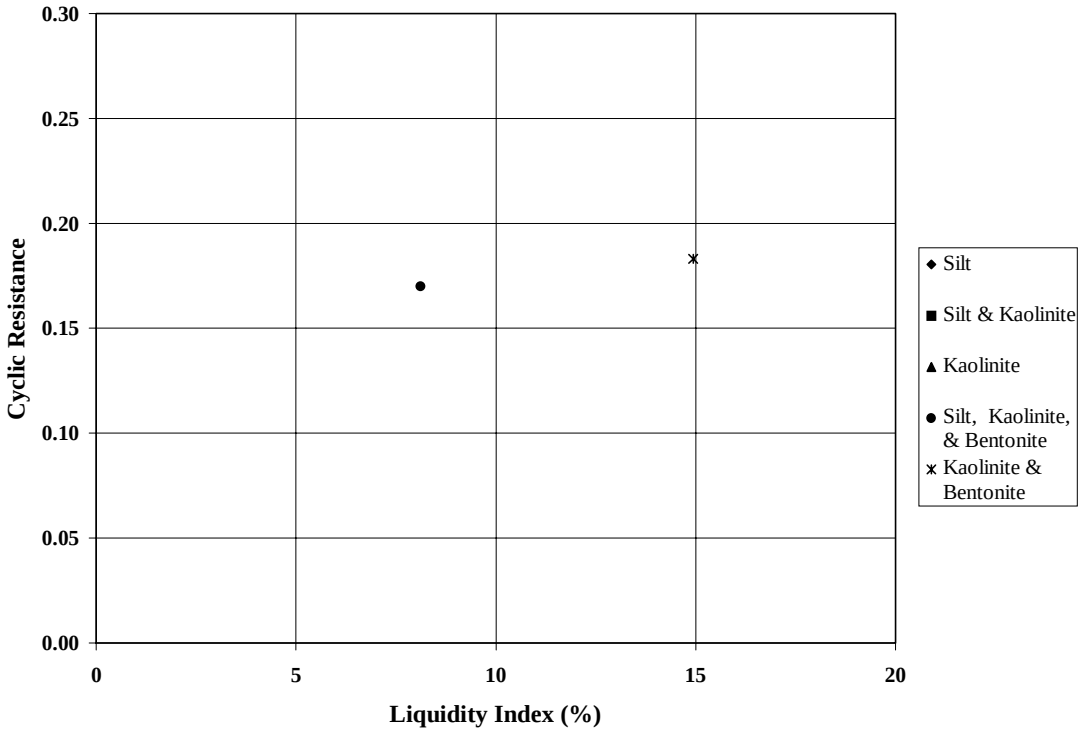

Figure 5-15: Variation in cyclic resistance with liquidity index for specimens prepared to a constant fines content

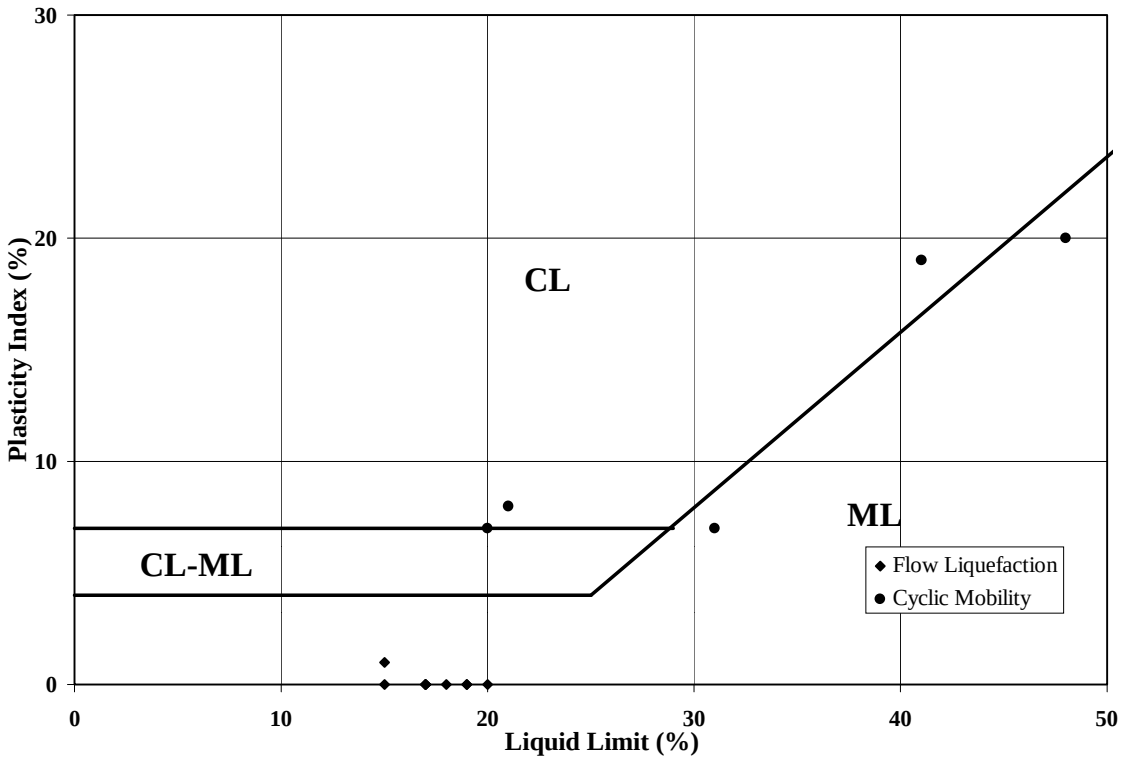

Figure 5-16: Liquefaction behavior as a function of Atterberg limits for Yatesville sand with silty and clayey fines

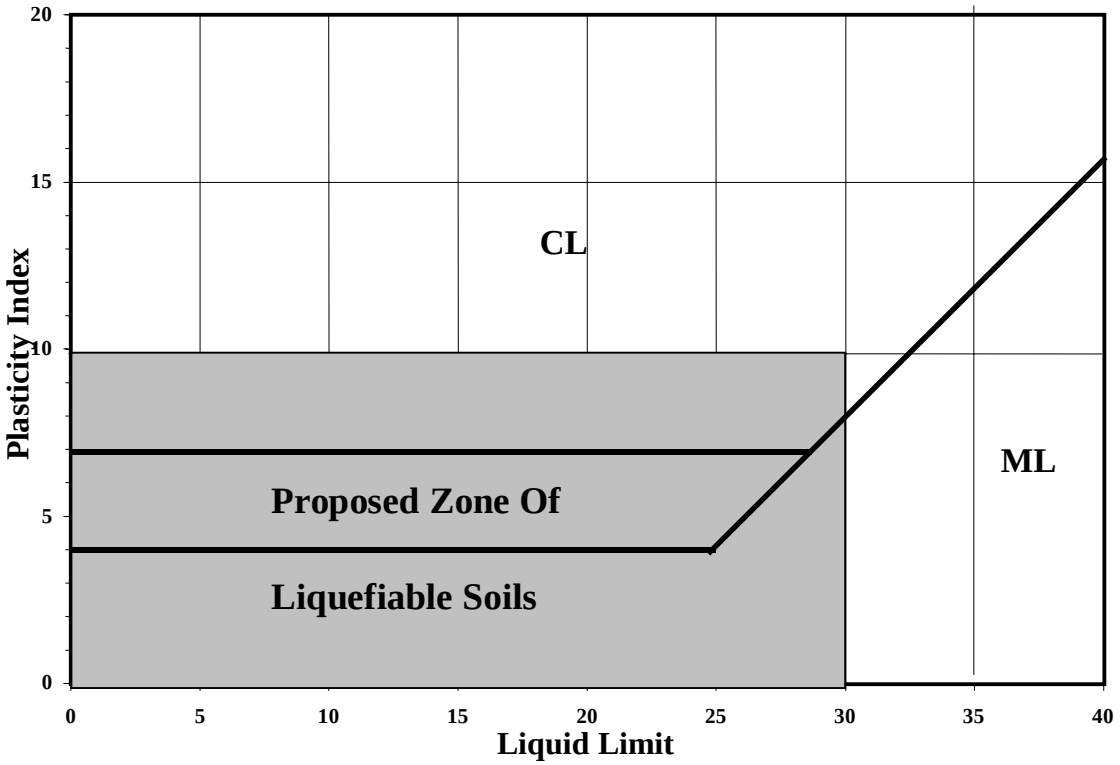

Figure 5-17: Proposed zone of liquefiable soils

Chapter 6: Pore Pressure Generation

The effects of fines content and plasticity on the generation of pore pressures during cyclic loading may be evaluated using several methods. First the data acquired during this study will be examined in terms of the number of cycles of loading applied to the specimen relative to the number of cycles of loading required to cause initial liquefaction. Next, the data will be examined in terms of the strains required to produce a given level of pore pressure ratio.

6.1 Pore Pressure Development As A Function Of Loading

Several studies (Lee and Albaisa, 1974; De Alba et al., 1976) have shown that if the pore pressure ratio of a clean sand is plotted against the ratio of the cycle of loading to the total number of cycles required to cause initial liquefaction (this ratio is referred to as the loading ratio), over a wide range of densities the results will plot within a narrow band. The pore pressure generation bands produced in this manner by these two studies are presented in Figure 6-1.

6.1.1 Clean Sands

The method of plotting pore pressure increase as a function of the loading ratio was applied to the results of the tests on clean Yatesville and Monterey sands using the peak pore pressure measured during each cycle. While the pore pressure generation characteristics of both those soils subject to flow liquefaction and cyclic mobility were found to plot within narrow bands, those bands were found to be different in both shape and location.

6.1.1.1 Clean Sands Susceptible To Flow Liquefaction

For specimens that underwent flow liquefaction, it was found that the data plotted within a narrow band with a distinctive shape. This may be seen in Figure 6-2 for Yatesville

sand specimens ranging in relative density from -35 to 25 percent. Similarly, this may be seen in Figure 6-3 for Monterey sand specimens ranging in relative density from -5 to 58 percent. The pore pressure ratio first increases rapidly, then slows with increasing loading ratio, and finally begins increasing rapidly again as the number of cycles begins to approach the number of cycles required to cause initial liquefaction. This gives the lower boundary curve its distinctive concave downward, then concave upward shape.

6.1.1.2 Clean Sands Susceptible To Cyclic Mobility

For specimens that underwent cyclic mobility, it was found that the data plotted within a narrow band with a distinctive shape. This may be seen in Figure 6-4 for Yatesville sand specimens with a relative density 68 percent. For these specimens the pore pressure ratio at first increases even more rapidly than in the specimens susceptible to flow liquefaction. However, unlike those specimens, the specimens susceptible to cyclic mobility continue to produce pore pressures at a nearly constant rate and begin to slow as the number of cycles begins to approach the number of cycles required to cause initial liquefaction. This gives the lower boundary curve its flatter, concave downward shape.

6.1.2 Sands With Non-Plastic Fines

This method of plotting described above was applied to the results of the tests on mixtures of Yatesville or Monterey sands with silt that failed in both in flow liquefaction and cyclic mobility.

6.1.2.1 Silty Soils Subject To Flow Liquefaction

For each mixture of silty sand and sandy silt subject to flow liquefaction, the data was plotted in the manner described, and banded by upper and lower bounds. The average of these two bounds, which represents the average pore pressure generation in the soil, was then plotted with the averages from the other soil mixtures. As may be seen in Figure 6-5 for mixtures of Yatesville sand and silt, and in Figure 6-6 for mixtures of Monterey sand and silt, these also plot within a relatively narrow band.

Because these soils show consistent pore pressure generation characteristics over such a large range of silt contents, it can be concluded that in soils subject to flow liquefaction, pore pressure generation is independent of silt content when examined in terms of the loading ratio.

6.1.2.2 Silty Soils Subject To Cyclic Mobility

For each mixture of silty sand and sandy silts subject to cyclic mobility, the data was plotted in the manner described, and banded by upper and lower bounds. The average of those two bounds, which represented the average pore pressure generation in the soil, was then plotted with the averages from the other soil mixtures. As may be seen in Figure 6-7 for mixtures of Yatesville sand and silt these also plot within a relatively narrow band. The shape of the boundary curves in Figure 6-7 are very similar to those in Figure 6-4 for the clean Yatesville sand subject to cyclic mobility.

6.1.2.3 Comparison Of Pore Pressure Generation In Silty Soils

While the pore pressure development in sands susceptible to flow liquefaction and cyclic mobility each plot within a narrow band, the pore pressure development in specimens susceptible to cyclic mobility is different than that for specimens susceptible to flow liquefaction. At a given loading ratio, the pore pressures developed in a specimen susceptible to cyclic mobility are generally higher than those developed in a specimen susceptible to flow liquefaction. This can be seen in Figure 6-8, which plots the average pore pressure generation as a function of loading ratio for Yatesville sand with 12 percent fines. The two curves represent the average of those specimens that develop cyclic mobility and those that develop flow liquefaction. When the loading ratio has reached 40 percent, the specimens that will undergo cyclic mobility have developed pore pressures equal to 55 percent of the initial confining stress, while those that will undergo flow liquefaction have only developed pore pressures equal to 41 percent of the initial confining stress.

6.1.3 Sands With Plastic Fines

In order to examine the effects of plastic fines content on pore pressure generation, the data from a series of specimens formed with kaolinite in varying percentages were compared. This comparison is presented in Figure 6-9. Next, in order to examine the effects of fines plasticity on pore pressure generation, the data from a series of specimens formed with 17 percent fines of varying composition were compared. This comparison is presented in Figure 6-10.

6.1.4.1 Effects of Plastic Fines Content

The effect of plastic fines content was examined by testing a series of specimens prepared at a constant soil specific relative density while altering the quantity of kaolinite added. As may be seen in Figure 6-9, as the quantity of plastic fines increases, the shapes of the average pore pressure generation curves for these specimens transform from that characteristic of soils susceptible to flow liquefaction to that characteristic of soils susceptible to cyclic mobility. This agrees with the previous findings of this study which show that as the plasticity of the soil increases, the liquefaction mode of the soil moves from flow liquefaction to cyclic mobility.

6.1.4.2 Effects of Fines Plasticity

The effect of fines plasticity was examined by testing a series of specimens at a constant fines content and relative density while altering the make-up of the fines in order to alter their plasticity. As can be seen in Figure 6-10 that, the shape of the average pore pressure generation curves for these specimens transform from that characteristic of soils susceptible to flow liquefaction to that characteristic of soils susceptible to cyclic mobility as the plasticity of the fines increases. This agrees with the previous findings of this study which show that as the plasticity of the soil increases, the liquefaction mode of the soil moves from flow liquefaction to cyclic mobility.

6.2 Pore Pressure Development As A Function Of Strain

Dobry et al (1982) examined of pore pressure generation during undrained cyclic loading as a function of shear strain. They found, that for a constant number of loading cycles to a constant shear strain, the relationship between pore pressure generation and shear strain is essentially identical over a wide range of relative densities. Additionally, they found that pore pressure do not begin to increase until some level of cyclic strain, deemed the threshold strain, is reached.

While no constant strain tests were run during this study, the data was examined to see what strain levels were required to achieve one percent and seventy percent residual pore pressure ratios and how the presence of non-plastic and plastic fines affected these strains. Residual pore pressure ratio is the pore pressure ratio calculated at the end of a cycle or a half cycle when the deviator stress (and hence the applied shear stress) is equal to zero.

6.2.1 Clean Sands and Soils with Non-Plastic Fines

The amount of axial strain required to reach residual pore pressure ratios of one and seventy percent were measured for both clean Yatesville and Monterey sands and Yatesville and Monterey sands with silt . Unlike the loading ratio method used in the previous section, the strain-based method seem to be independent of the liquefaction mode of the specimen, whether it is flow liquefaction and cyclic mobility.

6.2.2.1 Effect Of Fines Content

The amount of axial strain required to achieve a residual pore pressure ratio of one percent is independent of silt content as may be seen in Figure 6-11. While the data presented in Figure 6-11 is for Yatesville sand with silt, Monterey sand and silt show a similar distribution. Additionally, when examined at a residual pore pressure ratio of seventy percent, the strain level is again independent of the silt content.

6.2.2.2 The Effect of Density

The amount of axial strain required to achieve a residual pore pressure ratio of one percent is a function of both the limiting silt content and the density of the specimen.

For specimens below the limiting silt content, the amount of axial strain required to achieve a residual pore pressure ratio of one percent increases as the soil specific relative density of the specimen increases. This may be seen for a residual pore pressure ratio of one percent in Yatesville sand and silt mixtures in Figure 6-12. Yatesville sand and silt mixtures show a similar trend at a residual pore pressure ratio of seventy percent. The Monterey sand and silt mixtures show a similar trend at each residual pore pressure ratio.

For soils above the limiting silt content, the soil shows decreasing strain to one percent residual pore pressure ratio with increasing silt fraction void ratio. This may be seen in Figure 6-13. Additionally, when examined at a residual pore pressure ratio of seventy percent this trend is again observed.

6.2.2.3 Soil Tested At A Constant Relative Density

For Yatesville sand and silt specimens prepared to a constant soil specific relative density of 25 percent there is some trend towards increasing strain to one percent residual pore pressure ratio with increasing silt content. This may be seen in Figure 6-14.

Conversely, as may be seen in Figure 6-15, when examined at a residual pore pressure ratio of seventy percent, this trend is reversed, with the strain required to achieve seventy percent residual pore pressure ratio decreasing as silt content increases.

6.2.2 Sands with Plastic Fines

In order to examine the effects of plastic fines content on the strains required for pore pressure generation, the data from all tests with plastic fines were examined in terms of

their fines content, their clay content, their liquid limit, their plasticity index, and their activity. Next, in order to examine the effects of fines plasticity on the strains required for pore pressure generation, the data from a series of specimens formed with 17 percent fines of varying composition were compared in terms of their clay content, their liquid limit, their plasticity index, and their activity.

6.2.2.1 Soils With Varying Fines Content And Plasticity

A series of tests were run to evaluate the effects of fines content and plasticity. This was done by varying the fines content from 4 to 37 percent while also varying the fines composition and plasticity.

6.2.2.1.1 The Effects of Fines Content

The strain required for pore pressure generation in clayey sands appears to be independent of fines content, whether measured at a residual pore pressure ratio of one percent or seventy percent. This may be seen in Figure 6-16 which shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to pure bentonite, and with fines contents from 4 to 37 percent.

6.2.2.1.2 The Effects of Clay Content

The strain required for pore pressure generation in clayey sands appears to be independent of clay content, whether measured at a residual pore pressure ratio of one percent or seventy percent. This may be seen in Figure 6-17 which shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to pure bentonite, and with clay contents from 2 to 37 percent.

6.2.2.1.3 The Effects of Liquid Limit

The strain required for pore pressure generation in clayey sands increases with increasing liquid limit, whether measured at a residual pore pressure ratio of one percent or seventy

percent. This may be seen in Figure 6-18 which shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to pure bentonite, and with fines contents from 4 to 37 percent. These soils have liquid limits ranging from 15 to 48.

6.2.2.1.3 The Effects of Plasticity Index

The strain required for pore pressure generation in clayey sands increases with increasing plasticity index, whether measured at a residual pore pressure ratio of one percent or seventy percent. This may be seen in Figure 6-19 which shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to pure bentonite, and with fines contents from 4 to 37 percent. These soils have plasticity indexes ranging from 0 to 20.

6.2.2.1.3 The Effects of Activity

The strain required for pore pressure generation in clayey sands increases with increasing activity, whether measured at a residual pore pressure ratio of one percent or seventy percent. This may be seen in Figure 6-19 which shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to pure bentonite, and with clay contents from 2 to 37 percent. These soils have activities ranging from 0 to 1.67.

6.2.2.2 Soils With Constant Fines Content And Varying Plasticity

A series of tests were run to isolate the effects of fines plasticity. This was done by holding the fines content constant at 17 percent and varying the fines composition and plasicity.

6.2.2.2.1 The Effect of Clay Content

The strain required for pore pressure generation in clayey sands with a constant fines content appears to be independent of clay content, whether measured at a residual pore pressure ratio of one percent or seventy percent. This may be seen in Figure 6-21 which

shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to a half kaolinite, half bentonite mix. The clay contents ranged from 2 to 17 percent.

6.2.2.1.2 The Effects of Liquid Limit

The strain required for pore pressure generation in clayey sands with a constant fines content increases with increasing liquid limit, whether measured at a residual pore pressure ratio of one percent or seventy percent. This may be seen in Figure 6-22 which shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to a half kaolinite, half bentonite mix. The fines contents ranged from 4 to 17 percent. These soils have liquid limits ranging from 15 to 41.

6.2.2.1.3 The Effects of Plasticity Index

The strain required for pore pressure generation in clayey sands with a constant fines content increases with increasing plasticity index, whether measured at a residual pore pressure ratio of one percent or seventy percent. This may be seen in Figure 6-23 which shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to a half kaolinite, half bentonite mix. The clay contents fines contents ranged from 4 to 17 percent. These soils have plasticity indexes ranging from 0 to 19.

6.2.2.1.3 The Effects of Activity

The strain required for pore pressure generation in clayey sands with a constant fines content increases with increasing activity, whether measured at a residual pore pressure ratio of one percent or seventy percent. This may be seen in Figure 6-24 which shows the strain required to achieve a residual pore pressure ratio of one percent in Yatesville sand with fines ranging from a half silt and half kaolinite mix to a half kaolinite, half bentonite mix. The clay contents ranged from 2 to 17 percent. These soils have activities ranging from 0 to 1.12.

6.3 Conclusions

Data from the cyclic triaxial tests were analyzed to evaluate the effects of fines content and plasticity on the pore pressure generation characteristics of silty and clayey sands. Pore pressure generation was evaluated both in terms of the number of cycles of loading relative to the number of cycles of loading required to achieve initial liquefaction and in terms of the amount of strain require to achieve a certain level of pore pressure ratio.

When analyzed in terms of the number of cycles of loading relative to the number of cycles of loading required to achieve initial liquefaction (i.e. loading ratio), the pore pressure ratio for each sand with silt was found to fit into a narrow band. The shape and position of that band depends on whether the soils is susceptible to flow liquefaction or cyclic mobility, and is independent of silt content. At any given loading ratio the pore pressure ratio is larger for a soil susceptible cyclic mobility than it is for a soil susceptible to flow liquefaction.

When analyzed in terms of the loading ratio, the pore pressure ratio for sands with plastic fines varies as the plasticity of the soils varies. Soils with low plasticity produced pore pressure generation curves similar to those produced for soils with non-plastic fines susceptible to flow liquefaction. Soils with high plasticity produced pore pressure generation curves similar to those produced for soils with non-plastic fines susceptible to cyclic mobility.

Because the pore pressures generated by these soils tend to fall into narrow bands when plotted using the loading ratio, these curves may be used to estimate the pore pressures generated in these soils if liquefaction does not occur. If the number of cycles required to liquefy the soil at the stress levels imposed by the design earthquake are known, the curve may be used to estimate the pore pressure ratio developed in the soil mass. This is

done by entering the chart at the point corresponding to the ratio of the number of cycles in the design earthquake to the number of cycles required to liquefy the soil, moving up to the average pore pressure generation curve, and determining the corresponding pore pressure ratio.

When evaluated in terms of the strain required to reach some level of residual pore pressure ratio, where the residual pore pressure ratio is the pore pressure ratio measured during the point of zero deviator stress, behavior for soils with non-plastic fines is governed by the soil's silt content relative to the limiting silt content.

For soils with non-plastic fines below the limiting silt content, the strain required to generate pore pressures is related to soil specific relative density and the pore pressure level. For one percent residual pore pressure ratio, the strain required increases with increasing soil specific relative density. Conversely, for seventy percent residual pore pressure ratio, the strain required decreases with increasing soil specific relative density.

For soils with non-plastic fines above the limiting silt content the strain required to generate pore pressures is related to void ratio of the silt fraction. For both one percent and seventy percent residual pore pressure ratio, the strain required decreases with increasing void ratio of the silt fraction.

When analyzed in terms of the strain required to achieve a specified level of residual pore pressure ratio, pore pressure generation in sands with plastic fines is independent of fines content and clay content. The strain required increases with increasing liquid limit, plasticity index, and activity.

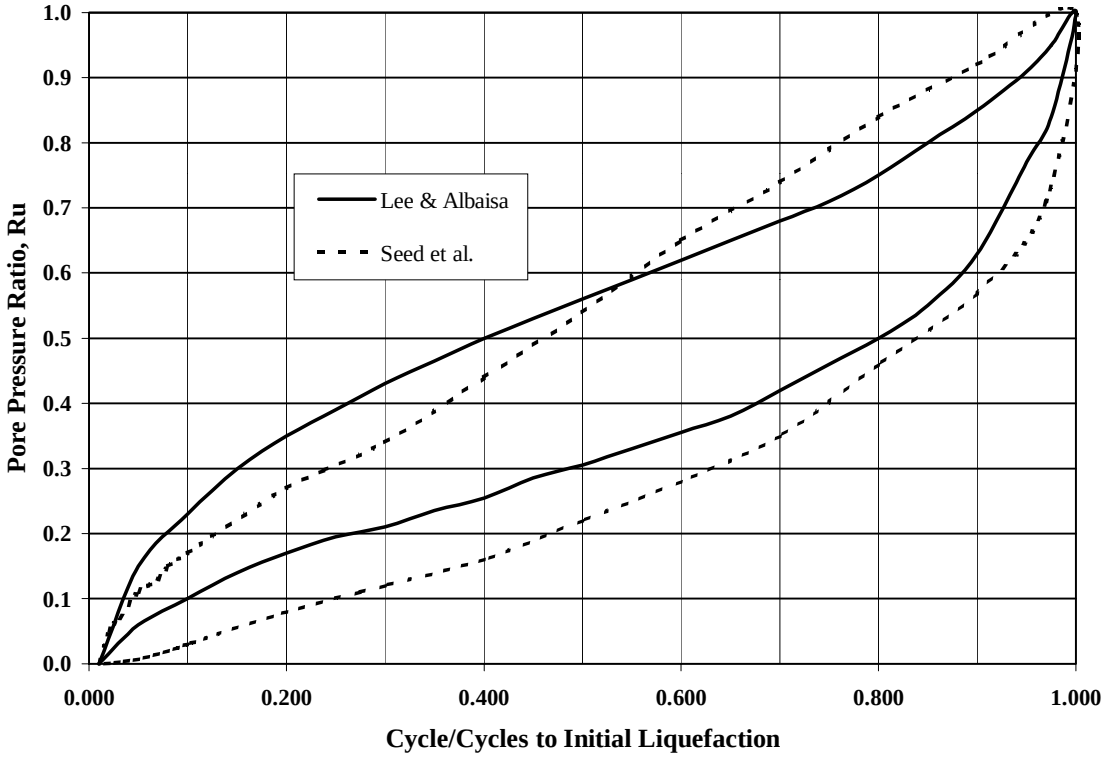

Figure 6-1: Pore pressure generation as a function of loading ratio (Lee and Albaisa, 1974; De Alba et al., 1976)

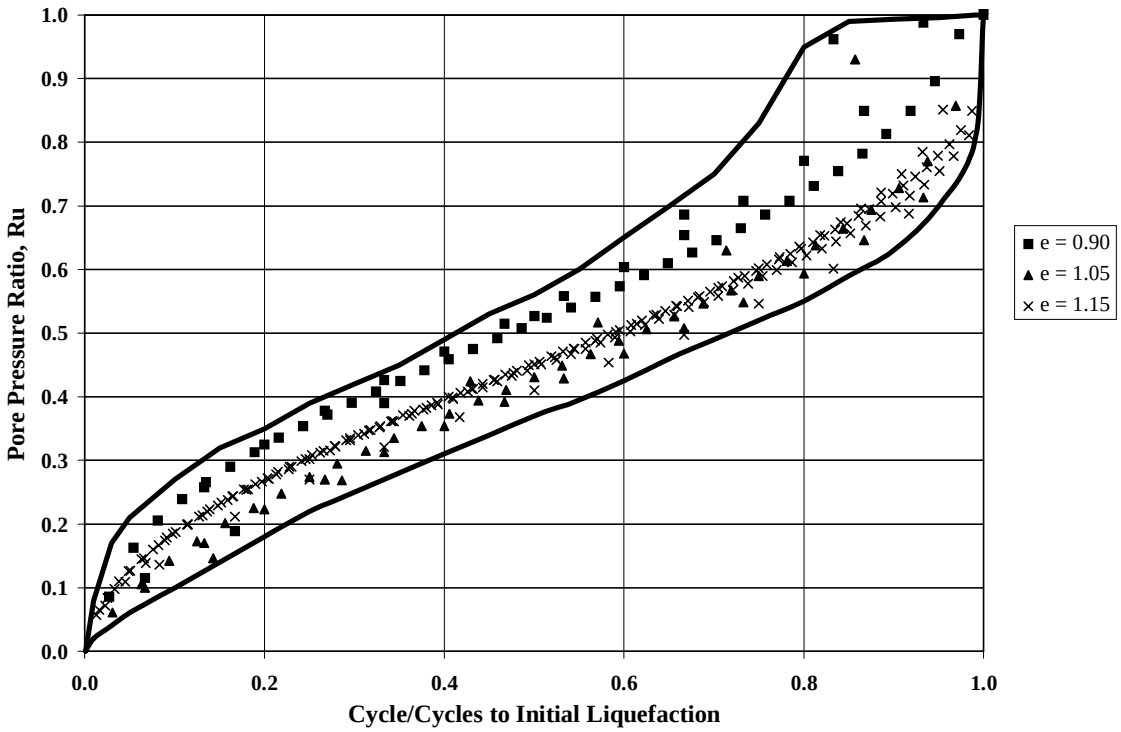

Figure 6-2: Pore pressure generation as a function of loading ratio for specimens of Yatesville sand susceptible to flow liquefaction

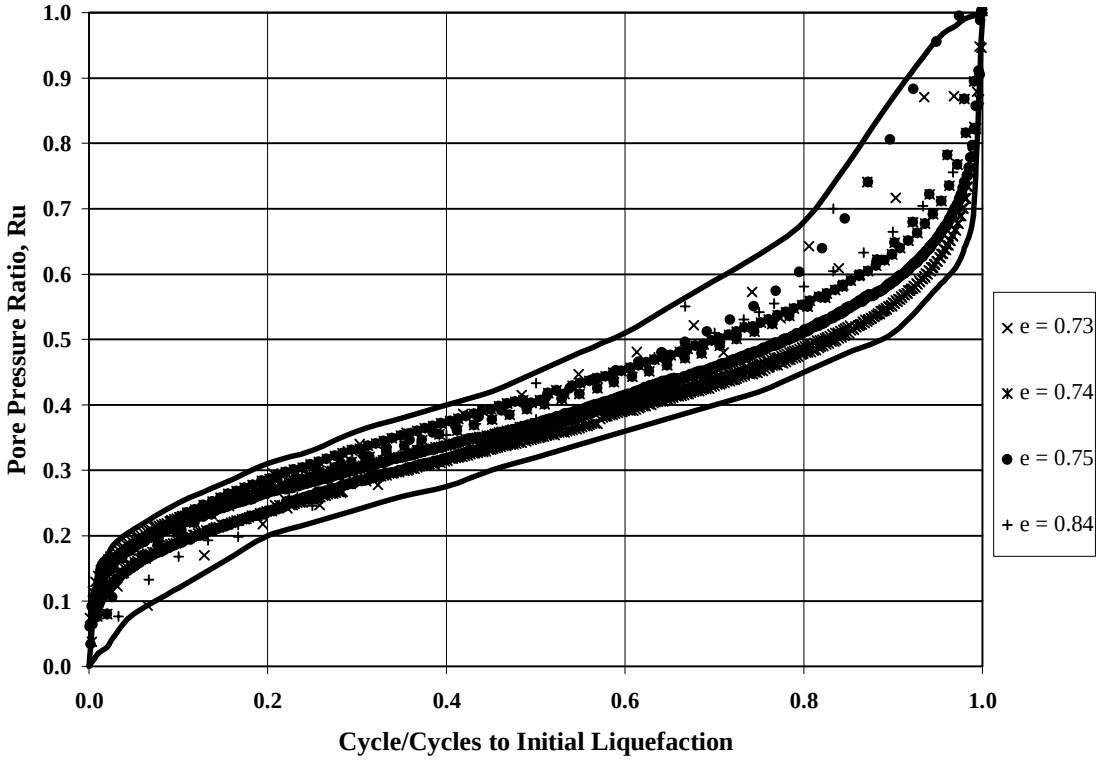

Figure 6-3: Pore pressure generation as a function of loading ratio for specimens of Monterey sand susceptible to flow liquefaction

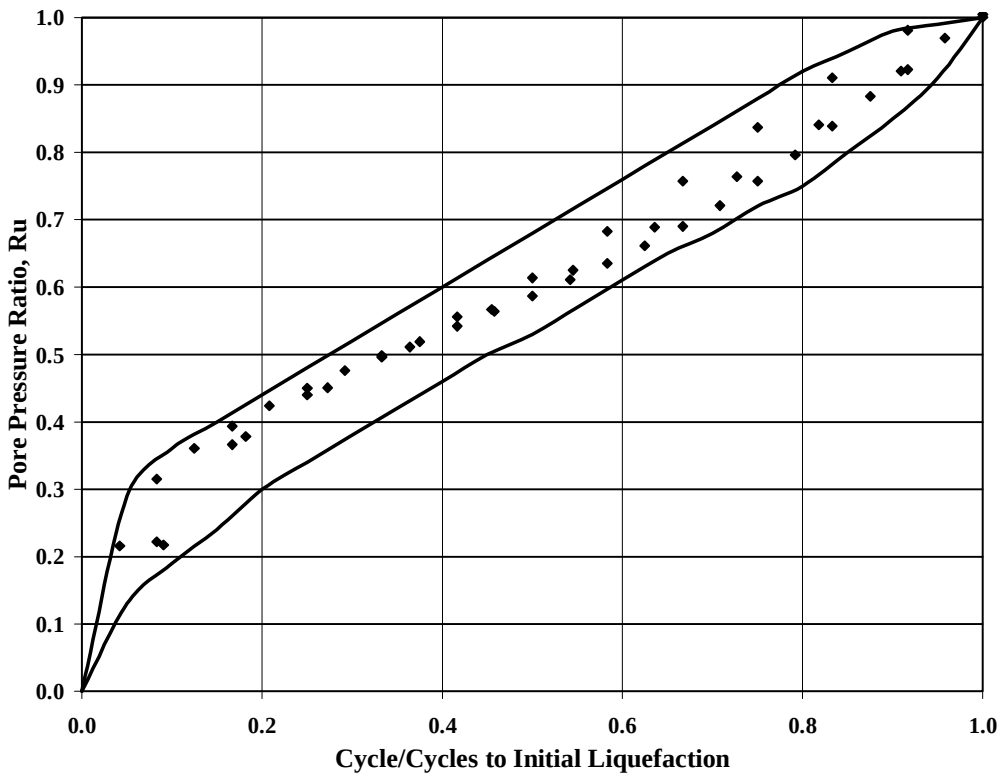

Figure 6-4: Pore pressure generation as a function of loading ratio for specimens of Yatesville sand susceptible to cyclic mobility

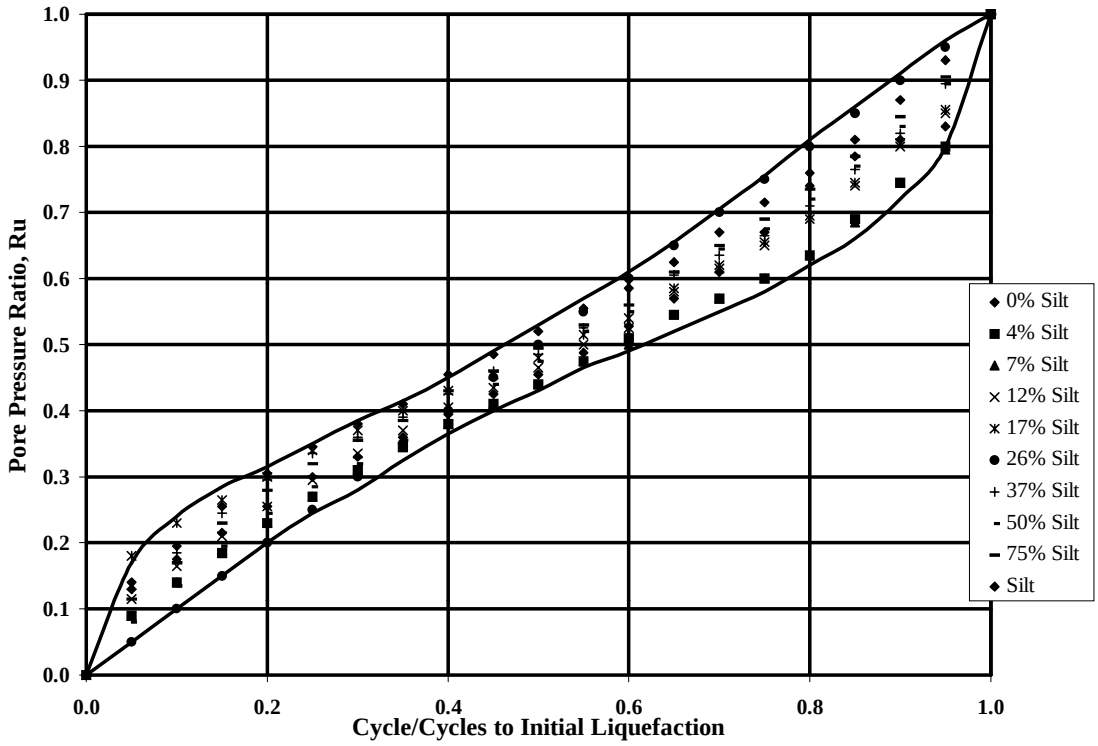

Figure 6-5: Pore pressure generation as a function of loading ratio for specimens of Yatesville sand and silt susceptible to flow liquefaction

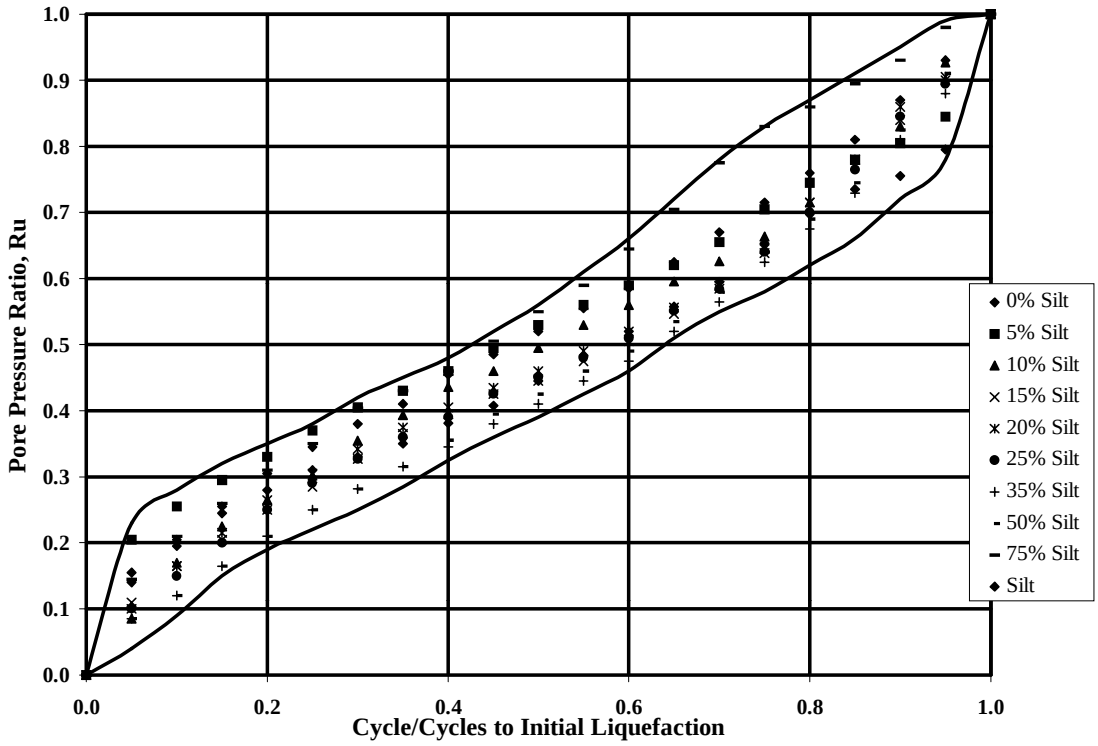

Figure 6-6: Pore pressure generation as a function of loading ratio for specimens of Monterey sand and silt susceptible to flow liquefaction

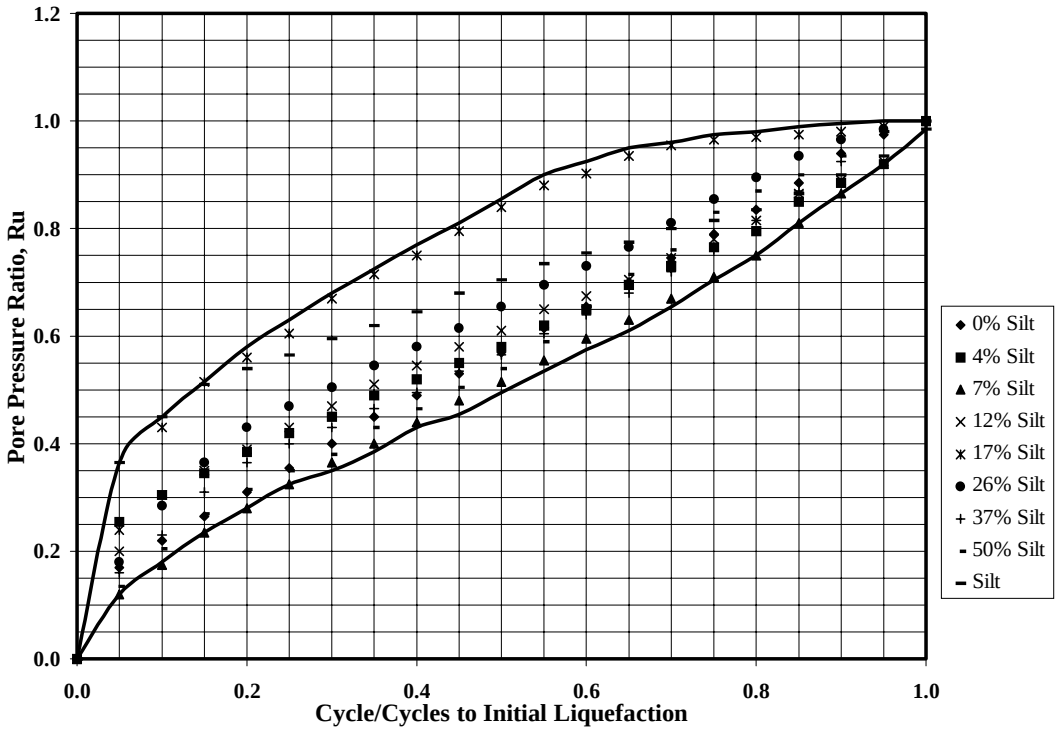

Figure 6-7: Pore pressure generation as a function of loading ratio for specimens of Yatesville sand and silt susceptible to cyclic mobility

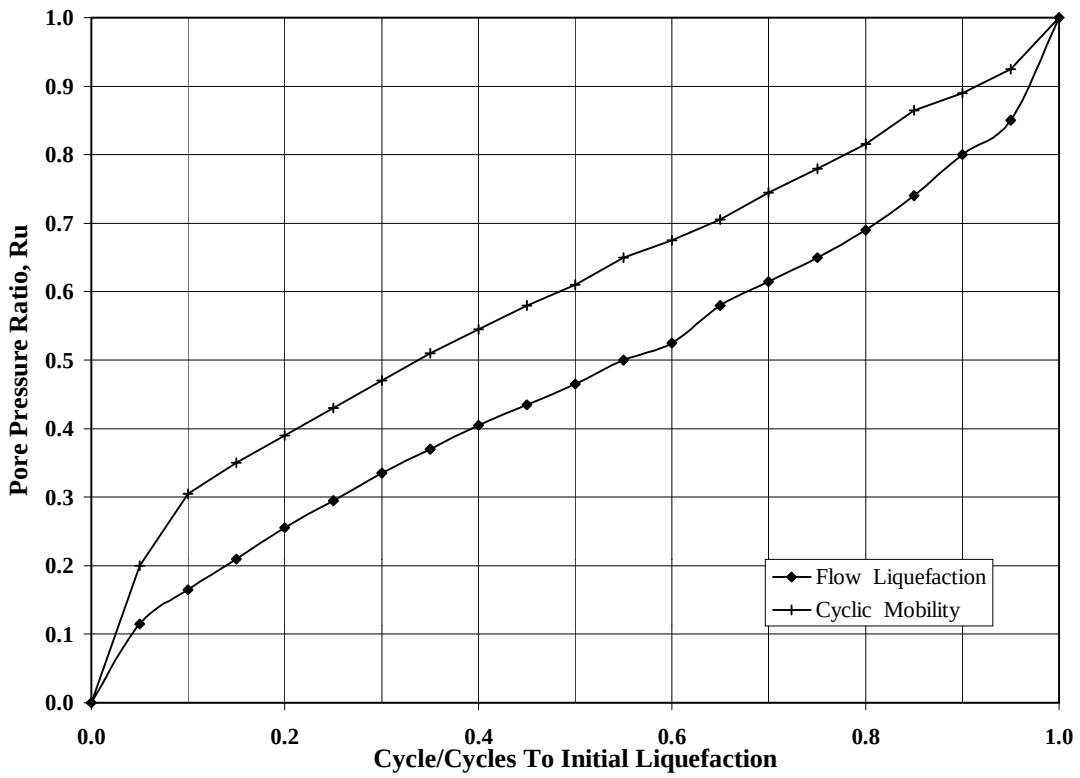

Figure 6-8: Comparison of average pore pressure generation for specimens of Yatesville sand with 12 percent silt susceptible to flow liquefaction and cyclic mobility

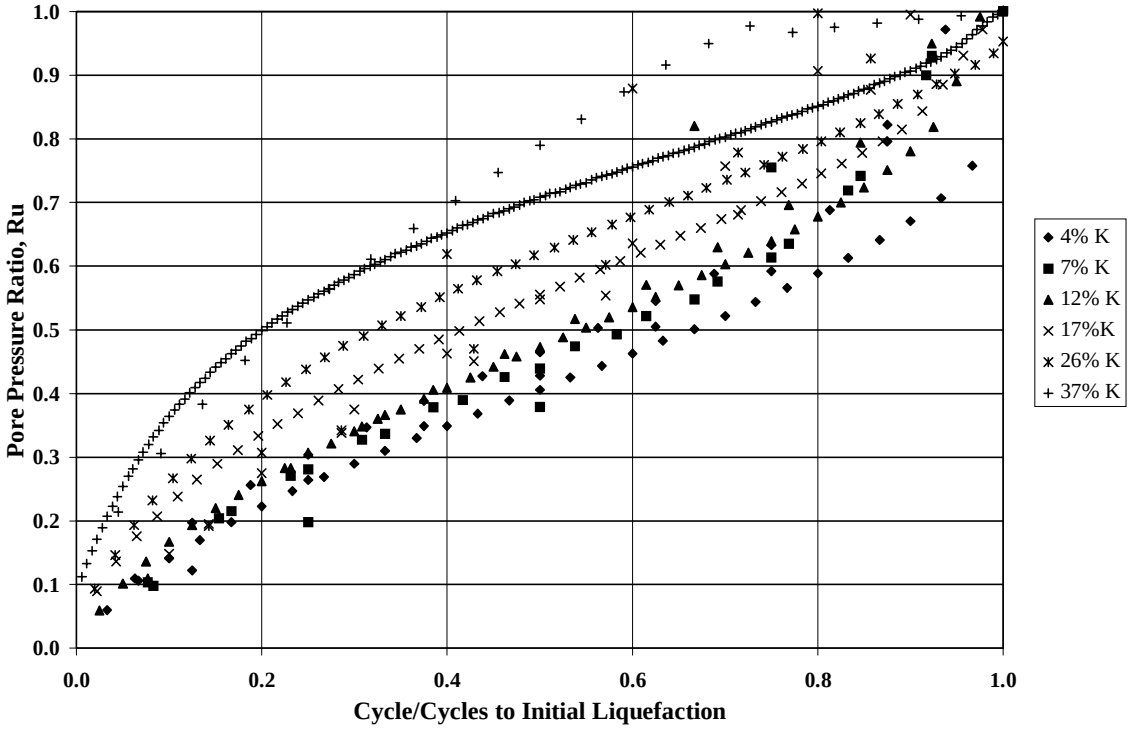

Figure 6-9: Pore pressure generation as a function of plasticity for specimens of Yatesville sand with kaolinite

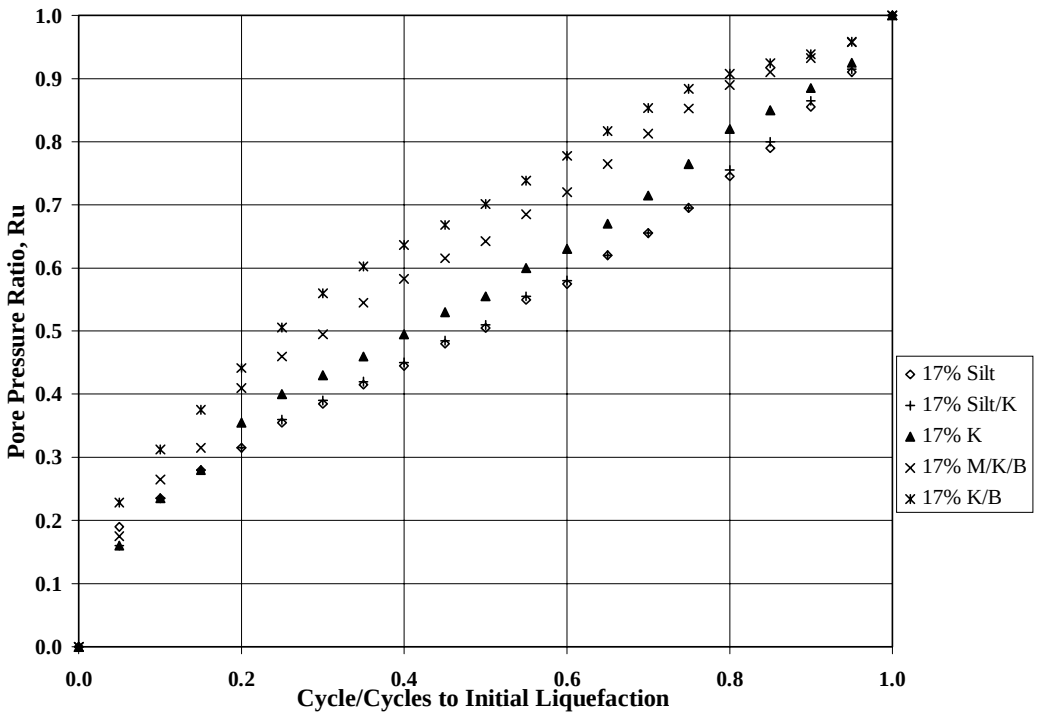

Figure 6-10: Pore pressure generation as a function of loading ratio for specimens of Yatesville sand with 17 percent plastic fines

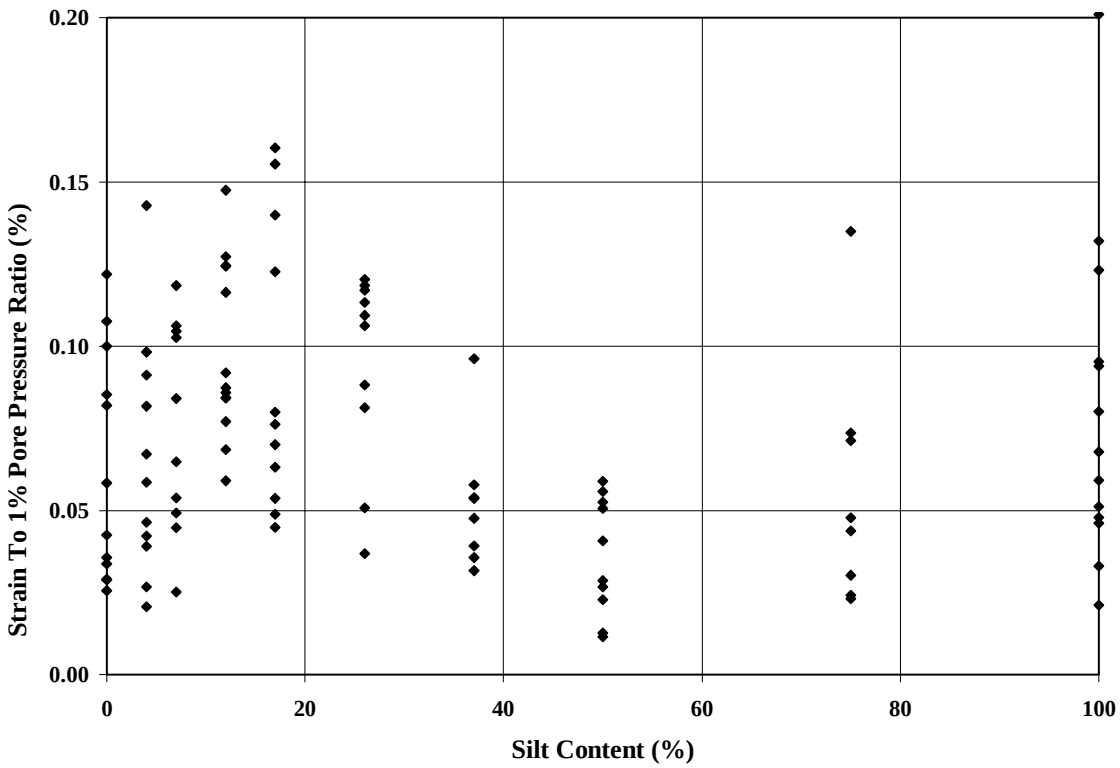

Figure 6-11: Effect of silt content on the strain required to achieve one percent residual pore pressure ratio for specimens of Yatesville sand with silt

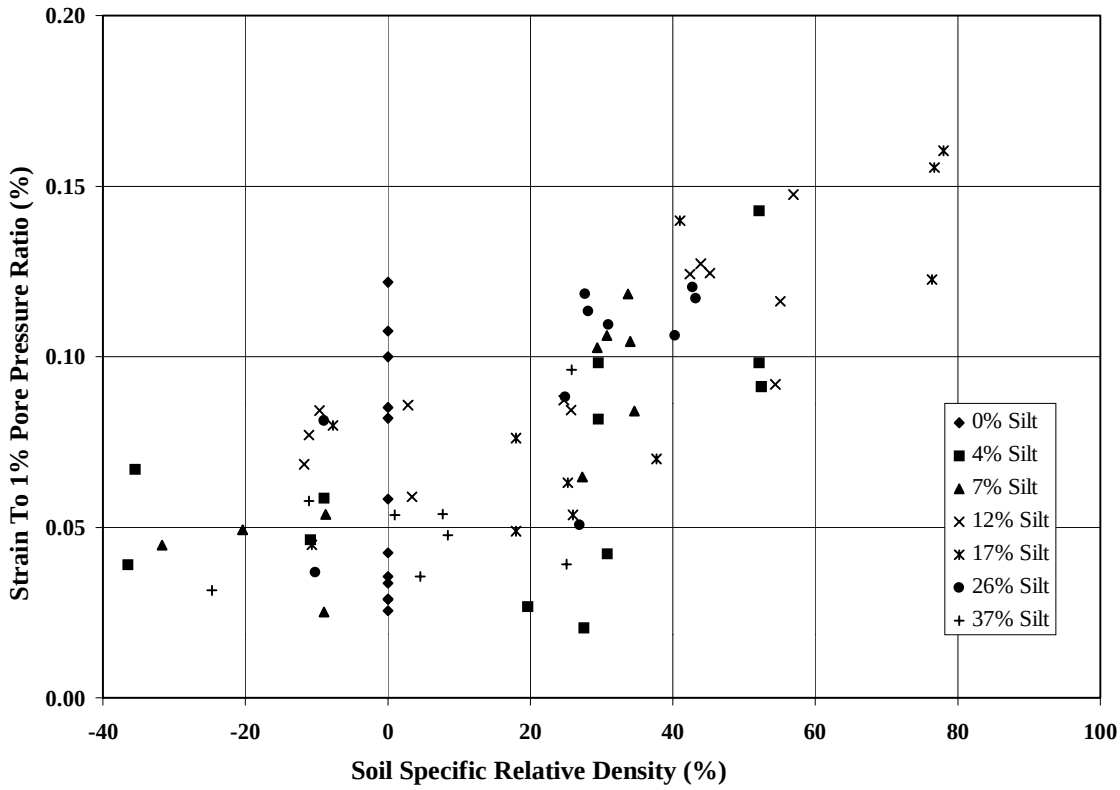

Figure 6-12: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of soil specific relative density for specimens of Yatesville sand with silt below the limiting silt content

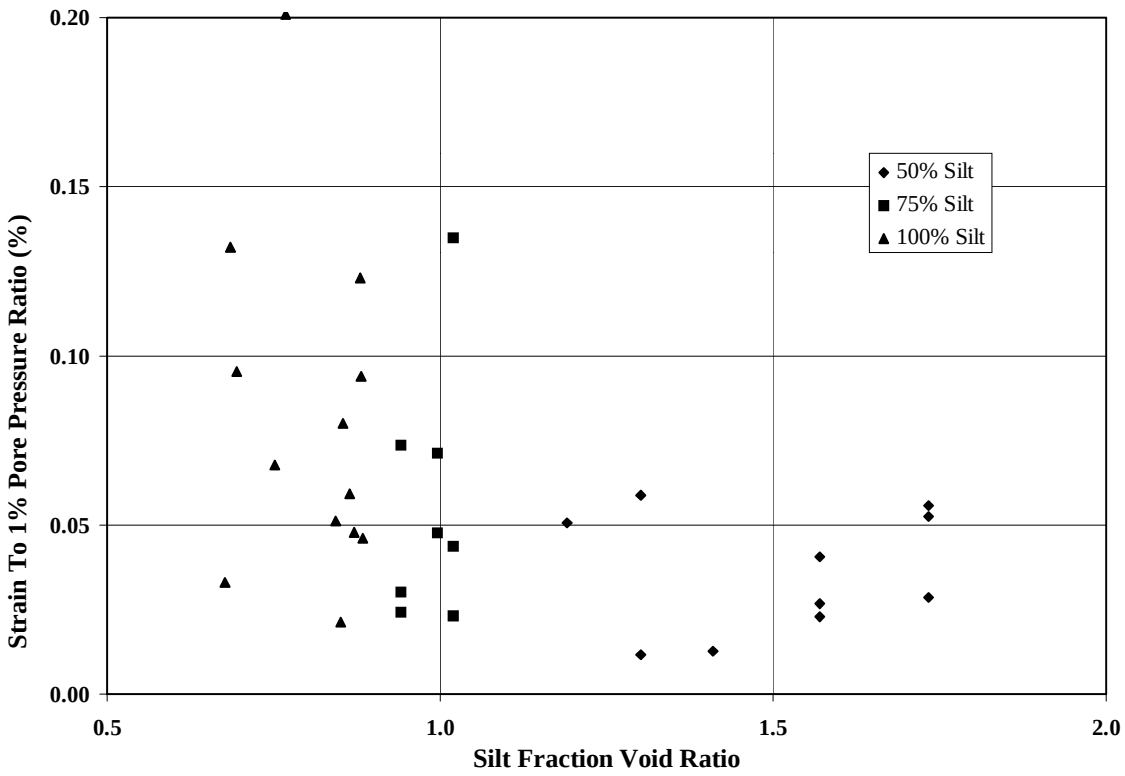

Figure 6-13: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of the silt fraction void ratio for specimens of Yatesville sand with silt above the limiting silt content

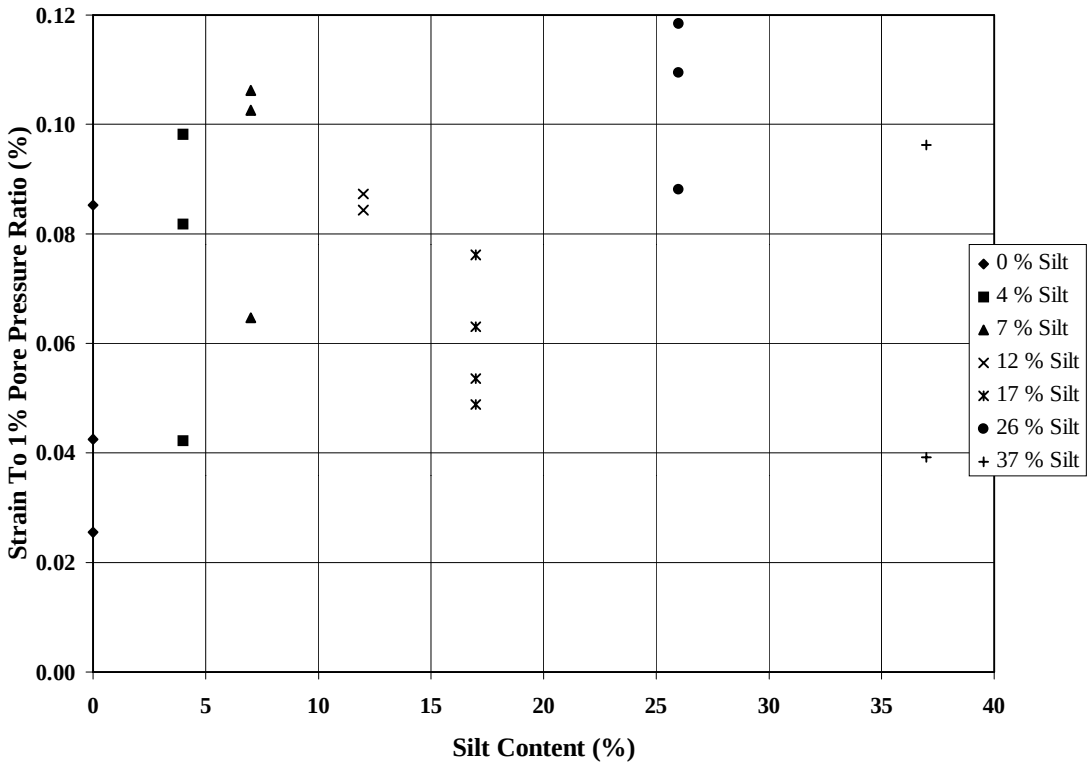

Figure 6-14: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of the silt content for specimens of Yatesville sand with silt prepared to a constant soil specific relative density

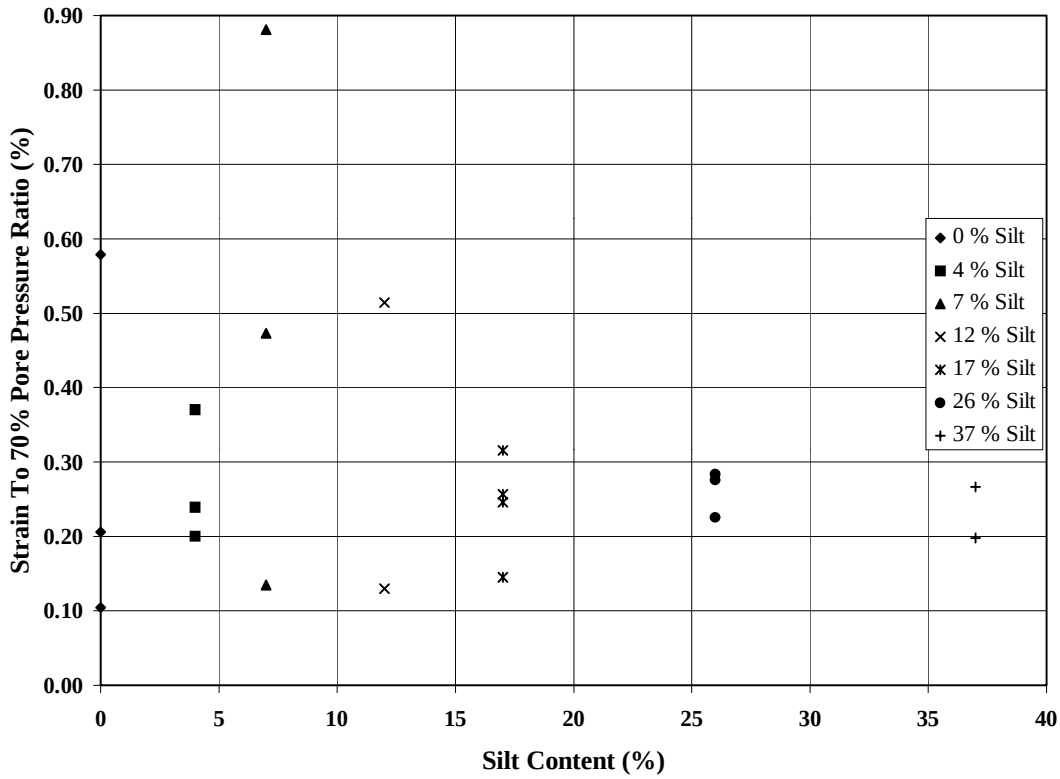

Figure 6-15: Variation in the strain required to achieve seventy percent residual pore pressure ratio as a function of the silt content for specimens of Yatesville sand with silt prepared to a constant soil specific relative density

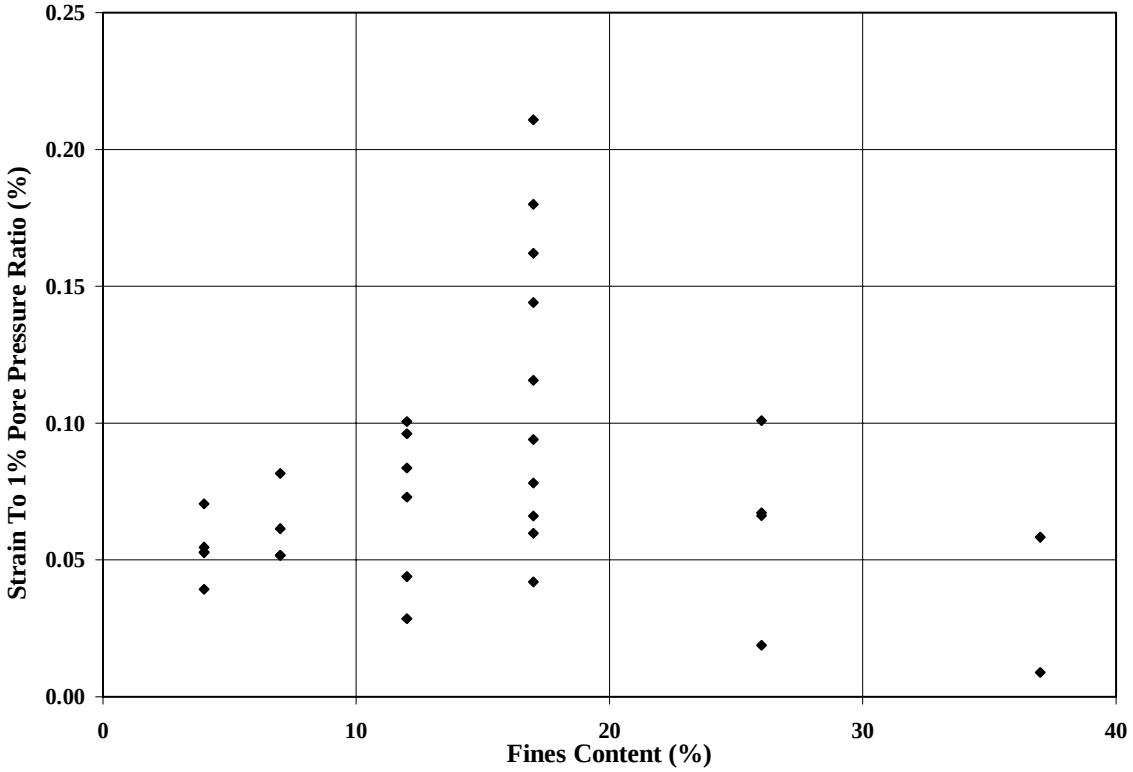

Figure 6-16: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of fines content for specimens of Yatesville sand with plastic fines

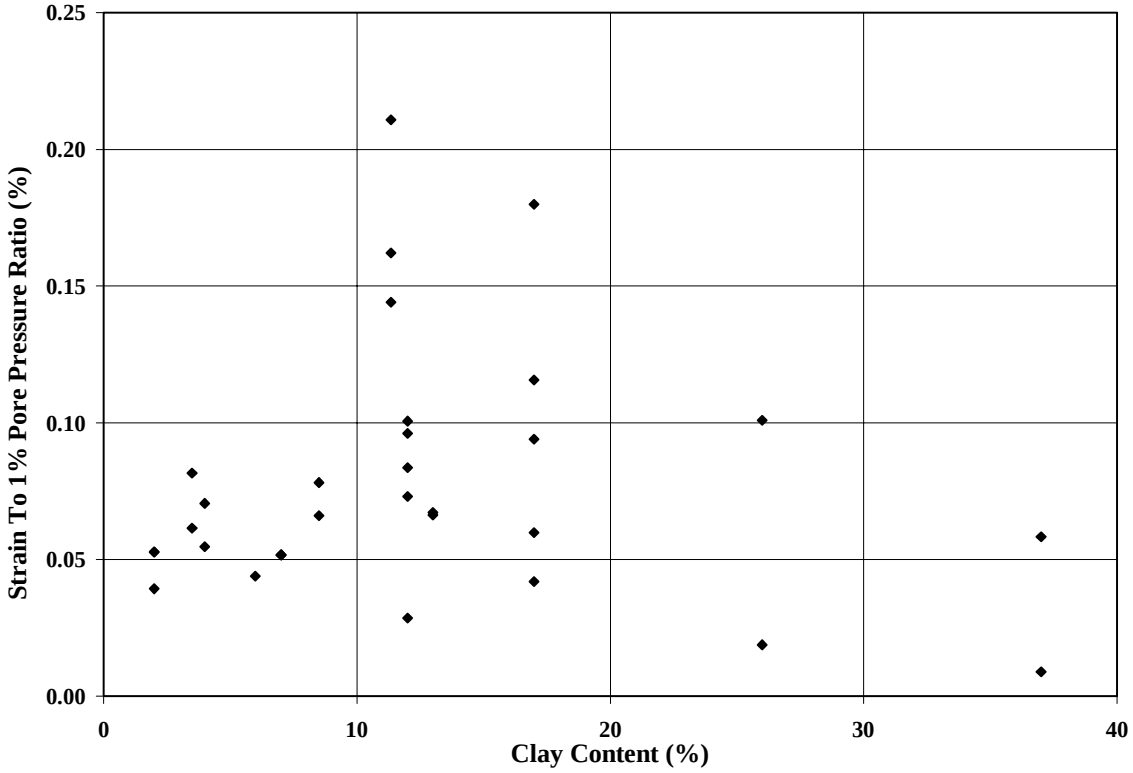

Figure 6-17: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of clay content for specimens of Yatesville sand with plastic fines

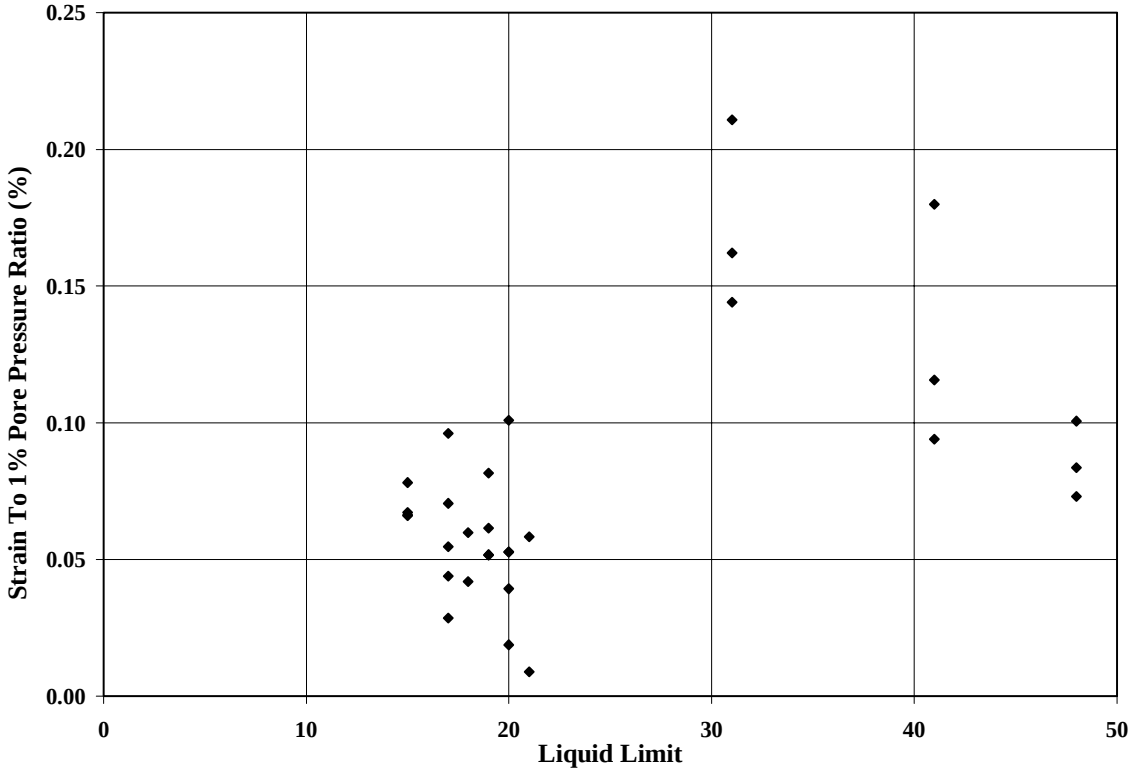

Figure 6-18: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of liquid limit for specimens of Yatesville sand with plastic fines

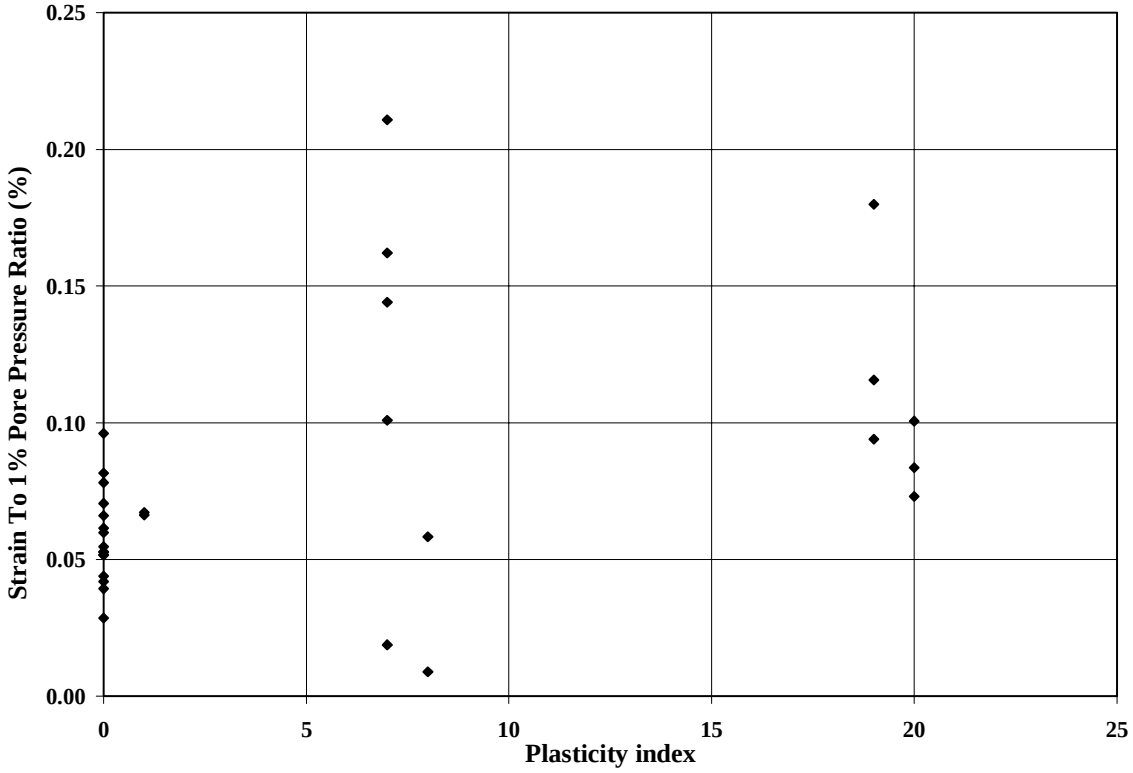

Figure 6-19: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of plasticity index for specimens of Yatesville sand with plastic fines

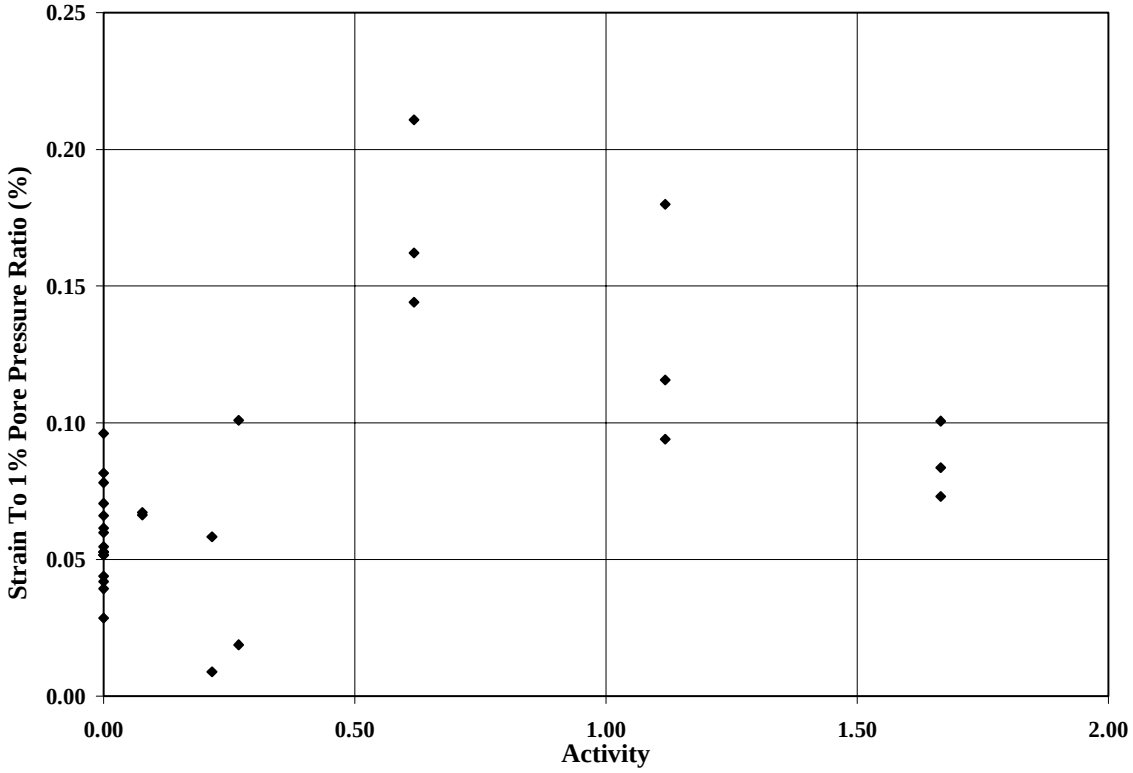

Figure 6-20: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of activity for specimens of Yatesville sand with plastic fines

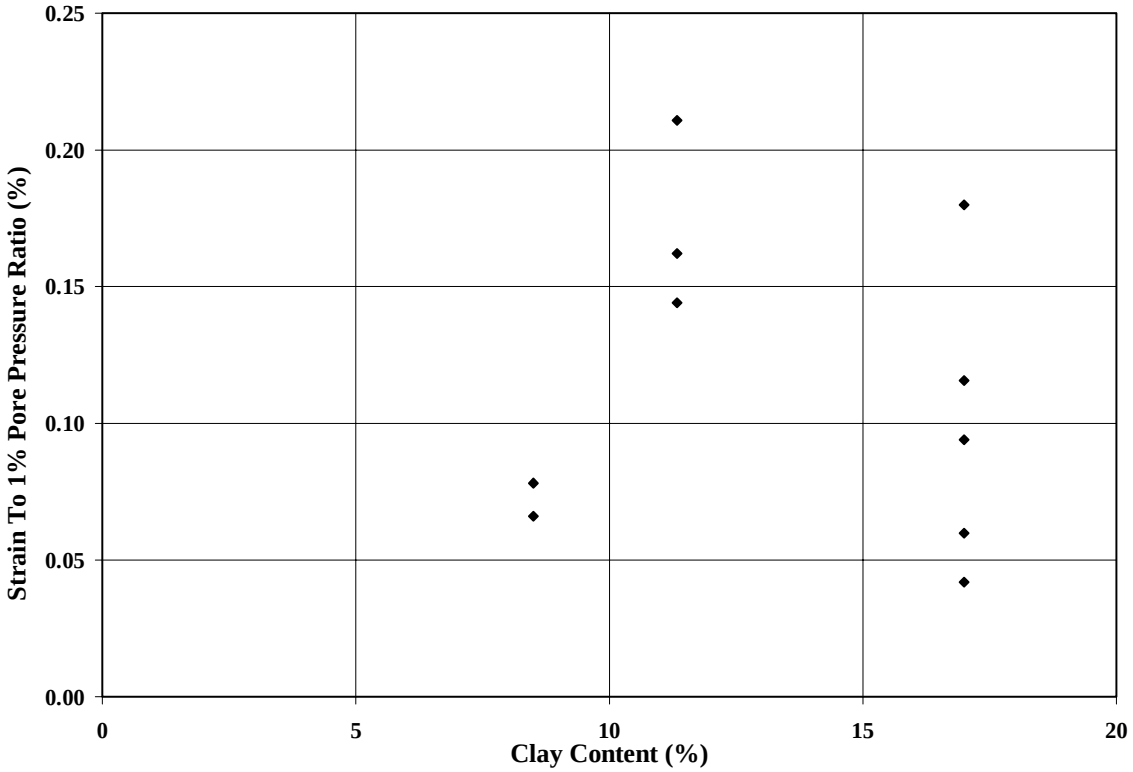

Figure 6-21: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of clay content for specimens of Yatesville sand with 17 percent plastic fines

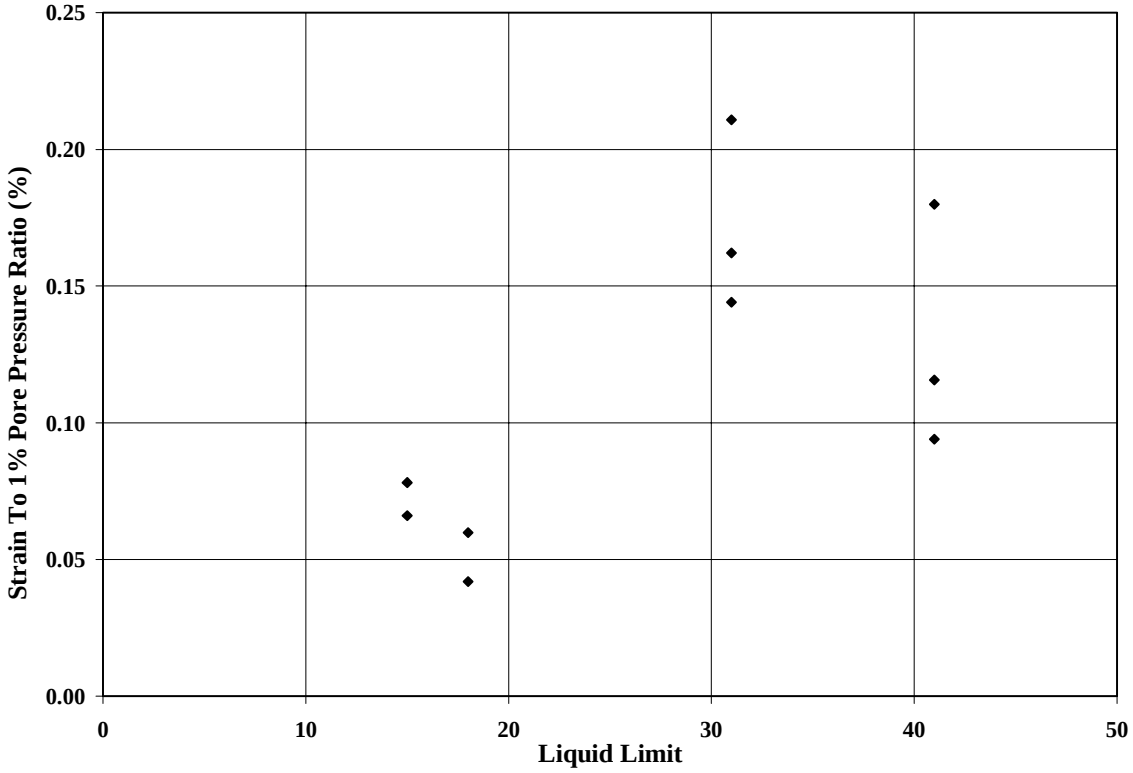

Figure 6-22: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of liquid limit for specimens of Yatesville sand with 17 percent plastic fines

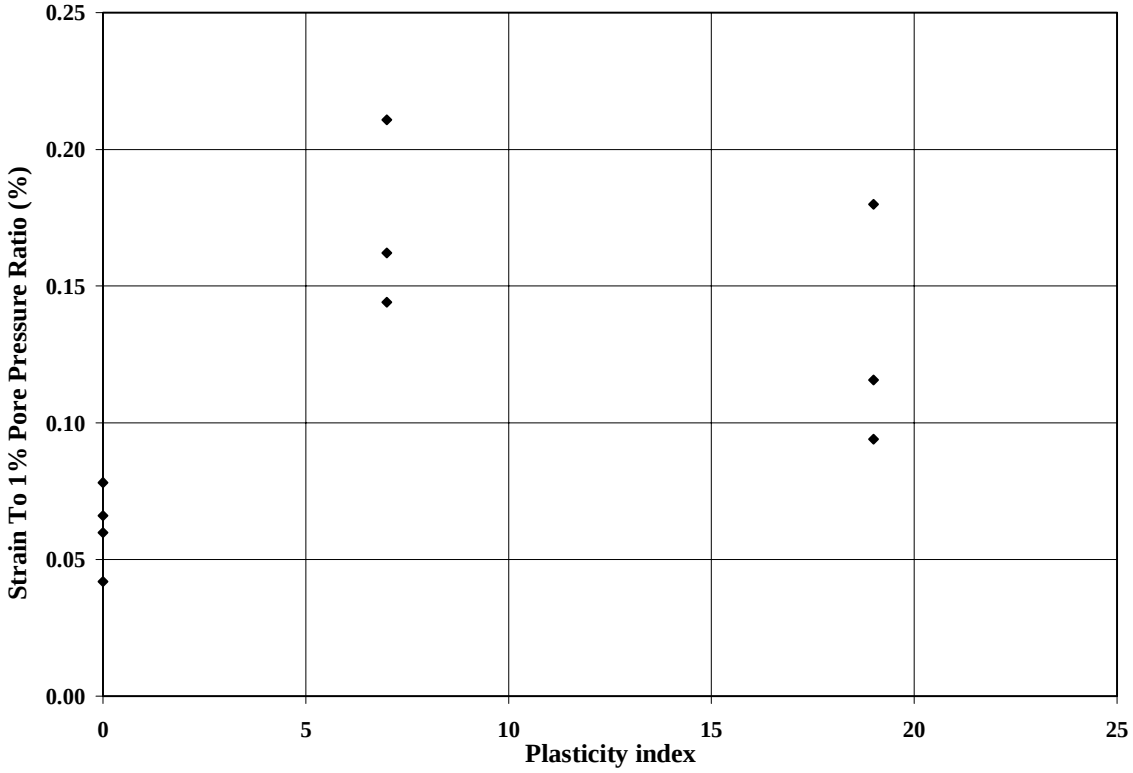

Figure 6-23: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of plasticity index for specimens of Yatesville sand with 17 percent plastic fines

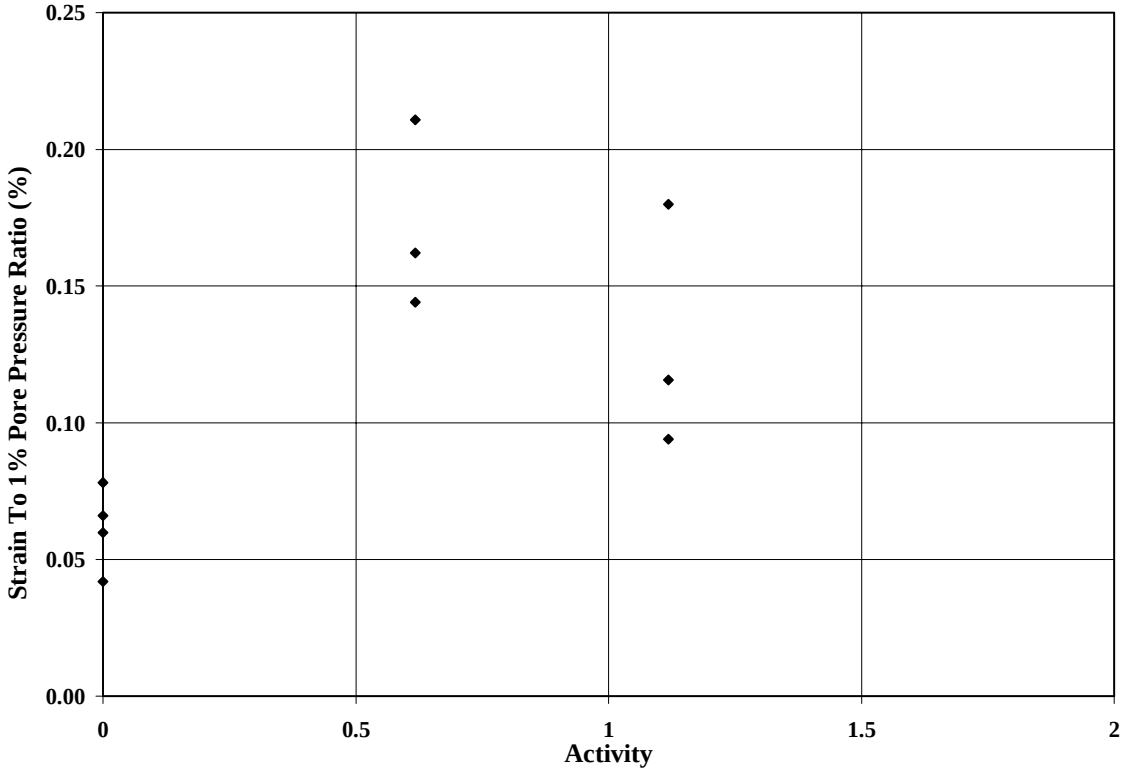

Figure 6-24: Variation in the strain required to achieve one percent residual pore pressure ratio as a function of activity for specimens of Yatesville sand with 17 percent plastic fines

Chapter 7: Implications Of Findings

The findings of the current study were examined to determine what effect they might have on the methods of liquefaction analysis currently used in engineering practice. The examination focused on the most commonly used method of liquefaction analysis, the simplified method. The implications of the findings on pore pressure generation were also examined.

7.1 Implications Of Findings About Sands With Non-Plastic Fines

During this research it was found that the cyclic resistance of sands with non-plastic fines can be divided into two categories depending whether the silt content is above or below the limiting silt content. The cyclic resistance of soils below the limiting silt content is controlled by the soil specific relative density of the soil, while the cyclic resistance of soils above the limiting silt content is controlled by the silt fraction void ratio of the soil. The difference in behavior has a possible influence on the methodology commonly used to assess the liquefaction susceptibility of sand soils. Following a brief review of the simplified procedure used to analyze liquefaction susceptibility, the results of the current study will be used to examine possible corrections to and recommendations for the methodology for soils both above and below the limiting silt content.

7.1.1 The Simplified Procedure

The “simplified method” of liquefaction analysis, which was first introduced by Seed and Idriss (1971) in the early 1970’s, compares the expected level of loading measured by a cyclic stress ratio against the cyclic resistance of the soil. The cyclic resistance of the soil is determined as function of either SPT blowcount or CPT tip resistance and fines content. These parameters are plotted together in the form of a chart that is divided into

liquefiable and non-liquefiable zones. An example of this chart may be seen in Figure 7-1.

The cyclic stress ratio is the ratio of the shear stress applied by the earthquake to the vertical effective confining stress acting on the soil element. In the simplified method, the CSR is taken as the product of the effective acceleration applied by the earthquake (commonly taken as 65 percent of the peak acceleration measured as a fraction of gravity), the ratio of total to effective vertical stresses acting on the soil element, and a reduction factor, r_d. This factor, r_d, which is necessary because the soil mass is not a rigid body, decreases from a value of one at the ground surface to a value of 0.9 at a depth of approximately 30 feet.

The penetration resistance is normally corrected to an overburden pressure of one ton per square foot, and in the case of the SPT blowcount, to a hammer energy of 60 percent. Other corrections made to the SPT blowcount include corrections for borehole diameter, rod length, and the presence or absence of a sampler liner.

The cyclic resistance of the soil is determined by entering the chart at the corrected penetration resistance, going to the appropriate curve based on the fines content of the soil and then reading the corresponding cyclic resistance off the vertical axis. Some charts do not have multiple curves for the various fines contents but employ correction factors for fines content.

For more information on the details of the simplified method and the corrections applied within it, the reader is directed to “The Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils” edited by Youd and Idriss (1997).

7.1.2 Implications of Findings On Soils Below The Limiting Silt Content

The cyclic resistance of soils with non-plastic fines below the limiting silt content has been shown to be solely a function of the soil specific relative density and to be independent of silt content. This may be seen in Figure 4- 12. Because the cyclic resistance is independent of silt content for these soils, the differences which occur in the cyclic resistance versus penetration resistance curves with fines content is due solely to the differences in penetration resistance caused by the presence of the silty.

The presence of silt has long been known to lower the penetration of a loose sand because it prevents full drainage of the soil. In order to correct for this lowering of the penetration resistance, some correction factor must be applied to the penetration resistance in order to make the use of the clean sands curve applicable. Because this correction has been shown to be correcting only for penetration resistance and not for cyclic resistance, the appropriate fines correction factors will cause the cyclic resistance curves for different fines contents to collapse on top of the clean sands curve. This, of course, is only the case when the soil under investigation has a limiting silt content of greater than the non-plastic fines content for which the cyclic resistance curve was intended.

Different fines correction factors have been suggested for analyses based upon the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT).

7.1.2.1 Analyses Using The Standard Penetration Test

Several proposed fines corrections were examined. First, the initial Seed et al (1983) correction of simply adding 7.5 to the measured blowcount was examined. As may be seen in Figure 7-2, this simple correction tends to be conservative, predicting cyclic resistances which are too low at all adjusted blowcounts for soils with 15 percent fines

and at adjusted blowcounts less than 20 for soils with 35 percent fines. It is unconservative for soils with 35 percent fines and adjusted blowcounts greater than 20.

Next, a new blowcount correction factor proposed by Robertson and Wride (1997) at the recent NCEER workshop was investigated. The factor varies as a linear function of fines content. Details of the factor are provided in Table 7-1.

The correction factor was applied to the standard curves and the results plotted in Figure 7-3. The correction is unconservative at blowcounts below about 20, because it predicts a larger cyclic resistance than may be available. This is dangerous as this is the range in which liquefaction is most likely. Above a blowcount of 20, the correction factor for 15 percent fines is rather conservative, while it works well for blowcounts above approximately 25 for the 35 percent fines curve provided the soil is below the limiting silt content.

Lastly, the fines correction factors recently recommended by NCEER (Youd and Idriss, 1997) were examined. These factors are a function of the fines content and consist of two portions, α and β. Details of these factors are presented in Table 7-1. These corrections were applied to the standard curves and the results plotted in Figure 7-4. The correction factors for 35 percent fines is accurate because it causes the 35 percent fines and clean sand curves to overlap for their full lengths. The 15 percent fines correction is unconservative because for a given adjusted blowcount it assigns a larger cyclic resistance than is assigned by the clean sand curve.

A new term is proposed to adjust the latest NCEER curves so that they become coincidental with the clean sands curve when the fines corrections are applied. This term, γ, is to be used in conjunction with the NCEER α and β factors. It is a function of both the fines content and the corrected SPT blow count, $N_{1,60}$, and is detailed in Table 7-1.

The proposed correction factors were applied to the standard curves and the results plotted in Figure 7-5. The cyclic resistance curves for 15 and 35 percent fines have converged onto the clean sand curve. This indicates that, when applied to soils with silt contents below the limiting silt content, for soils an identical cyclic resistance will be determined regardless of the silt content. This reflects the fact that, for soils below the limiting silt content, the cyclic resistance of these soils is independent of their silt contents, and is only a function of their soil specific relative density.

7.1.2.2 Analyses Using The Cone Penetration Test

In recent years the performing of the simplified procedure using data from cone penetration tests has become more popular as the data base of case histories has increased. Cone penetration testing has the advantage of providing a continuous record of penetration resistance versus depth, which allows for the detection of thin liquefiable layers which might be missed during a SPT investigation.

The procedure for performing the simplified method based upon CPT results recommended in the recent NCEER proceedings (Youd and Idriss, 1997) involves correcting the penetration resistance for fines based upon the soil behavior type index, I_C, proposed by Robertson and Wride (1997). The soil behavior type index is a function of the determined based upon the tip resistance and normalized friction ratio recorded during the cone penetration test.

Unfortunately, as actual cone testing was not performed during this study, it is not possible to evaluate the accuracy of Robertson and Wride's (1997) procedure for fines correction. If accurate, it should provide constant corrected penetration resistances for soils with constant soil specific relative densities below the limiting silt content. This is a possible area of future study.

7.1.3 Implications Of Findings On Soils Above The Limiting Silt Content

For soils above the limiting silt content, a large decrease in cyclic resistance has been seen in this study. Figure 7-6 plots the cyclic resistance curves for Monterey sand with 25 percent silt and Monterey sand with 35 percent silt adjusted to a soil specific relative density of 50 percent. These silt contents represent conditions just above and below the limiting silt content of 32 percent calculated for this sand. It may be seen in the figure that in crossing the limiting silt the cyclic resistance decreases by approximately 300 percent from 0.24 to 0.08 when evaluated at 10 cycles.

If the 35 percent fines curve or the correction factors are applied to soils with limiting silt contents below 35 percent, the predicted cyclic resistance may be grossly overpredicted. This represents a very dangerous situation that may also occur in soils with fines contents above 35 percent, if they are above their limiting silt content. The adequacy of the fines corrections can not be fully evaluated for these situations as no penetration testing was done on these soils.

In order to evaluate the typical range of limiting silt contents found in soils, a brief analysis using 37 sands and 5 silts was performed. The details on the silts and sands are provided in Appendix D: Limiting Silt Content. Figure 7-7 provides a histogram of the distribution of limiting silt contents. From the histogram it can be seen that approximately 15 percent of the sand and silt combinations examined had limiting silt contents below 35 percent.

While it is possible to evaluate the limiting silt content of a soil based upon the results of index tests upon it, the development of correlations between limiting silt content and such parameters as fines content or mean grain size offer another area for future research. Additionally, the evaluation of the liquefaction resistance of soils with fines contents

above the limiting silt content is another possible avenue of research. Whether the it is possible to correct the penetration resistance using some fines correction or whether actual laboratory testing is required, should be examined.

Appendix D also provides information on the calculation of the limiting silt content based on index data.

7.2 Implications Of Findings About Sands With Plastic Fines

It has been shown in this research that, in contrast to much of the data shown in the literature, the addition of plastic fines may increase the susceptibility of a sand to liquefaction. This may be seen in Figure 7-8 which shows the difference in liquefaction resistance which occurs between a sand with 17 percent silt and the same sand with 17 percent kaolinite when both are adjusted to a soil specific relative density of 25 percent.

The methods currently recommended by NCEER either do not account for the plasticity of the fines (in the case of the SPT method) or account for it by its affect on the resistances measured during a cone penetration test. As actual cone penetration records are not available it is impossible to evaluate the manner in which the plasticity of the fines is taken into account. This is another possible area for further research.

For the SPT-based simplified method, the current NCEER guidelines do nothing to account for the presence of plastic fines. The use of the current fines corrections for the two soils shown in Figure 7-8 would produce the same cyclic resistance at the same uncorrected blowcount. This would clearly be unconservative for the sand with 17 percent kaolinite as it would produce a cyclic resistance approximately 50 percent too large if evaluated using the curves based on non-plastic fines.

Because the curves currently used in the simplified procedure are not accurate for sands with plastic fines, an alternative method of evaluation should be used until adequate data is available to make accurate fines corrections. A plasticity based liquefaction criteria is therefore recommended. Based on the results of this study, the chart shown in Figure 7-9 is recommended for the evaluation of the liquefaction of sands with plastic fines. Soils plotting outside the zone of potentially liquefiable soils (i.e. those with plasticity indexes larger than 10, or liquid limits greater than 35) may be deemed not susceptible to flow liquefaction, but only susceptible to cyclic mobility. Soils plotting with the zone of potentially liquefiable soils may be considered susceptible to flow liquefaction, and should be the subjected to a laboratory testing program to determine their actual liquefaction resistance.

7.3 Implications Of Findings About Pore Pressure Generation

For soils with non-plastic fines, the generation of pore pressures was found to be relatively insensitive to relative density, cyclic stress ratio, or silt content when plotted in terms of pore pressure ratio versus the loading ratio. Therefore, if laboratory data is available to plot such a curve, it may be used to predict the level of pore pressure generated in a soil mass if liquefaction does not occur. Separate curves are however necessary for soils that are susceptible to flow liquefaction and for those which are susceptible to cyclic mobility.

For soils with plastic fines, the generation of pore pressures was found to be dependent on the plasticity of the soil when plotted in terms of pore pressure ratio versus the loading ratio. Therefore, the laboratory data may only be used to predict the level of pore pressure generated in a soil mass if laboratory data is available for that specific soil.7.4

7.4 Conclusions

The findings of the current study were examined to determine what effect they might have on the methods of liquefaction analysis currently used in engineering practice. The examination focused on the most commonly used method of liquefaction analysis, the simplified method.

It was first shown that for soils below the limiting silt content a constant cyclic resistance should be obtained for a constant soil specific relative density when a proper fines correction is applied. This is because the cyclic resistance below the limiting silt content is independent of the silt content. Thus the separate curves for different fines content are a result of the differences in penetration resistance brought about by the varying silt content, not a difference in liquefaction resistance.

For soils with non-plastic fines, the fines correction factors currently used when the simplified method is implemented using the results of standard penetration tests were found to unconservative and were adjusted so as to produce a constant cyclic resistance with a constant soil specific relative density.

No conclusions could be draw regarding the validity of analyses performed using the data from cone penetration tests as no cone penetration test data was available for the soils examined in this study.

The fines corrections currently recommended by NCEER (Youd and Idriss, 1997) were deemed unconservative for soils with silt content above the limiting silt content. These soils are known to have cyclic resistances lower than similar soils below the limiting silt content. However as no penetration testing was performed, it was not possible to make conclusive statements regarding the fines corrections.

For soils with plastic fines, the simplified procedure will not give accurate results with the fines corrections currently in use. For this reason a plasticity based liquefaction criteria was suggested. Soils meeting the criteria are assumed to be not susceptible to flow liquefaction, but only to cyclic mobility. Soils not meeting the criteria are assumed to be susceptible to flow liquefaction, and should have their liquefaction resistance evaluated using laboratory testing.

The pore pressure generation data developed during the laboratory testing portion of a study may be used to determine pore pressure development in soils not reaching full liquefaction. For clean sands and sands with non-plastic fines, separate curves are required for soils which are susceptible to flow liquefaction, and those that are susceptible to cyclic mobility. For sands with plastic fines separate curves are required for each specific soil due to the variation in pore pressure development with changing soil plasticity.

Table 7-1: Review of fines correction factors

Source	Fines Correction	Applicable Range
Seed et al. (1983)	$(N_1)_{60CS} = N_{1,60}+7.5$	
Robertson and Wride (1997)	$(N_1)_{60CS} = K_s(N_1)_{60}$ Where: $K_s = 1 + [(0.75/30)(FC-5)]$	
NCEER Youd and Idriss (1997)	$(N_1)_{60CS} = \alpha + \beta(N_1)_{60}$ Where: $\alpha = 0$ $\alpha = \exp[1.76-(190/FC^2)]$ $\alpha = 5.0$ $\beta = 1.0$ $\beta = [0.99+(FC^{1.5}/1000)]$ $\beta = 1.2$	 For FC < 5% For 5% < FC < 35% For FC > 35% For FC < 5% For 5% < FC < 35% For FC > 35%
Polito	$(N_1)_{60CS} = \alpha + \beta(N_1)_{60} + \gamma$ Where: α and β as for NCEER $\gamma = 0.1 + 0.008((N_1)_{60}-4)(FC-5)$	 For all FC and $(N_1)_{60}$

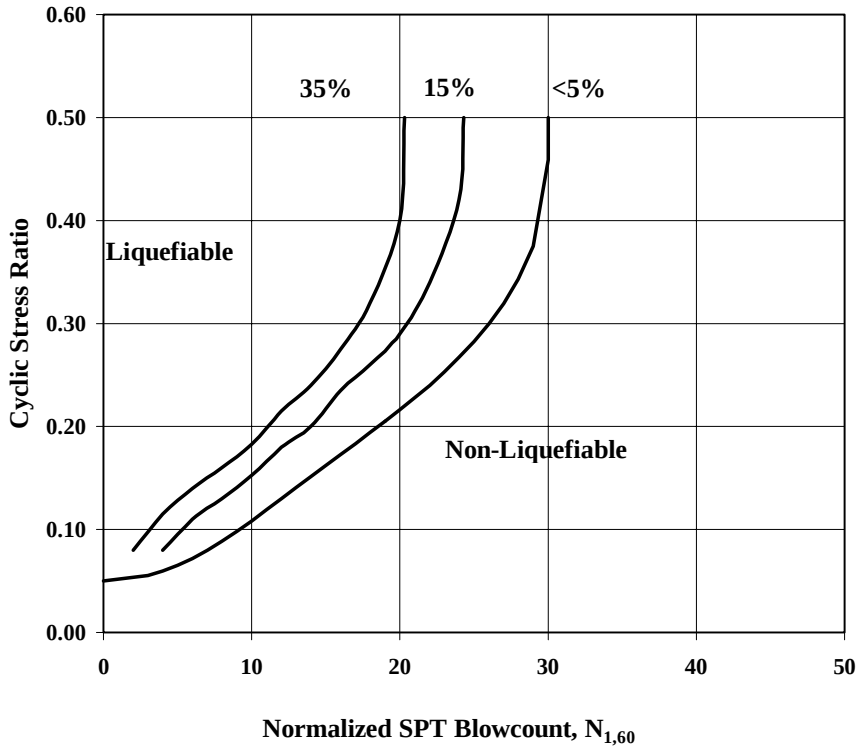

Figure 7-1: Liquefaction chart for the simplified procedure

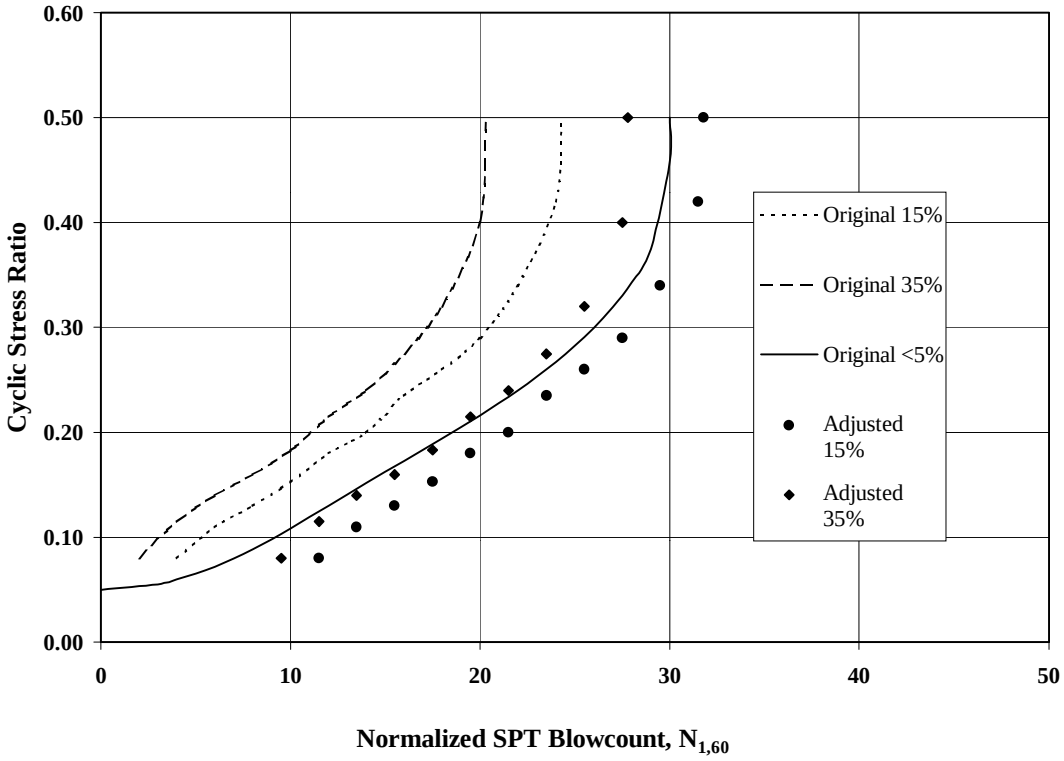

Figure 7-2: Cyclic resistance curves based on Seed et al.'s (1983) correction

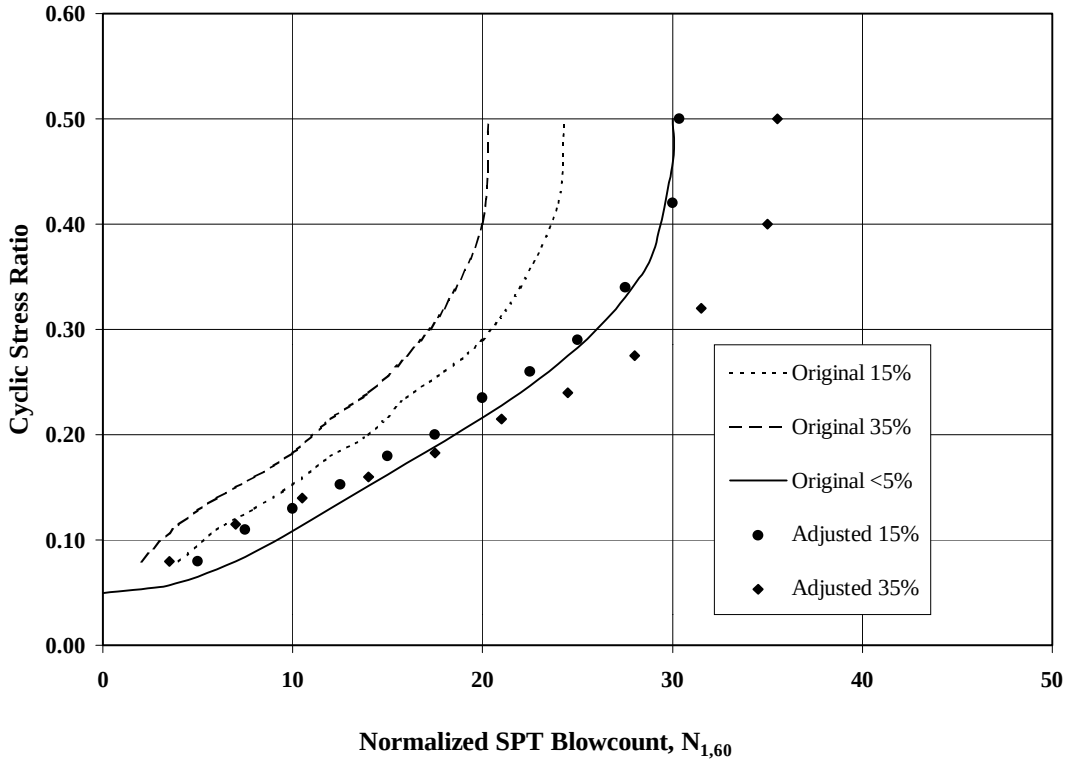

Figure 7-3: Cyclic resistance curves based on Robertson and Wride's (1997) correction

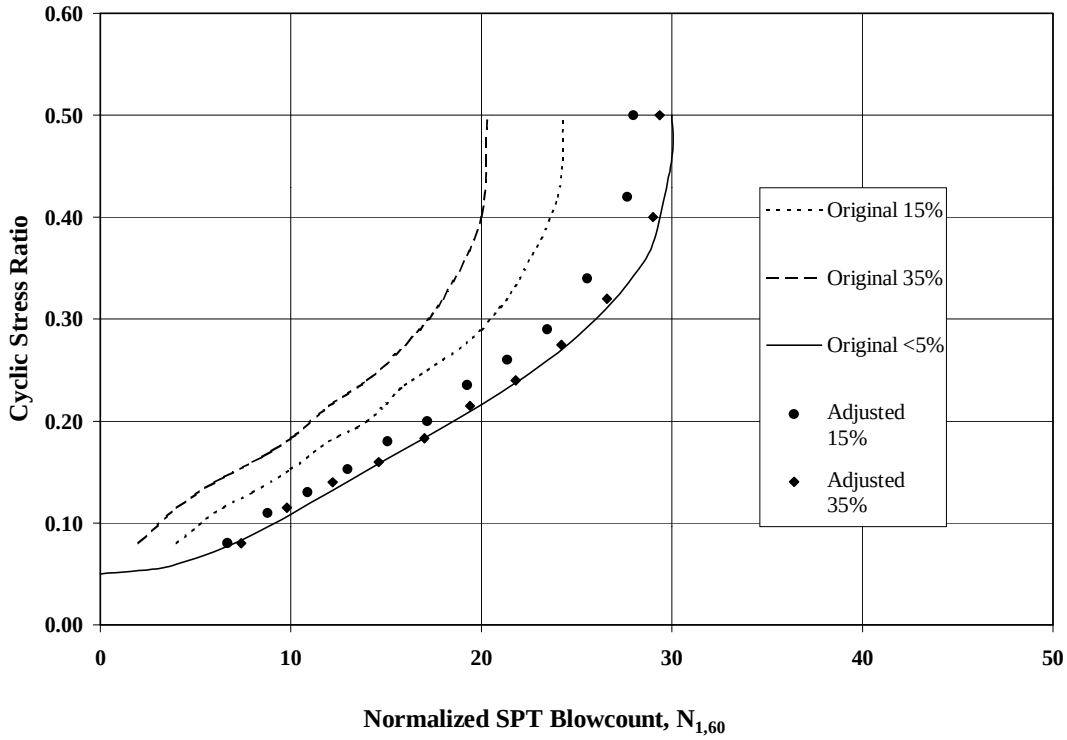

Figure 7-4: Cyclic resistance curves based on NCEER (1997) correction

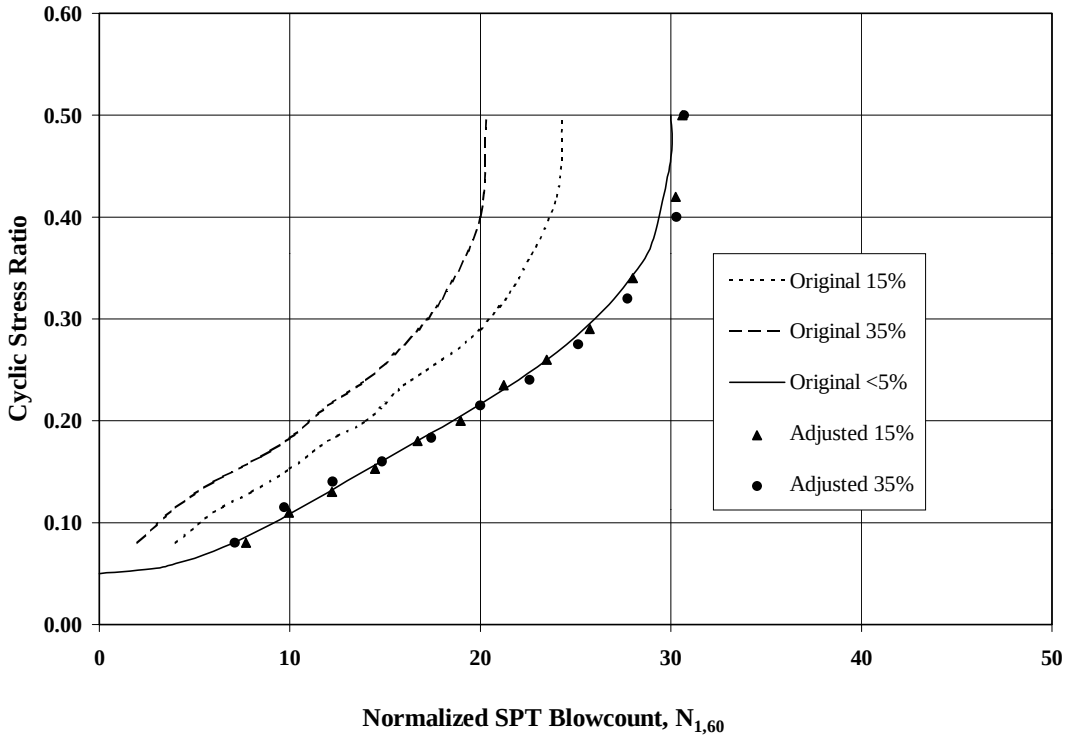

Figure 7-5: Cyclic resistance curves based on new, recommended correction

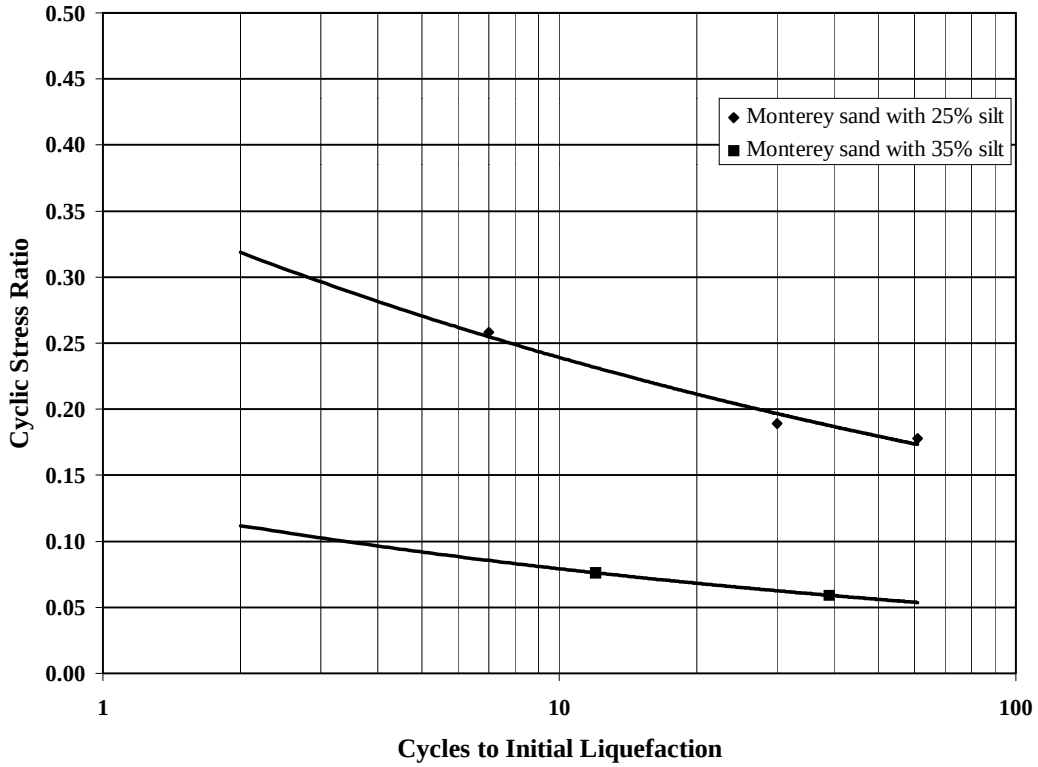

Figure 7-6: Variation in cyclic resistance above and below the limiting silt content

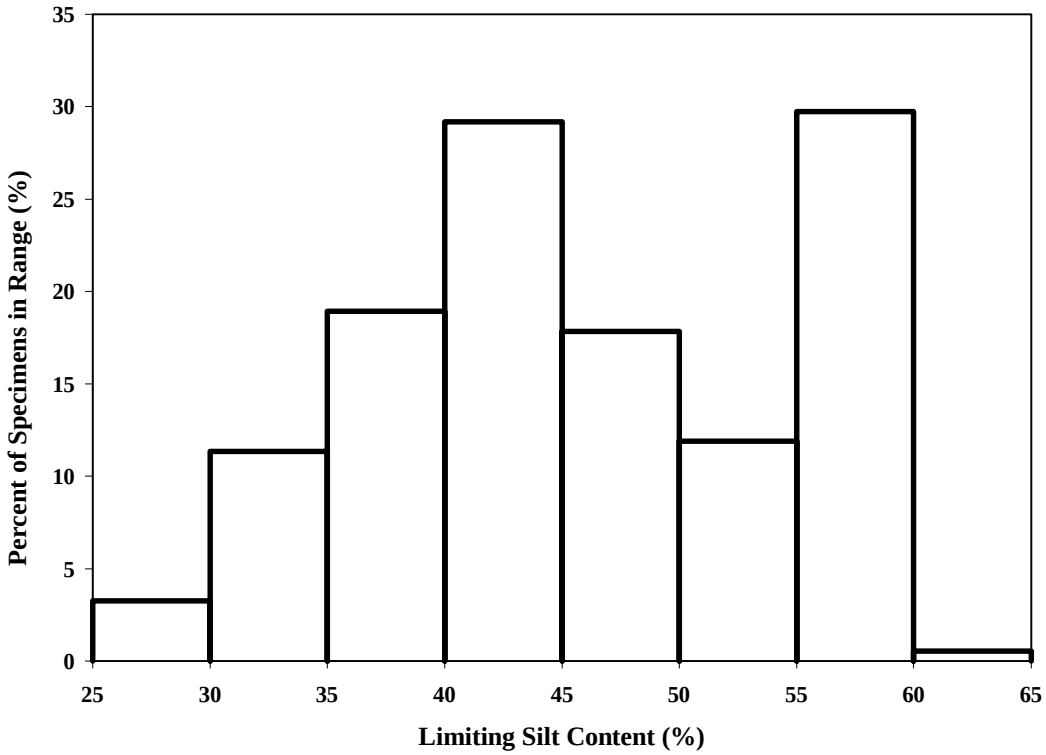

Figure 7-7: Distribution of limiting silt contents

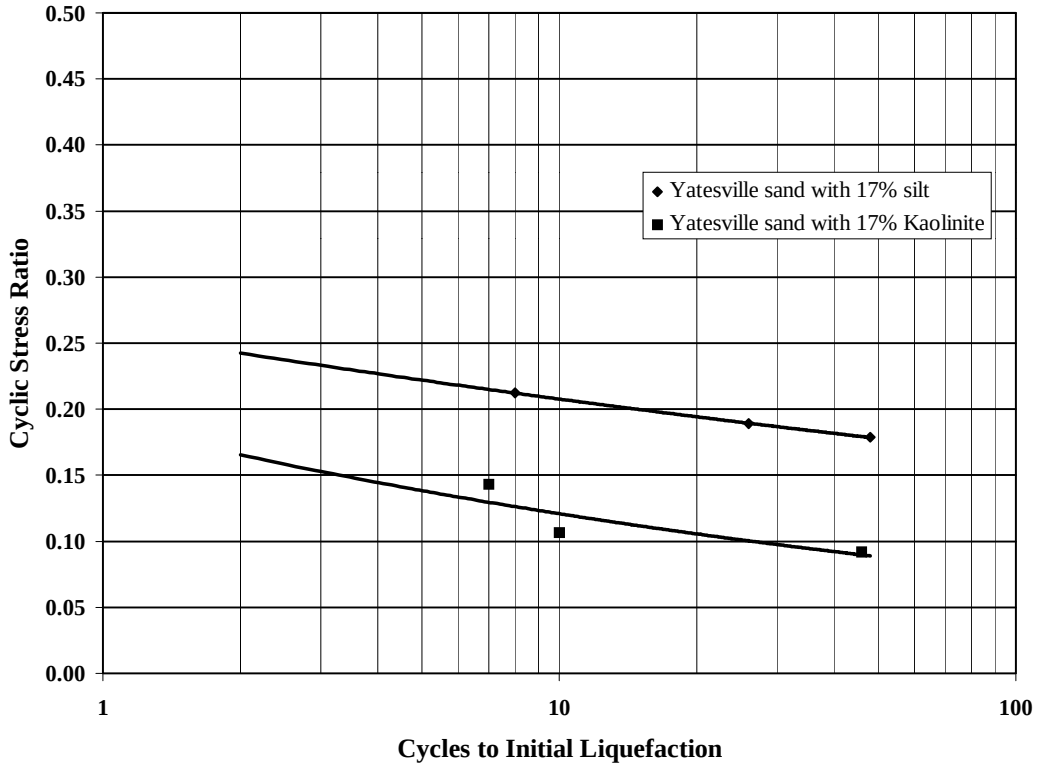

Figure 7-8: Variation in cyclic resistance with fines composition

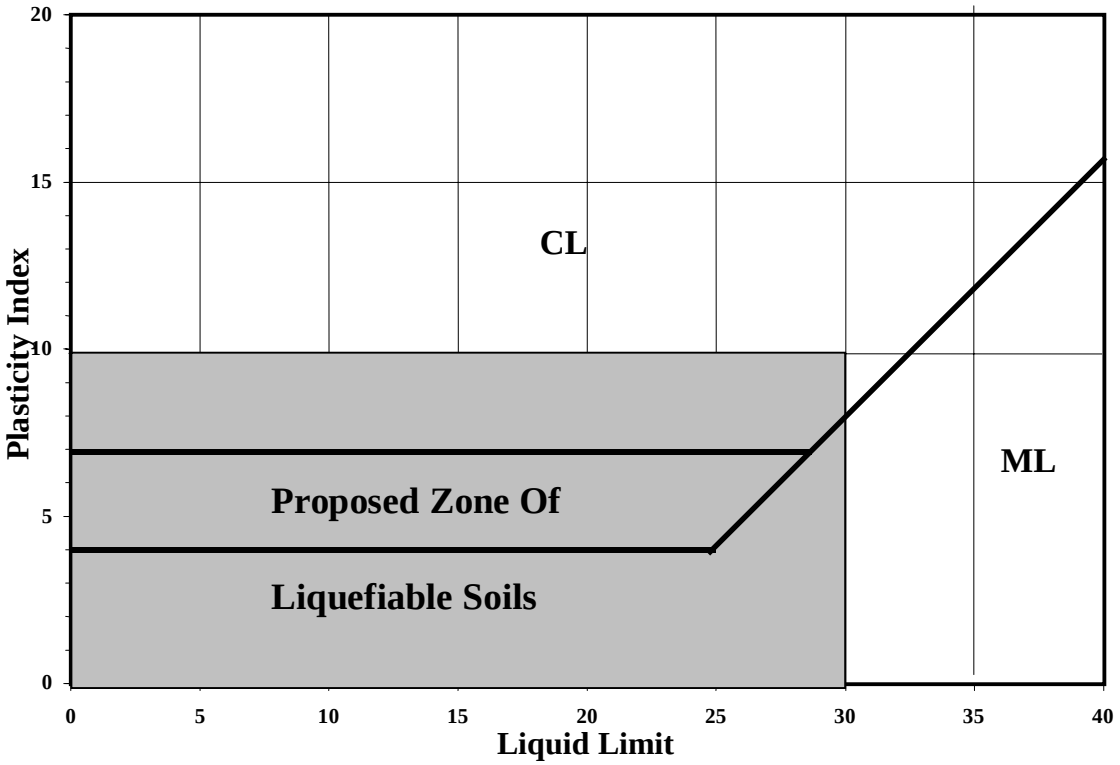

Figure 7-9: Proposed zone of liquefiable soils

Chapter 8: Conclusions and Recommendations

A laboratory parametric study intended to clarify the effects which varying fines content and plasticity have upon the liquefaction resistance of sandy sands was undertaken using cyclic triaxial tests. The program of research also examined the applicability of plasticity based liquefaction criteria and the effects of fines content and plasticity on pore pressure generation. Lastly, a review was made of how the findings of this study may affect the manner in which simplified analyses are performed in engineering practice.

The conclusions drawn from this study and recommendations with regard to implementing these findings into engineering practice are summarized below.

8.1 Effects Of Non-Plastic Fines

The understanding of the liquefaction of sands containing non-plastic, fine-grained material (i.e. silt) is less complete than the understanding of liquefaction in clean sands. A review of the literature shows that there is no clear consensus as to what effect an increase in non-plastic silt content has upon the liquefaction resistance of a sand. In fact, seemingly contradictory behaviors are reported.

The current study was able to isolate two factors, soils specific relative density and limiting silt content, which govern the liquefaction behavior of silty sands and sandy silts. Using these factors, the majority of the seemingly contradictory behaviors reported in the literature can be explained.

Several conclusions regarding the effects of non-plastic fines on the liquefaction susceptibility of sandy soils were drawn from this study. These findings are summarized below:

- Three distinct behavioral patterns were found for the cyclic resistance of soils composed of sand and non-plastic silt. These three patterns may be seen in Figure 8-1. Which of these behaviors controls is determined by whether there is sufficient room in the voids created by the sand skeleton to contain the silt present without disturbing the sand structure. This silt content has been called the limiting silt content.

- If the silt content of the soil is below the limiting silt content, there is sufficient room in the voids created by the sand skeleton to contain the silt, and the soil can be described as having silt contained in a sand matrix. The cyclic resistance of the soil is then controlled by the soil specific relative density of the specimen, where the soil specific relative density is calculated using the gross void ratio of the specimen and the maximum and minimum index void ratios for that particular mixture of sand and silt. Increasing the soil specific relative density increases the soil's cyclic resistance.

- If the silt content is greater than the limiting silt content, the specimen's structure consists predominately of sand grains suspended within a silt matrix with little sand grain to sand grain contact. Above the limiting silt content, the amounts of sand present in the soil and its soil specific relative density have little effect on its cyclic resistance. The cyclic resistance for these soils is controlled by silt fraction void ratio of the soil. Decreasing the silt fraction void ratio increases the soil's cyclic resistance.

- There is a transition zone consisting of soils with silt contents at or slightly above the limiting silt content. This zone occurs as a result of the structure of the soil changing from a predominately sand controlled fabric to a predominately silt controlled fabric as silt content increases. Soils in this zone possess cyclic resistances intermediate to those with greater or lesser silt contents.

- The seemingly contradictory reports concerning the effects of non-plastic fines content on the liquefaction resistance of sands that have appeared in the literature were reconciled in light of the behavioral patterns found by the authors. These behaviors were found to depend primarily upon the limiting silt content of the soil and the soil specific relative density of the specimen. It was shown that the trends of decreasing cyclic resistance or decreasing and then increasing cyclic resistance with increases in silt content reported in the literature can be explained in view of the results of this study. The concept that cyclic resistance is controlled by the sand skeleton void ratio of the soil was also reconciled with results of this study. The reports of increasing cyclic resistance with increasing silt content reported in the literature can not be explained based upon the findings of this study.

8.2 Effects Of Plastic Fines And Plasticity Based Liquefaction Criteria

The presence of plastic or clayey fines is generally considered to decrease the liquefaction susceptibility of a soil. Numerous field studies have shown that soils with more than 10 or 20 percent fines tend not to liquefy during earthquakes. Several investigators have proposed criteria by which to deem certain soils as “non-liquefiable” based primarily on their plastic nature. How the introduction of clayey fines actually affects the liquefaction susceptibility of a soil and the validity of the various plasticity based liquefaction criteria formed a major portion of this research.

The effects of plastic fines content were analyzed by testing a series of specimens prepared to a constant soil specific relative density with various fines content and composition. The effects of fines plasticity were isolated by testing a series of specimens prepared to a constant fines content and soil specific relative density with varying fines composition and plasticity.

Several conclusions regarding the effects of plastic fines on the liquefaction susceptibility of sandy soils and the validity of plasticity based liquefaction criteria were drawn from this study. These findings are summarized below:

- The cyclic resistance of sands with plastic fines appears to be relatively independent of fines content, clay content, water content, and liquidity index.

- The cyclic resistance of sands with plastic fines increases with increases in liquid limit, plasticity index and activity.

- The cyclic resistance of sands with plastic fines is dependent on the plasticity of the soil whether that plasticity is measured in terms of liquid limit, plasticity index, or activity. It is important to determine the plasticity of the soil mixture and not just the plasticity of the fines.

- If the plasticity of the fines is low, the addition of even a moderate amount of plastic fines (up to 25 percent) may decrease the liquefaction resistance of the soil.

- The key factor in the plasticity based liquefaction criteria is the requirement for the plasticity of the soil. If this requirement, whether measured in terms of liquid limit or plasticity index, is met, the soil does not appear to be susceptible to flow liquefaction.

- The reason plasticity based criteria work is that they mirror a change in behavior in the soil following the onset of initial liquefaction. Soils meeting the plasticity requirement tend to undergo a cyclic mobility form of failure that results in only minor strength loss and small deformations of the soils mass. Soils not meeting the plasticity requirement tend to undergo flow liquefaction that results in significant strength loss and very large deformations.

- Although zero effective stress and large values of strain may be achieved in a laboratory test for soils undergoing cyclic mobility, these behaviors are transient and the soil does not behave in any of the manners commonly associated with flow liquefaction, such as significant strength loss or large deformations.

- The differences in behavior between flow liquefaction and cyclic mobility make it inappropriate to evaluate liquefaction for soils meeting the plasticity requirements using either initial liquefaction or some level of strain measured in a lab test.

- A plasticity index of 10 appears to be a reasonable threshold value between flow liquefaction and cyclic mobility, while the liquid limit values in the range of 34 to 36 which have been suggested, appear rather conservative. A liquid limit value of 25 appears to ensure that a soil will fail in cyclic mobility rather than in flow liquefaction.

- A proposed zone of liquefiable soils is indicated on the plasticity chart shown in Figure 8-2, and includes soils with plasticity indexes less than 10 and liquid limits less than 35. Soils that plot in this region and contain plastic fines should be tested to determine their liquefaction susceptibility.

- This tendency towards cyclic mobility also explains why soils meeting the plasticity based liquefaction criteria are frequently reported during field investigations as being non-liquefiable. While they may develop a zero effective stress condition, their lack of deformation and the absence of sand boil emission tend to hide this occurrence.

8.3 Pore Pressure Generation

Data from the cyclic triaxial tests were analyzed to evaluate the effects of fines content and plasticity on the pore pressure generation characteristics of silty and clayey sands. Pore pressure generation was evaluated both in terms of the number of cycles of loading relative to the number of cycles of loading required to achieve initial liquefaction and in terms of the amount of strain require to achieve a certain level of pore pressure ratio.

Several conclusions regarding the effects of fines content and plasticity on pore pressure generation were drawn from this study. These findings are summarized below:

- The loading ratio is defined as the number of cycles of loading relative to the number of cycles of loading required to achieve initial liquefaction

- When analyzed in terms of loading ratio, pore pressure generation for each sand with silt was found to fit into a narrow band. The shape and position of that band depends on whether the soils is susceptible to flow liquefaction or cyclic mobility, and is independent of silt content.

- At any given loading ratio the pore pressure ratio is larger for a soil susceptible to cyclic mobility than it is for a soil susceptible to flow liquefaction.

- When analyzed in terms of the loading ratio, the pore pressure ratio for sands with plastic fines varies as the plasticity of the soils varies. Soils with low plasticity produced pore pressure generation curves similar to those produced for soils with non-plastic fines susceptible to flow liquefaction. Soils with high plasticity produced

pore pressure generation curves similar to those produced for soils with non-plastic fines susceptible to cyclic mobility.

- Because the pore pressures generated by these soils tend to fall into narrow bands when plotted using the loading ratio, these curves may be used to estimate the pore pressures generated in these soils if liquefaction does not occur. This analysis requires knowledge of the number of cycles required to liquefy the soil at the stress levels imposed by the design earthquake and the number of cycles in the design earthquake.

- The residual pore pressure ratio is the pore pressure ratio measured during the point of zero deviator stress

- When evaluated in terms of the strain required to reach some level of residual pore pressure ratio, the behavior for soils with non-plastic fines is governed by the soil's silt content relative to the limiting silt content.

- For soils with non-plastic fines below the limiting silt content the strain required to generate pore pressures is related to soil specific relative density and the pore pressure level. For one percent residual pore pressure ratio, the strain required increases with increasing soil specific relative density. Conversely, for seventy percent residual pore pressure ratio, the strain required decreases with increasing soil specific relative density.

- For soils with non-plastic fines above the limiting silt content the strain required to generate some level of residual pore pressure ratio is related to the silt fraction void ratio. The strain required decreases with increasing silt fraction void ratio.

- When analyzed in terms of the strain required to achieve a specified level of residual pore pressure ratio, pore pressure generation in sands with plastic fines is independent of fines content and clay content. The strain required increases with increasing liquid limit, plasticity index, and activity.

8.4 Implications Of Research To Current Practice

The findings of the current study were examined to determine what effect they might have on the methods of liquefaction analysis currently used in engineering practice. The examination focused on the most commonly used method of liquefaction analysis, the simplified method. These findings are summarized below:

- For soils below the limiting silt content, a constant cyclic resistance should be obtained for a constant soil specific relative density when a proper fines correction is applied. This is because the cyclic resistance below the limiting silt content is independent of the silt content.

- The separate cyclic resistance curves used in the simplified method for different fines content are a result of the differences in penetration resistance brought about by the varying silt content, not a difference in liquefaction resistance.

- For soils with non-plastic fines below the limiting silt content, the fines correction factors currently used with the SPT-based simplified method were found to unconservative. They were adjusted to produce a constant cyclic resistance with a constant soil specific relative density.

- No conclusions could be draw regarding the validity of analyses performed using the data from cone penetration tests as no cone penetration test data was available for the soils examined in this study.

- The fines corrections currently used were deemed potentially unconservative for soils with silt content above the limiting silt content. These soils are known to have cyclic resistances lower than similar soils below the limiting silt content, however as no penetration testing was performed, it was not possible to make concrete conclusions regarding the fines corrections.

- A small parametric study using data for 37 sands and 5 silts showed that approximately 15 percent of the sands and silt combinations investigated had limiting silt contents below 35 percent.

- For soils with plastic fines, the simplified procedure will not give accurate results with the fines corrections currently in use.

- A plasticity based liquefaction criteria was suggested. It may be seen in Figure 8-2.

- Soils meeting the proposed criteria of either a plasticity index greater than 10 or a liquid limit greater than 25 appear to be susceptible to cyclic mobility but not to flow liquefaction.

- Soils not meeting the proposed criteria appear to be susceptible to flow liquefaction, and should have their liquefaction resistance evaluated using laboratory testing.

- The pore pressure generation data developed during the laboratory testing portion of a study may be used to determine pore pressure development in soils not reaching full liquefaction.

- For clean sands and sands with non-plastic fines, separate curves are required for soils which are susceptible to flow liquefaction, and those which are susceptible to cyclic mobility.

- For sands with plastic fines separate curves are required for each specific soil due to the variation in pore pressure development with changing soil plasticity.

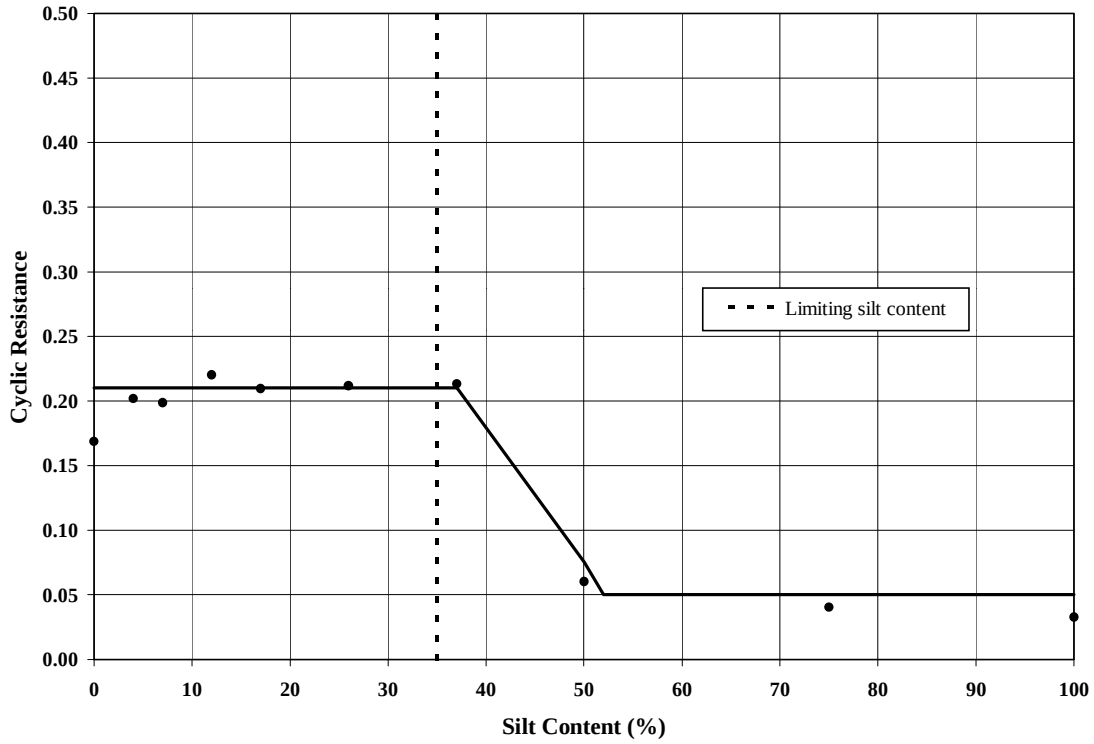

Figure 8-1: Variation in cyclic resistance with silt content for Yatesville sand specimens adjusted to 25% soil specific relative density

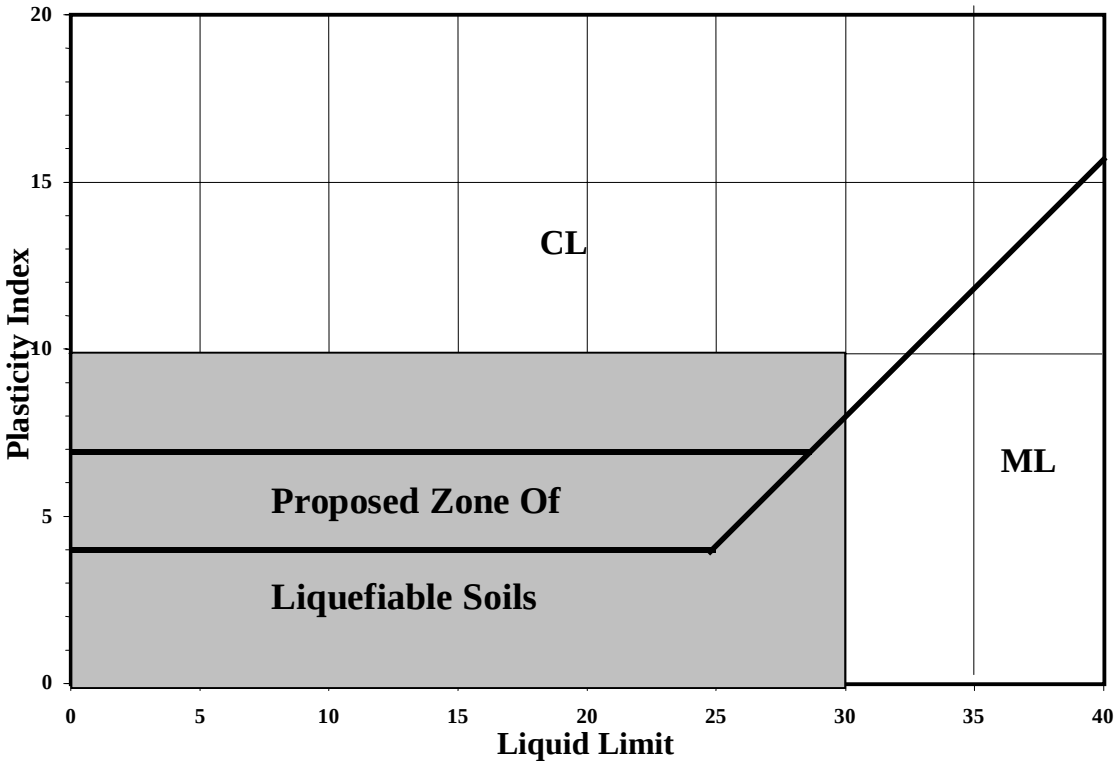

Figure 8-2: Proposed zone of liquefiable soils

References

Arman, A. and Thorton, S.I. (1973) “Identification Of Collapsible Soils In Louisiana” Highway Research Record Number 426, Soil Classification, Highway Research Board, National Research Council, Washington D.C.

Casagrande, A., (1975) “Liquefaction And Cyclic Mobility Of Sands. A Critical Review” *Proceedings of the 5th Pan American Conference on Soil Mechanics and Foundation Engineering*, Buenas Aires, Vol. 5, pp. 80-133

Castro, G. and Poulos, S.J., (1977) “Factors Affecting Liquefaction And Cyclic Mobility” *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 103(6), pp.501 - 516.

Chan, C. K., (1985) “Instruction Manual, CKC E/P Cyclic Loading Triaxial System Users Manual” Soil Engineering Equipment Company. San Francisco, CA.

Chaney, R. C., (1976) “Deformations Of Earth Dams Due To Earthquake Loadings” thesis presented to the University of California, at Los Angeles, California, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Chang, N.Y., Yeh, S.T., and Kaufman, L.P. (1982) “Liquefaction Potential Of Clean And Silty Sands” *Proceedings of the 3rd International Earthquake Microzonation Conference*, Seattle, USA, Vol. 2, pp. 1017-1032.

Clukey, E.C., Kulhawy, F.H., and Liu, P. (1983) “Laboratory and Field Investigation of Wave-Sediment Interaction” Report to National Science Foundation, Geotechnical Engineering Report 83-9, Cornell University, October.

Dezfulian, H., (1982) . “Effects of Silt Content on Dynamic Properties of Sandy Soils” *Proceedings of the Eighth World Conference on Earthquake Engineering*, San Francisco, USA, pp. 63-70.

De Alba, P., Seed,H., and Chan, C., (1976). “Sand Liquefaction In Large Simple Shear Tests” *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 102(9), pp.909-927.

Dobry, R., and Alvarez, L., (1967). “Seismic Failures Of Chilean Tailings Dams” *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 93(6), pp. 237-260.

Dobry, R., Ladd, R., Yokel, F., Chung, R., Powell, D., (1982). “Prediction Of Pore Water Pressure Buildup And Liquefaction Of Sands During Earthquakes By The Cyclic Strain Method” *NBS Building Science Series 138*, National Bureau of Standards, US Department of Commerce.

El Hosri, M. S., Biarez, J., and Hicher, P.Y., (1982). “Liquefaction Characteristics Of Silty Clay” *Proceedings. of the Eighth World Conference on Earthquake Engineering*, San Francisco, USA, pp. 277-284.

Fei, H.C. (1991). “The Characteristics of Liquefaction Of Silt Soil” Soil Dynamics and Earthquake Engineering V, Computational Mechanics Publications, Southhampton, pp. 293-302.

Finn, W.L., (1982). “Soil Liquefaction Studies In The People’s Republic of China” Soil Mechanics - Transient and Cyclic Loads, John Wiley and Sons, New York, pp. 609-626.

Finn, W.L., Ledbetter, R.H., and Wu, G., (1994). "Liquefaction In Silty Soils: Design And Analysis" Ground Failures Under Seismic Conditions, Geotechnical Special Publication No. 44, ASCE, pp. 51-76.

Florin, V.A., and Ivanov, P.L., (1961). "Liquefaction Of Saturated Sandy Soils" *Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering*, Paris, France, Vol. 1, pp.107-111.

Garga, V., and McKay, L., (1984). "Cyclic Triaxial Strength Of Mines Tailings" *Journal of Geotechnical Engineering*, ASCE, Vol. 110(8), pp.1091- 1105.

Holzer, T. L., Youd, T. L., and Hanks, T. C. (1989). "Dynamics of Liquefaction During the 1987 Superstition Hills, California, Earthquake" *Science*, Vol. 244, April 7, pp. 56-59.

Ishihara, K. (1993). "Liquefaction And Flow Failure During Earthquakes" *Géotechnique*, Vol. 43, No. 3, pp. 351-415.

Ishihara, K. (1996). "*Soil Behaviour in Earthquake Geotechnics*", 1rst ed., Oxford, Claredon Press, 350 pp.

Ishihara, K., and Koseki, (1989). "Discussion On The Cyclic Shear Strength Of Fines-Containing Sands" Earthquakes Geotechnical Engineering, *Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering*, Rio De Janiero, Brazil, pp. 101-106.

Ishihara, K., Sodewkawa, M., and Tanaka, Y. (1977). "Effects Of Overconsolidation On Liquefaction Characteristics Of Sands Containing Fines" Dynamic Geotechnical Testing, ASTM STP 654, American Society for Testing and Materials, pp. 246-264.

Jennings, P.C. (1980) *"Earthquake Engineering and Hazards Reduction in China, CSCPRC Report No. 8"*, National Academy of Sciences, Washington, D.C., 1980

Koester, J.P. (1994) "The Influence Of Fine Type And Content On Cyclic Strength" Ground Failures Under Seismic Conditions, Geotechnical Special Publication No. 44, ASCE, pp. 17-33.

Kuerbis, R., Negussey, D., and Vaid, V. P. (1988). "Effect Of Gradation And Fines Content On The Undrained Response Of Sand" *Proceedings. Hydraulic Fill Structures*, Fort Collins, USA, pp. 330-345.

Ladd, R.S., (1978). "Preparing Test Specimens Using Undercompaction" Geotechnical Testing Journal, GTJODJ, Vol. 1, No. 1, March 1978, pp.16-23.

Lade, P.V. and Lee, K.L. (1976) "Engineering Properties Of Soils, Report No. UCLA-ENG-7652" Soil Mechanics Laboratory, UCLA, Los Angeles, CA, May.

Law, K.T. and Ling, Y.H. (1992). "Liquefaction Of Granular Soils With Non-Cohesive and Cohesive Fines" Proceedings of the Tenth World Conference on Earthquake Engineering, Rotterdam, pp. 1491-1496.

Lee, K.L., and Albaisa, A., (1974). "Earthquake Induced Settlements In Saturated Sands" *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 100(4), pp. 387 - 406.

Lee, K.L., and Fitton, J.A., (1968). "Factors Affecting The Cyclic Loading Strength Of Soil," Vibration Effects of Earthquakes on Soils and Foundations, ASTM STP 450, American Society for Testing and Materials, pp. 71-95.

Lee, K.L., and Seed, H.B., (1967a). “Cyclic Stress Conditions Causing Liquefaction Of sand” *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 93, SM1, pp. 47-70.

Lee, K.L., and Seed, H.B., (1967b). “Dynamic Strength Of Anisotropically Consolidated Sand” *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 93, SM5, pp. 169-190.

Lee, K., and Singh, A (1971) "Compaction of Granular Soils" Proceedings of the 7th Annual Symposium on Engineering Geology, Boise Idaho, April 5-7, 1971

Marsuson, W.F., Hynes, M.E., and Franklin, A.G. (1990). “Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankments” *Earthquake Spectra*, EERI, Vol 6, No. 3, pp. 529-572.

Mitchell, J.K. (1993). “*Fundamentals of Soil Behavior*”, 2nd ed., New York, John Wiley & Sons, Inc., 450 pp.

Mogami, T., and Kubo, K., (1953). “The Behaviour Of Soil During Vibration” *Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 152-153.

Mulilis, J. P., (1975). “The Effect Of Sample Preparation On The Cyclic Stress-Strain Behavior Of Sands” thesis presented to the University of California, at Berkeley, California, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Mulilis, J. P., Townsend, F. C., and Horz, R. C., (1978). "Triaxial Testing Techniques And Sand Liquefaction" Dynamic Geotechnical Testing, ASTM STP 654, American Society for Testing and Materials, pp. 265-279.

Mullen, G.W., (1991) "Evaluation Of The Utility Of Four In-Situ Test Methods For Transmission Line Foundation Design" thesis presented to the Faculty of Virginia Polytechnic Institute and State University, in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

Okashi, Y. (1970) "Effects Of Sand Compaction On Liquefaction During Tokachioki Earthquake" *Soils and Foundations*, JSSMFE, Vol. 10, No. 2, pp. 112-128.

Okusa, S., Anma, S., and Maikuma, H. (1980). "Liquefaction Of Mine Tailings In The 1978 Izu-Oshima-Kinkai Earthquake, Central Japan" *Proceedings. of the Seventh World Conference on Earthquake Engineering*, Istanbul, Turkey, Vol. 3, pp. 89-96.

Polito, C.P., and Martin, J.R., (1999). "The Effects Of Non-Plastic Fines On the Liquefaction Resistance Of Sands" Currently Under Review by Journal of Geotechnical and Geoenvironmental Engineering.

Prakash, S., and Puri, V.K., (1982). "Liquefaction Of Loessial Soils" *Proceedings. of the 3rd International Earthquake Microzonation Conference*, Seattle, USA, Vol. 2, pp.1101-1107.

Robertson, P.K., and Campanella, R.G., (1985). "Liquefaction Potential Of Sands Using CPT" *Journal of Geotechnical Engineering*, ASCE, Vol. 111(3), pp. 384-403.

Robertson, P.K., and Wride, C.E.(1997). “Cyclic Liquefaction and its Evaluation based on the SPT and CPT” Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER-97-0022, December, pp. 41-87.

Seed, H.B., (1979). “Soil Liquefaction And Cyclic Mobility Evaluation For Level Ground During Earthquakes” *Journal of Geotechnical Engineering*, ASCE, Vol. 105, GT2, pp. 201-255.

Seed, H.B., (1987). “Design Problems In Soil Liquefaction” *Journal of Geotechnical Engineering*, ASCE, Vol. 113(8), pp. 827-845.

Seed, H.B., and Idriss, I.M., (1971). “Simplified Procedure For Evaluation Soil Liquefaction Potential” *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 97(9), pp.1249-1273.

Seed, H.B., Idriss, I.M., and Arango, I., (1983). “Evaluation Of Liquefaction Potential Using Field Performance Data” *Journal of Geotechnical Engineering*, ASCE, Vol. 109(3), pp. 458-482.

Seed, H.B., and Lee, K.L., (1966). “Liquefaction Of Saturated Sands During Cyclic Loading” *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 92, SM6, pp. 105-134.

Seed, H.B., Lee, K.L., Idriss, I.M., and Makdisi, F. (1973). “Analysis of the Slides in the San Fernando Dams During the Earthquake of February 9, 1971.” *Report No. UCB/ EERC 73-2*, Earthquake Engineering Research Center, University of California, Berkeley, Calif.

Seed, H.B., and Pyke, R.M., (1975). “Analysis of the Effect of Multi-Directional Shaking on the Liquefaction Characteristics of Sands During Cyclic Loading” *Report No. EERC 75-41,* Earthquake Engineering Research Center, University of California, Berkeley, Calif., Dec., 1975.

Seed, H.B., Tokimatsu, K., Harder, L., and Chung, R. (1985). “Influence Of SPT Procedures In Soil Liquefaction Resistance Evaluations” *Journal of Geotechnical Engineering*, ASCE, Vol. 111(12), pp.1425-1445.

Shen, C.K., Vrymoed, J.L., and Uyeno, C.K., (1977) “The Effects Of Fines On Liquefaction Of Sands” *Proceedings of the Ninth International Conference on Soil Mech. and Found. Eng.*, Tokyo, Japan, Vol. 2, pp.381-385.

Shibata, T., and Teparaksa, W., (1988). “Evaluation Of Liquefaction Potentials Of Soils Using Cone Penetration Tests” *Soils and Foundations*, JSSMFE, Vol. 28, No. 2, pp. 49-60.

Silver, M. L., (1977). “Laboratory Triaxial Testing Procedures To Determine The Cyclic Strength Of Soils” NUREG-0031, National Technical Information Service, Springfield, VA.

Silver, M., Chan, C., Ladd, R., Lee, K., Tiedmann, D., Townsend, F., Valera, J., Wilson, J., (1976). “Cyclic Strength Of Standard Test Sand” *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 102(5), pp. 511-523.

Singh, S., (1994). “Liquefaction Characteristics Of Silt” Ground Failures Under Seismic Conditions, Geotechnical Special Publication No. 44, ASCE, pp. 105-116.

Skempton, A.W., (1954). “The Pore-Pressure Coefficients A and B” *Géotechnique*, Vol. IV, pp.143-147.

Tatsuoka, F., Iwasaki, T., Tokida, K., Yasuda, S., Hirose, M., Imai, T., and Kon-No, M., (1980). “Standard Penetration Tests And Soil Liquefaction Potential Evaluation” *Soils and Foundations*, JSSMFE, Vol. 20, No. 4, pp. 95-111.

Tokimatsu, K., and Yoshimi, Y., (1981). “Field Correlation Of Soil Liquefaction With SPT And Grain Size” *Proceedings of the International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, Rolla, USA, Vol.1, pp. 203-208.

Tokimatsu, K., and Yoshimi, Y., (1983). “Empirical Correlation Of Soil Liquefaction Based On SPT N-Value And Fines Content” *Soils and Foundations*, JSSMFE, Vol. 23, No. 4, pp. 56-74.

Townsend, F., (1978). “A Review Of Factors Affecting Cyclic Triaxial Tests” Dynamic Geotechnical Testing, ASTM STP 654, American Society for Testing and Materials, pp. 356-383.

Tronsco, J.H., (1990). “Failure Risks Of Abandoned Tailings Dams” *Proceedings. Int. Symposium. on Safety and Rehabilitation of Tailings Dams*, CIGB ICOLD, Sydney, Australia, pp. 34-47.

Tronsco, J.H., and Verdugo, R., (1985). “Silt Content And Dynamic Behavior Of Tailing Sands” *Proceedings. Twelfth International Conference on Soil Mech. and Found. Eng.*, San Francisco, USA, pp.1311-1314.

Vaid, V.P., (1994). “Liquefaction Of Silty Soils” Ground Failures Under Seismic Conditions, Geotechnical Special Publication No. 44, ASCE, pp.1-16.

Yasuda, S., Wakamatsu, K., Nagase, H., (1994). “Liquefaction Of Artificially Filled Silty Sands” Ground Failures Under Seismic Conditions, Geotechnical Special Publication No. 44, ASCE, pp. 91-104.

Youd, L., and Bennett, M.J. (1983). “Liquefaction Sites, Imperial Valley, California” *Journal of Geotechnical Engineering*, ASCE, Vol. 109(3), pp. 440-457.

Youd, L., and Idriss, I. M.(1997). “Summary Report” Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER-97-0022, December pp. 1-40.

Wang, W. (1979). “Some Findings in Soil Liquefaction” Water Conservancy and Hydroelectric Power Scientific Research Institute, Beijing, China, August.

Appendix A: Cyclic Triaxial Tests - Test Parameters

Table A-1: Parameters for cyclic triaxial tests

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
M0E68C30.prn	Mont	N/A	0	0.667	81.1	100	0.270
M0E68C35.prn	Mont	N/A	0	0.667	81.1	100	0.442
M0E68C40.prn	Mont	N/A	0	0.667	81.1	100	0.510
M0E71C28.prn	Mont	Silt	0	0.688	70.0	100	0.259
M0E71C31.prn	Mont	Silt	0	0.683	72.6	100	0.275
M0E71C33.prn	Mont	Silt	0	0.689	69.5	100	0.308
M0E72C27.prn	Mont	N/A	0	0.697	65.3	100	0.363
M0E72C30.prn	Mont	N/A	0	0.699	64.2	100	0.387
M0E72C35.prn	Mont	N/A	0	0.699	64.2	100	0.439
M0E73C25.prn	Mont	N/A	0	0.712	57.4	100	0.218
M0E73C31.prn	Mont	N/A	0	0.710	58.4	100	0.374
M0E73C37.prn	Mont	N/A	0	0.710	58.4	100	0.442
M0E74C22.prn	Mont	N/A	0	0.733	46.3	100	0.355
M0E74C25.prn	Mont	N/A	0	0.732	46.8	100	0.380
M0E75C20.prn	Mont	N/A	0	0.731	47.4	100	0.169
M0E75C22.prn	Mont	N/A	0	0.733	46.2	100	0.402
M0E75C25.prn	Mont	N/A	0	0.731	47.4	100	0.294
M0E75C30.prn	Mont	N/A	0	0.730	47.9	100	0.351
M0E76C22.prn	Mont	N/A	0	0.740	42.6	100	0.265
M0E76C25.prn	Mont	N/A	0	0.740	42.6	100	0.300
M0E76C28.prn	Mont	N/A	0	0.739	42.9	100	0.351
M0E84C19.prn	Mont	Silt	0	0.823	-1.1	100	0.166
M0E84C20.prn	Mont	Silt	0	0.830	-4.7	100	0.169
M0E84C21.prn	Mont	Silt	0	0.828	-3.7	100	0.173
M5E61C32.prn	Mont	Silt	5	0.600	82.1	100	0.362
M5E61C35.prn	Mont	Silt	5	0.604	79.9	100	0.441
M5E61C40.prn	Mont	Silt	5	0.601	81.6	100	0.496
M5E65C27.prn	Mont	Silt	5	0.633	64.7	100	0.306
M5E65C30.prn	Mont	Silt	5	0.633	64.4	100	0.365
M5E65C33.prn	Mont	Silt	5	0.635	63.3	100	0.415

Table A-1: Parameters for cyclic triaxial tests (Continued)

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
M5E67C22.prn	Mont	Silt	5	0.654	53.4	100	0.198
M5E67C23.prn	Mont	Silt	5	0.653	54.0	100	0.288
M5E67C25.prn	Mont	Silt	5	0.652	54.5	100	0.307
M5E67C28.prn	Mont	Silt	5	0.650	55.6	100	0.348
M5E68C20.prn	Mont	Silt	5	0.661	49.7	100	0.171
M5E68C25.prn	Mont	Silt	5	0.661	49.7	100	0.289
M5E68C30.prn	Mont	Silt	5	0.654	53.4	100	0.379
M10E53C30.prn	Mont	Silt	10	0.514	81.0	100	0.351
M10E53C35.prn	Mont	Silt	10	0.517	79.7	100	0.441
M10E53C40.prn	Mont	Silt	10	0.515	80.6	100	0.512
M10E57C25.prn	Mont	Silt	10	0.556	62.9	100	0.295
M10E57C35.prn	Mont	Silt	10	0.556	62.9	100	0.403
M10E60C20.prn	Mont	Silt	10	0.581	52.2	100	0.252
M10E60C25.prn	Mont	Silt	10	0.582	51.7	100	0.289
M10E60C30.prn	Mont	Silt	10	0.582	51.7	100	0.361
M10E68C22.prn	Mont	Silt	10	0.673	12.5	100	0.276
M10E68C25.prn	Mont	Silt	10	0.670	13.8	100	0.298
M15E48C30.prn	Mont	Silt	15	0.465	81.7	100	0.339
M15E48C35.prn	Mont	Silt	15	0.464	82.1	100	0.426
M15E48C38.prn	Mont	Silt	15	0.465	81.7	100	0.472
M15E53C20.prn	Mont	Silt	15	0.515	61.8	100	0.250
M15E53C23.prn	Mont	Silt	15	0.513	62.5	100	0.269
M15E53C28.prn	Mont	Silt	15	0.512	62.9	100	0.330
M15E68C13.prn	Mont	Silt	15	0.643	10.8	100	0.184
M15E68C15.prn	Mont	Silt	15	0.642	11.2	100	0.163
M15E68C20.prn	Mont	Silt	15	0.643	10.8	100	0.217
M20E36C25.prn	Mont	Silt	20	0.353	96.8	100	0.424
M20E36C35.prn	Mont	Silt	20	0.351	97.5	100	0.491
M20E36C40.prn	Mont	Silt	20	0.353	96.8	100	0.220
M20E46C18.prn	Mont	Silt	20	0.455	60.8	100	0.233
M20E46C20.prn	Mont	Silt	20	0.454	61.1	100	0.295
M20E46C25.prn	Mont	Silt	20	0.455	60.8	100	0.323
M20E46C28.prn	Mont	Silt	20	0.456	60.4	100	0.146

Table A-1: Parameters for cyclic triaxial tests (Continued)

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
M20E68C10.prn	Mont	Silt	20	0.624	1.1	100	0.125
M20E68C13.prn	Mont	Silt	20	0.633	-2.1	100	0.173
M20E68C17.prn	Mont	Silt	20	0.630	-1.1	100	0.295
M25E35C30.prn	Mont	Silt	25	0.341	97.9	100	0.488
M25E35C40.prn	Mont	Silt	25	0.341	97.9	100	0.628
M25E35C50.prn	Mont	Silt	25	0.340	98.3	100	0.217
M25E42C20.prn	Mont	Silt	25	0.405	75.9	100	0.291
M25E42C25.prn	Mont	Silt	25	0.404	76.2	100	0.343
M25E42C28.prn	Mont	Silt	25	0.406	75.5	100	0.216
M25E47C17.prn	Mont	Silt	25	0.449	60.7	100	0.227
M25E47C20.prn	Mont	Silt	25	0.451	60.0	100	0.317
M25E47C30.prn	Mont	Silt	25	0.447	61.4	100	0.135
M25E68C09.prn	Mont	Silt	25	0.631	-2.1	100	0.114
M25E68C12.prn	Mont	Silt	25	0.626	-0.3	100	0.155
M25E68C15.prn	Mont	Silt	25	0.630	-1.7	100	0.113
M35E61C11.prn	Mont	Silt	35	0.516	55.8	100	0.075
M35E61C13.prn	Mont	Silt	35	0.474	69.2	100	0.104
M35E61C15.prn	Mont	Silt	35	0.495	62.5	100	0.112
M35E68C10.prn	Mont	Silt	35	0.489	64.4	100	0.076
M35E68C13.prn	Mont	Silt	35	0.479	67.6	100	0.103
M35E68C15.prn	Mont	Silt	35	0.463	72.8	100	0.111
M50E68C14.prn	Mont	Silt	50	0.563	73.9	100	0.106
M50E68C16.prn	Mont	Silt	50	0.546	76.9	100	0.118
M50E68C18.prn	Mont	Silt	50	0.544	77.3	100	0.148
M50E68C20.prn	Mont	Silt	50	0.531	79.6	100	0.140
M75E68C16.prn	Mont	Silt	75	0.624	92.0	100	0.126
M75E68C18.prn	Mont	Silt	75	0.630	90.9	100	0.134
M75E68C20.prn	Mont	Silt	75	0.632	90.5	100	0.145
Y0E76C30.prn	Yates	N/A	0	0.755	68.0	100	0.273
Y0E76C33.prn	Yates	N/A	0	0.749	70.0	100	0.312
Y0E76C35.prn	Yates	N/A	0	0.754	68.3	100	0.332

Table A-1: Parameters for cyclic triaxial tests (Continued)

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
Y0E90C19.prn	Yates	N/A	0	0.885	27.3	100	0.130
Y0E90C22.prn	Yates	N/A	0	0.893	24.8	100	0.155
Y0E90C25.prn	Yates	N/A	0	0.890	25.6	100	0.200
Y0E11C10.prn	Yates	N/A	0	1.006	-10.6	100	0.076
Y0E11C12.prn	Yates	N/A	0	1.005	-10.4	100	0.088
Y0E11C14.prn	Yates	N/A	0	1.007	-10.9	100	0.104
Y0E12C09.prn	Yates	N/A	0	1.076	-32.7	100	0.023
Y0E12C10.prn	Yates	N/A	0	1.114	-44.5	100	0.060
Y0E12C11.prn	Yates	N/A	0	1.091	-37.2	100	0.063
Y0E12C13.prn	Yates	N/A	0	1.075	-32.3	100	0.085
Y0E12C14.prn	Yates	N/A	0	1.074	-32.0	100	0.103
Y4E76C27.prn	Yates	Silt	4	0.754	52.1	100	0.239
Y4E76C30.prn	Yates	Silt	4	0.753	52.4	100	0.274
Y4E76C35.prn	Yates	Silt	4	0.754	52.1	100	0.323
Y4E83C22.prn	Yates	Silt	4	0.822	29.5	100	0.184
Y4E83C25.prn	Yates	Silt	4	0.818	30.8	100	0.223
Y4E83C28.prn	Yates	Silt	4	0.822	29.5	100	0.234
Y4E86C17.prn	Yates	Silt	4	0.828	27.5	100	0.136
Y4E86C18.prn	Yates	Silt	4	0.842	22.9	100	0.142
Y4E86C20.prn	Yates	Silt	4	0.826	28.2	100	0.156
Y4E86C23.prn	Yates	Silt	4	0.852	19.6	100	0.207
Y4E97C12.prn	Yates	Silt	4	0.944	-10.9	100	0.096
Y4E97C14.prn	Yates	Silt	4	0.938	-9.0	100	0.109
Y4E97C15.prn	Yates	Silt	4	0.940	-9.6	100	0.123
Y4E11C10.prn	Yates	Silt	4	1.021	-36.5	100	0.080
Y4E11C12.prn	Yates	Silt	4	1.018	-35.5	100	0.076
Y4E11C14.prn	Yates	Silt	4	1.002	-30.2	100	0.105
Y7E76C26.prn	Yates	Silt	7	0.755	34.0	100	0.228
Y7E76C27.prn	Yates	Silt	7	0.753	34.6	100	0.240
Y7E76C30.prn	Yates	Silt	7	0.756	33.7	100	0.273
Y7E76C32.prn	Yates	Silt	7	0.756	33.7	100	0.296

Table A-1: Parameters for cyclic triaxial tests (Continued)

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
Y7E78C25.prn	Yates	Silt	7	0.778	27.3	100	0.220
Y7E78C31.prn	Yates	Silt	7	0.771	29.4	100	0.287
Y7E78C34.prn	Yates	Silt	7	0.767	30.7	100	0.312
Y7E92C12.prn	Yates	Silt	7	0.903	-9.0	100	0.084
Y7E92C15.prn	Yates	Silt	7	0.902	-8.7	100	0.122
Y7E92C18.prn	Yates	Silt	7	0.891	-5.5	100	0.152
Y7E10C11.prn	Yates	Silt	7	0.942	-20.4	100	0.083
Y7E10C12.prn	Yates	Silt	7	0.981	-31.7	100	0.087
Y7E10C14.prn	Yates	Silt	7	0.959	-25.3	100	0.110
Y7E10C16.prn	Yates	Silt	7	0.959	-25.3	100	0.127
Y12E67C28.prn	Yates	Silt	12	0.664	54.4	100	0.231
Y12E67C31.prn	Yates	Silt	12	0.662	55.1	100	0.269
Y12E67C33.prn	Yates	Silt	12	0.656	56.9	100	0.298
Y12E70C35.prn	Yates	Silt	12	0.694	45.2	100	0.314
Y12E70C38.prn	Yates	Silt	12	0.703	42.4	100	0.350
Y12E70C41.prn	Yates	Silt	12	0.698	43.9	100	0.385
Y12E76C23.prn	Yates	Silt	12	0.757	25.7	100	0.194
Y12E76C26.prn	Yates	Silt	12	0.760	24.7	100	0.222
Y12E76C28.prn	Yates	Silt	12	0.751	27.5	100	0.255
Y12E84C18.prn	Yates	Silt	12	0.829	3.4	100	0.145
Y12E84C22.prn	Yates	Silt	12	0.831	2.8	100	0.179
Y12E84C25.prn	Yates	Silt	12	0.825	4.6	100	0.206
Y12E92C12.prn	Yates	Silt	12	0.876	-11.1	100	0.090
Y12E92C13.prn	Yates	Silt	12	0.878	-11.8	100	0.098
Y12E92C15.prn	Yates	Silt	12	0.871	-9.6	100	0.113
Y12E92C17.prn	Yates	Silt	12	0.872	-9.9	100	0.140
Y17E59C36.prn	Yates	Silt	17	0.573	76.4	100	0.315
Y17E59C39.prn	Yates	Silt	17	0.568	78.0	100	0.345
Y17E59C42.prn	Yates	Silt	17	0.572	76.7	100	0.401
Y17E70C25.prn	Yates	Silt	17	0.689	37.7	100	0.093
Y17E70C30.prn	Yates	Silt	17	0.679	41.0	100	0.149
Y17E70C32.prn	Yates	Silt	17	0.687	38.3	100	0.192

Table A-1: Parameters for cyclic triaxial tests (Continued)

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
Y17E73C22.prn	Yates	Silt	17	0.726	25.3	100	0.195
Y17E73C25.prn	Yates	Silt	17	0.724	26.0	100	0.201
Y17E73C27.prn	Yates	Silt	17	0.719	27.7	100	0.235
Y17E76C22.prn	Yates	Silt	17	0.748	18.0	100	0.195
Y17E76C25.prn	Yates	Silt	17	0.748	18.0	100	0.215
Y17E76C28.prn	Yates	Silt	17	0.751	17.0	100	0.249
Y17E84C12.prn	Yates	Silt	17	0.834	-10.7	100	0.089
Y17E84C15.prn	Yates	Silt	17	0.825	-7.7	100	0.121
Y17E84C18.prn	Yates	Silt	17	0.822	-6.7	100	0.150
Y26E63C30.prn	Yates	Silt	26	0.629	40.3	100	0.265
Y26E63C33.prn	Yates	Silt	26	0.623	42.8	100	0.277
Y26E63C36.prn	Yates	Silt	26	0.622	43.2	100	0.314
Y26E67C25.prn	Yates	Silt	26	0.662	26.9	100	0.180
Y26E67C27.prn	Yates	Silt	26	0.659	28.1	100	0.236
Y26E67C29.prn	Yates	Silt	26	0.658	28.5	100	0.259
Y26E71C19.prn	Yates	Silt	26	0.667	24.9	100	0.158
Y26E71C22.prn	Yates	Silt	26	0.652	31.0	100	0.186
Y26E71C24.prn	Yates	Silt	26	0.660	27.7	100	0.209
Y26E76C15.prn	Yates	Silt	26	0.753	-10.2	100	0.126
Y26E76C18.prn	Yates	Silt	26	0.750	-9.0	100	0.145
Y26E76C20.prn	Yates	Silt	26	0.750	-9.0	100	0.161
Y37E62C25.prn	Yates	Silt	37	0.612	25.1	100	0.212
Y37E62C28.prn	Yates	Silt	37	0.610	25.8	100	0.226
Y37E62C31.prn	Yates	Silt	37	0.614	24.4	100	0.266
Y37E67C14.prn	Yates	Silt	37	0.671	4.5	100	0.109
Y37E67C15.prn	Yates	Silt	37	0.660	8.4	100	0.115
Y37E67C17.prn	Yates	Silt	37	0.662	7.7	100	0.134
Y37E69C19.prn	Yates	Silt	37	0.681	1.0	100	0.114
Y37E69C20.prn	Yates	Silt	37	0.684	0.0	100	0.165
Y37E69C22.prn	Yates	Silt	37	0.673	3.8	100	0.180

Table A-1: Parameters for cyclic triaxial tests (Continued)

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
Y37E76C10.prn	Yates	Silt	37	0.755	-24.7	100	0.079
Y37E76C12.prn	Yates	Silt	37	0.716	-11.1	100	0.089
Y37E76C14.prn	Yates	Silt	37	0.731	-16.4	100	0.113
Y50E55C16.prn	Yates	Silt	50	0.539	80.9	100	0.115
Y50E55C18.prn	Yates	Silt	50	0.538	81.1	100	0.143
Y50E55C20.prn	Yates	Silt	50	0.545	80.0	100	0.155
Y50E55C23.prn	Yates	Silt	50	0.477	91.2	100	0.183
Y50E67C14.prn	Yates	Silt	50	0.592	72.2	100	0.106
Y50E67C16.prn	Yates	Silt	50	0.607	69.8	100	0.126
Y50E67C18.prn	Yates	Silt	50	0.596	71.6	100	0.135
Y50E76C09.prn	Yates	Silt	50	0.646	63.4	100	0.067
Y50E76C11.prn	Yates	Silt	50	0.636	65.0	100	0.082
Y50E76C13.prn	Yates	Silt	50	0.653	62.2	100	0.099
Y50E76C15.prn	Yates	Silt	50	0.675	58.6	100	0.114
Y50E78C10.prn	Yates	Silt	50	0.654	62.0	100	0.070
Y50E78C12.prn	Yates	Silt	50	0.700	54.4	100	0.088
Y50E78C14.prn	Yates	Silt	50	0.610	69.3	100	0.102
Y75E76C13.prn	Yates	Silt	75	0.696	85.5	100	0.102
Y75E76C15.prn	Yates	Silt	75	0.688	86.6	100	0.115
Y75E76C17.prn	Yates	Silt	75	0.694	85.8	100	0.115
Y75E85C14.prn	Yates	Silt	75	0.738	79.7	100	0.113
Y75E85C16.prn	Yates	Silt	75	0.750	78.0	100	0.125
Y75E85C18.prn	Yates	Silt	75	0.749	78.2	100	0.142
Y75E90C10.prn	Yates	Silt	75	0.771	75.1	100	0.073
Y75E90C13.prn	Yates	Silt	75	0.758	76.9	100	0.098
Y75E90C15.prn	Yates	Silt	75	0.778	74.2	100	0.111
SE68C13.prn	N/A	Silt	100	0.677	105.1	100	0.093
SE68C40.prn	N/A	Silt	100	0.685	104.3	100	0.347
SE68C43.prn	N/A	Silt	100	0.695	103.3	100	0.420
SE76C25.prn	N/A	Silt	100	0.752	97.5	100	0.226
SE76C35.prn	N/A	Silt	100	0.768	95.9	100	0.329
SE76C38.prn	N/A	Silt	100	0.742	98.5	100	0.355

Table A-1: Parameters for cyclic triaxial tests (Continued)

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
SE91C18.prn	N/A	Silt	100	0.871	85.6	100	0.150
SE91C20.prn	N/A	Silt	100	0.864	86.2	100	0.158
SE91C22.prn	N/A	Silt	100	0.855	87.2	100	0.177
SE95C10.prn	N/A	Silt	100	0.851	87.6	100	0.078
SE95C13.prn	N/A	Silt	100	0.843	88.4	100	0.092
SE95C15.prn	N/A	Silt	100	0.854	87.2	100	0.113
SE99C10.prn	N/A	Silt	100	0.884	84.3	100	0.073
SE99C13.prn	N/A	Silt	100	0.881	84.6	100	0.096
SE99C15.prn	N/A	Silt	100	0.880	84.7	100	0.105
YMK4C15.prn	Yates	S/K	4	0.804	35.5	100	0.121
YMK4C16.prn	Yates	S/K	4	0.784	42.0	100	0.120
YMK4C17.prn	Yates	S/K	4	0.812	32.9	100	0.129
YMK4C18.prn	Yates	S/K	4	0.781	43.2	100	0.148
YMK7C18.prn	Yates	S/K	7	0.748	36.0	100	0.147
YMK7C20.prn	Yates	S/K	7	0.749	35.8	100	0.159
YMK7C22.prn	Yates	S/K	7	0.751	35.3	100	0.178
YMK12C15.prn	Yates	S/K	12	0.718	37.9	100	0.114
YMK12C18.prn	Yates	S/K	12	0.716	38.5	100	0.142
YMK12C20.prn	Yates	S/K	12	0.719	37.5	100	0.158
YMK17C14.prn	Yates	S/K	17	0.652	50.1	100	0.105
YMK17C16.prn	Yates	S/K	17	0.661	47.1	100	0.125
YMK17C18.prn	Yates	S/K	17	0.676	42.1	100	0.139
YMK26C14.prn	Yates	S/K	26	0.581	60.1	100	0.110
YMK26C16.prn	Yates	S/K	26	0.578	61.0	100	0.122
YMK26C18.prn	Yates	S/K	26	0.587	57.6	100	0.144
YK4C10.prn	Yates	K	4	0.798	37.3	100	0.070
YK4C17.prn	Yates	K	4	0.810	33.3	100	0.128
YK4C20.prn	Yates	K	4	0.809	33.7	100	0.151
YK7C17.prn	Yates	K	7	0.760	32.6	100	0.133
YK7C20.prn	Yates	K	7	0.744	37.2	100	0.148
YK7C22.prn	Yates	K	7	0.743	37.4	100	0.173

Table A-1: Parameters for cyclic triaxial tests (Continued)

File Name	Sand Type	Fines Type	Fines Content (%)	Void Ratio	Relative Density (%)	Confining Stress (kPa)	Cyclic Stress Ratio
YK12C12.prn	Yates	K	12	0.713	39.3	100	0.080
YK12C15.prn	Yates	K	12	0.720	37.0	100	0.108
YK12C20.prn	Yates	K	12	0.719	37.3	100	0.139
YK17C12.prn	Yates	K	17	0.665	45.6	100	0.084
YK17C15.prn	Yates	K	17	0.646	52.0	100	0.111
YK17C17.prn	Yates	K	17	0.672	43.2	100	0.124
YK26C12.prn	Yates	K	26	0.567	65.8	100	0.061
YK26C16.prn	Yates	K	26	0.619	44.6	100	0.124
YK26C18.prn	Yates	K	26	0.648	32.4	100	0.137
YK37C14.prn	Yates	K	37	0.577	37.1	100	0.108
YK37C20.prn	Yates	K	37	0.605	27.6	100	0.143
YK37C23.prn	Yates	K	37	0.595	31.1	100	0.197
YKB17C19.prn	Yates	S/K/B	17	0.660	47.2	100	0.148
YKB17C21.prn	Yates	S/K/B	17	0.672	43.5	100	0.171
YKB17C23.prn	Yates	S/K/B	17	0.664	46.0	100	0.195
YMKB17C19.prn	Yates	S/K/B	17	0.665	45.8	100	0.150
YMKB17C21.prn	Yates	K/B	17	0.682	40.0	100	0.170
YMKB17C23.prn	Yates	K/B	17	0.662	46.7	100	0.183
YMKB17C25.prn	Yates	K/B	17	0.674	42.6	100	0.196
YB12C20.prn	Yates	B	12	0.745	29.2	100	0.166
YB12C22.prn	Yates	B	12	0.741	30.7	100	0.188
YB12C24.prn	Yates	B	12	0.741	30.8	100	0.201

Appendix B: Cyclic Triaxial Tests- Liquefaction Results

Table B-1: Liquefaction results from cyclic triaxial

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
M0E68C30.prn	0.270	167	173	N/A	167	N/A
M0E68C35.prn	0.442	25	31	33	31	31
M0E68C40.prn	0.510	8	13	16	13	13
M0E71C28.prn	0.259	37	39	40	39	39
M0E71C31.prn	0.275	10	12	13	12	12
M0E71C33.prn	0.308	10	12	13	12	12
M0E72C27.prn	0.363	72	75	77	74	74
M0E72C30.prn	0.387	56	59	61	58	58
M0E72C35.prn	0.439	10	13	14	13	13
M0E73C25.prn	0.218	351	353	N/A	352	N/A
M0E73C31.prn	0.374	14	16	17	16	16
M0E73C37.prn	0.442	8	12	13	12	12
M0E74C22.prn	0.355	109	110	112	109	109
M0E74C25.prn	0.380	51	52	N/A	51	51
M0E75C20.prn	0.169	457	458	N/A	456	N/A
M0E75C22.prn	0.402	241	243	244	241	N/A
M0E75C25.prn	0.294	34	35	36	36	36
M0E75C30.prn	0.351	7	8	8	8	8
M0E76C22.prn	0.265	87	88	N/A	87	N/A
M0E76C25.prn	0.300	36	37	37	36	37
M0E76C28.prn	0.351	12	13	13	13	13
M0E84C19.prn	0.166	31	31	N/A	30	30
M0E84C20.prn	0.169	5	5	5	4	4
M0E84C21.prn	0.173	6	6	N/A	6	6
M5E61C32.prn	0.362	39	49	53	47	47
M5E61C35.prn	0.441	22	31	35	29	29
M5E61C40.prn	0.496	9	17	21	16	16
M5E65C27.prn	0.306	80	82	87	83	83
M5E65C30.prn	0.365	19	24	26	24	24
M5E65C33.prn	0.415	6	11	13	11	11

Table B-1: Liquefaction results from cyclic triaxial tests (Continued)

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
M5E67C22.prn	0.198	445	447	448	444	N/A
M5E67C23.prn	0.288	164	166	168	164	N/A
M5E67C25.prn	0.307	23	26	27	26	26
M5E67C28.prn	0.348	7	10	10	10	10
M5E68C20.prn	0.171	418	421	N/A	416	N/A
M5E68C25.prn	0.289	36	40	41	38	39
M5E68C30.prn	0.379	8	10	12	11	11
M10E53C30.prn	0.351	106	116	121	108	N/A
M10E53C35.prn	0.441	34	48	53	45	46
M10E53C40.prn	0.512	10	21	24	19	19
M10E57C25.prn	0.295	114	116	118	113	N/A
M10E57C35.prn	0.403	5	9	11	9	10
M10E60C20.prn	0.252	174	177	180	174	N/A
M10E60C25.prn	0.289	33	37	38	36	36
M10E60C30.prn	0.361	6	9	10	9	9
M10E68C20.prn		42	43	44	42	N/A
M10E68C22.prn	0.276	21	22	22	22	22
M10E68C25.prn	0.298	6	6	7	7	7
M15E48C30.prn	0.339	42	56	60	50	51
M15E48C35.prn	0.426	17	27	30	23	23
M15E48C38.prn	0.472	10	22	26	21	23
M15E53C20.prn	0.250	203	206	207	202	N/A
M15E53C23.prn	0.269	44	49	51	47	47
M15E53C28.prn	0.330	10	12	13	10	23
M15E68C13.prn	0.184	98	99	N/A	99	N/A
M15E68C15.prn	0.163	38	39	39	38	39
M15E68C20.prn	0.217	8	9	9	9	9
M20E36C25.prn	0.424	346	356	374	337	N/A
M20E36C35.prn	0.491	39	61	67	59	67
M20E36C40.prn	0.220	24	40	44	45	45
M20E46C18.prn	0.233	315	318	321	310	N/A

Table B-1: Liquefaction results from cyclic triaxial tests (Continued)

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
M20E46C20.prn	0.295	52	58	60	61	61
M20E46C25.prn	0.323	12	16	17	15	15
M20E46C28.prn	0.146	10	14	15	13	13
M20E68C10.prn	0.125	150	153	N/A	147	N/A
M20E68C13.prn	0.173	56	57	57	57	N/A
M20E68C17.prn	0.295	8	9	9	9	N/A
M25E35C30.prn	0.488	40	52	56	54	N/A
M25E35C40.prn	0.628	13	26	26	26	N/A
M25E35C50.prn	0.217	7	11	13	15	N/A
M25E42C20.prn	0.291	126	129	N/A	124	N/A
M25E42C25.prn	0.343	19	22	23	22	22
M25E42C28.prn	0.216	11	15	16	15	15
M25E47C17.prn	0.227	59	62	63	61	61
M25E47C20.prn	0.317	27	31	32	30	30
M25E47C30.prn	0.135	5	7	7	7	7
M25E68C09.prn	0.114	156	158	N/A	156	N/A
M25E68C12.prn	0.155	55	56	56	56	56
M25E68C15.prn	0.113	8	8	9	9	N/A
M35E61C11.prn	0.075	43	43	43	45	45
M35E61C13.prn	0.104	9	9	N/A	12	N/A
M35E61C15.prn	0.112	6	6	6	6	N/A
M35E68C10.prn	0.076	38	38	38	39	N/A
M35E68C13.prn	0.103	11	11	N/A	12	N/A
M35E68C15.prn	0.111	12	12	12	12	N/A
M50E68C14.prn	0.106	8	8	9	9	N/A
M50E68C16.prn	0.118	9	9	N/A	10	N/A
M50E68C18.prn	0.148	3	3	3	3	N/A
M50E68C20.prn	0.140	5	5	5	5	N/A
M75E68C16.prn	0.126	12	14	14	15	N/A
M75E68C18.prn	0.134	17	18	18	18	18
M75E68C20.prn	0.145	6	7	7	8	8

Table B-1: Liquefaction results from cyclic triaxial tests (Continued)

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
Y0E76C30.prn	0.273	25	28	29	24	25
Y0E76C33.prn	0.312	11	13	14	12	12
Y0E76C35.prn	0.332	12	14	15	11	12
Y0E90C19.prn	0.130	38	38	38	37	37
Y0E90C22.prn	0.155	15	15	15	15	15
Y0E90C25.prn	0.200	6	6	6	6	6
Y0E11C10.prn	0.076	32	32	32	32	32
Y0E11C12.prn	0.088	15	15	N/A	15	15
Y0E11C14.prn	0.104	6	6	N/A	7	7
Y0E12C09.prn	0.023	44	44	44	44	44
Y0E12C10.prn	0.060	79	79	N/A	79	79
Y0E12C11.prn	0.063	61	61	61	61	61
Y0E12C13.prn	0.085	12	12	12	12	12
Y0E12C14.prn	0.103	3	3	3	3	3
Y4E76C27.prn	0.239	34	36	37	34	34
Y4E76C30.prn	0.274	12	14	15	12	13
Y4E76C35.prn	0.323	8	9	10	8	8
Y4E83C22.prn	0.184	56	57	57	55	55
Y4E83C25.prn	0.223	21	22	22	20	20
Y4E83C28.prn	0.234	12	13	13	12	12
Y4E86C17.prn	0.136	10	10	N/A	9	9
Y4E86C18.prn	0.142	7	7	7	7	7
Y4E86C20.prn	0.156	0	0	N/A	4	4
Y4E86C23.prn	0.207	17	17	17	16	16
Y4E97C12.prn	0.096	29	29	29	29	29
Y4E97C14.prn	0.109	12	13	N/A	13	13
Y4E97C15.prn	0.123	8	8	8	8	8
Y4E11C10.prn	0.080	31	31	31	31	31
Y4E11C12.prn	0.076	21	21	21	21	21
Y4E11C14.prn	0.105	7	7	7	7	7

Table B-1: Liquefaction results from cyclic triaxial tests (Continued)

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
Y7E76C26.prn	0.228	21	22	23	21	21
Y7E76C27.prn	0.240	26	27	28	26	26
Y7E76C30.prn	0.273	11	12	13	10	10
Y7E76C32.prn	0.296	7	8	8	7	7
Y7E78C25.prn	0.220	61	62	63	59	60
Y7E78C31.prn	0.287	17	18	19	16	16
Y7E78C34.prn	0.312	8	9	10	8	8
Y7E92C12.prn	0.084	66	66	66	66	66
Y7E92C15.prn	0.122	18	19	N/A	17	18
Y7E92C18.prn	0.152	5	5	5	5	5
Y7E10C11.prn	0.083	30	30	30	30	30
Y7E10C12.prn	0.087	31	31	N/A	30	30
Y7E10C14.prn	0.110	9	9	9	9	9
Y7E10C16.prn	0.127	5	5	5	5	5
Y12E67C28.prn	0.231	12	12	13	13	13
Y12E67C31.prn	0.269	13	13	14	12	12
Y12E67C33.prn	0.298	8	10	10	9	9
Y12E70C35.prn	0.314	14	16	17	14	14
Y12E70C38.prn	0.350	16	19	20	16	17
Y12E70C41.prn	0.385	13	15	16	13	13
Y12E76C23.prn	0.194	49	50	50	48	48
Y12E76C26.prn	0.222	14	14	15	14	14
Y12E76C28.prn	0.255	8	8	N/A	8	8
Y12E84C18.prn	0.145	31	31	N/A	30	30
Y12E84C22.prn	0.179	12	12	N/A	12	12
Y12E84C25.prn	0.206	5	5	5	5	N/A
Y12E92C12.prn	0.090	29	29	29	29	29
Y12E92C13.prn	0.098	32	32	N/A	32	32
Y12E92C15.prn	0.113	10	10	10	10	10
Y12E92C17.prn	0.140	4	4	4	4	4

Table B-1: Liquefaction results from cyclic triaxial tests (Continued)

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
Y17E59C36.prn	0.315	12	15	17	19	19
Y17E59C39.prn	0.345	14	16	18	19	19
Y17E59C42.prn	0.401	12	16	18	17	20
Y17E70C25.prn	0.093	120	123	N/A	120	120
Y17E70C30.prn	0.149	17	18	18	17	17
Y17E70C32.prn	0.192	3	3	3	3	3
Y17E73C22.prn	0.195	48	49	N/A	48	48
Y17E73C25.prn	0.201	27	27	28	26	26
Y17E73C27.prn	0.235	8	8	8	8	8
Y17E76C22.prn	0.195	40	40	41	39	39
Y17E76C25.prn	0.215	15	16	16	15	15
Y17E76C28.prn	0.249	7	7	7	7	7
Y17E84C12.prn	0.089	63	63	N/A	63	63
Y17E84C15.prn	0.121	22	22	N/A	21	21
Y17E84C18.prn	0.150	6	6	6	6	6
Y26E63C30.prn	0.265	14	16	16	13	13
Y26E63C33.prn	0.277	16	17	18	16	17
Y26E63C36.prn	0.314	15	16	17	15	16
Y26E67C25.prn	0.180	73	74	74	71	71
Y26E67C27.prn	0.236	15	16	16	14	14
Y26E67C29.prn	0.259	6	7	N/A	6	6
Y26E71C19.prn	0.158	22	23	N/A	22	22
Y26E71C22.prn	0.186	15	16	N/A	15	15
Y26E71C24.prn	0.209	10	10	10	10	10
Y26E76C15.prn	0.126	31	31	31	31	31
Y26E76C18.prn	0.145	10	10	10	10	10
Y26E76C20.prn	0.161	8	8	N/A	8	8
Y37E62C25.prn	0.212	14	15	N/A	13	13
Y37E62C28.prn	0.226	11	12	12	11	11
Y37E62C31.prn	0.266	5	5	5	5	N/A

Table B-1: Liquefaction results from cyclic triaxial tests (Continued)

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
Y37E67C14.prn	0.109	37	37	37	36	36
Y37E67C15.prn	0.115	19	19	19	18	18
Y37E67C17.prn	0.134	10	10	10	10	10
Y37E69C19.prn	0.114	30	30	30	30	30
Y37E69C20.prn	0.165	8	8	8	8	8
Y37E69C22.prn	0.180	4	4	4	4	4
Y37E76C10.prn	0.079	47	47	47	47	47
Y37E76C12.prn	0.089	20	20	20	20	N/A
Y37E76C14.prn	0.113	8	8	8	8	N/A
Y50E55C16.prn	0.115	16	17	18	20	N/A
Y50E55C18.prn	0.143	18	19	19	19	20
Y50E55C20.prn	0.155	9	10	10	12	12
Y50E55C23.prn	0.183	4	4	5	6	N/A
Y50E67C14.prn	0.106	13	13	13	13	N/A
Y50E67C16.prn	0.126	8	8	8	8	N/A
Y50E67C18.prn	0.135	5	5	N/A	5	N/A
Y50E76C09.prn	0.067	48	49	49	50	50
Y50E76C11.prn	0.082	28	28	28	28	N/A
Y50E76C13.prn	0.099	11	12	12	13	13
Y50E76C15.prn	0.114	6	6	N/A	7	N/A
Y50E78C10.prn	0.070	45	46	47	47	47
Y50E78C12.prn	0.088	15	16	16	17	17
Y50E78C14.prn	0.102	7	7	7	8	8
Y75E76C13.prn	0.102	27	28	28	27	28
Y75E76C15.prn	0.115	11	12	12	12	N/A
Y75E76C17.prn	0.115	14	14	14	15	15
Y75E85C14.prn	0.113	15	15	15	16	16
Y75E85C16.prn	0.125	9	9	9	10	N/A
Y75E85C18.prn	0.142	5	5	5	6	N/A
Y75E90C10.prn	0.073	35	36	37	38	38
Y75E90C13.prn	0.098	9	10	10	11	12
Y75E90C15.prn	0.111	5	6	6	7	8

Table B-1: Liquefaction results from cyclic triaxial tests (Continued)

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
SE68C13.prn	0.093	617	620	N/A	605	N/A
SE68C40.prn	0.347	26	31	35	16	N/A
SE68C43.prn	0.420	33	39	45	22	N/A
SE76C25.prn	0.226	197	N/A	N/A	182	N/A
SE76C35.prn	0.329	23	26	28	18	26
SE76C38.prn	0.355	4	5	6	4	N/A
SE91C18.prn	0.150	20	20	21	19	22
SE91C20.prn	0.158	10	11	11	12	N/A
SE91C22.prn	0.177	4	5	N/A	5	N/A
SE95C10.prn	0.078	51	52	53	54	55
SE95C13.prn	0.092	21	21	22	23	24
SE95C15.prn	0.113	9	9	10	10	11
SE99C10.prn	0.073	44	46	46	47	47
SE99C13.prn	0.096	11	12	12	13	14
SE99C15.prn	0.105	7	8	8	9	9
YMK4C15.prn	0.121	28	28	28	28	28
YMK4C16.prn	0.120	22	22	22	22	22
YMK4C17.prn	0.129	28	28	29	29	29
YMK4C18.prn	0.148	6	6	6	6	N/A
YMK7C18.prn	0.147	20	20	20	20	20
YMK7C20.prn	0.159	10	10	N/A	10	10
YMK7C22.prn	0.178	7	7	N/A	7	7
YMK12C15.prn	0.114	24	24	24	24	24
YMK12C18.prn	0.142	8	8	8	8	N/A
YMK12C20.prn	0.158	5	5	N/A	5	N/A
YMK17C14.prn	0.105	19	20	20	20	20
YMK17C16.prn	0.125	10	10	10	10	10
YMK17C18.prn	0.139	5	6	6	6	6
YMK26C14.prn	0.110	23	24	24	25	25
YMK26C16.prn	0.122	12	13	13	14	14
YMK26C18.prn	0.144	6	6	7	7	7

Table B-1: Liquefaction results from cyclic triaxial tests (Continued)

File Name	Cyclic Stress Ratio	Cycles to 1% DA Strain	Cycles to 2.5% DA Strain	Cycles to 5% DA Strain	Cycles to Initial Liq'n	Cycles to Ave Ru of 1.00
YK4C10.prn	0.070	30	30	30	30	30
YK4C17.prn	0.128	16	16	16	16	N/A
YK4C20.prn	0.151	8	8	8	8	N/A
YK7C17.prn	0.133	13	13	13	13	13
YK7C20.prn	0.148	12	12	12	12	12
YK7C22.prn	0.173	4	4	4	4	5
YK12C12.prn	0.080	39	40	40	40	40
YK12C15.prn	0.108	12	13	13	13	14
YK12C20.prn	0.139	3	3	3	3	4
YK17C12.prn	0.084	44	46	46	46	46
YK17C15.prn	0.111	8	9	9	10	10
YK17C17.prn	0.124	7	7	7	7	7
YK26C12.prn	0.061	45	48	49	48	N/A
YK26C16.prn	0.124	6	7	7	7	7
YK26C18.prn	0.137	3	4	4	5	5
YK37C14.prn	0.108	165	173	N/A	167	N/A
YK37C20.prn	0.143	11	12	13	22	22
YK37C23.prn	0.197	3	4	4	5	N/A
YKB17C19.prn	0.148	36	44	48	49	N/A
YKB17C21.prn	0.171	9	11	13	13	N/A
YKB17C23.prn	0.195	5	7	8	8	N/A
YMKB17C19.prn	0.150	14	16	17	19	N/A
YMKB17C21.prn	0.170	8	10	11	10	N/A
YMKB17C23.prn	0.183	4	5	5	6	N/A
YMKB17C25.prn	0.196	3	4	5	5	N/A
YB12C20.prn	0.166	23	26	27	23	25
YB12C22.prn	0.188	14	17	18	14	17
YB12C24.prn	0.201	14	17	19	14	N/A
Y12E67C28.prn	0.224	12	12	13	13	N/A

N/A = Not Achieved

Appendix C: Index Density Testing

Table C-1: Minimum index density test data for Yatesville sand with silt

Silt Content (%)	Specific Gravity	Method A		Method B		Method C	
		Minimum Density	Maximum e	Minimum Density	Maximum e	Minimum Density	Maximum e
0	2.72	88.5	0.918	86.1	0.972	86.2	0.968
4	2.72	91.0	0.864	88.8	0.911	85.7	0.983
7	2.72	91.7	0.851	90.7	0.872	87.1	0.951
11.8	2.73	93.3	0.819	92.2	0.840	88.9	0.913
16.9	2.73	95.3	0.782	94.1	0.802	90.5	0.882
25.9	2.73	97.8	0.741	98.6	0.728	92.9	0.835
37.2	2.74	99.1	0.719	101.1	0.684	94.9	0.801
50	2.75	N. P.	N. P.	84.3	1.021	83.6	1.050
75	2.76	N. P.	N. P.	74.3	1.312	77.0	1.234
100	2.77	N. P.	N. P.	63.5	1.733	65.3	1.647

N. P. = Not Performed

Table C-2: Maximum index density test data for Yatesville sand with silt

Silt Content (%)	Vibrating Table			Standard Proctor			Modified Proctor		
	Opt WC	Maximum GammaD	Minimum e	Opt WC	Maximum GammaD	Minimum e	Opt WC	Maximum GammaD	Minimum e
0	19.7	102.5	0.656	6.0	100.5	0.689	13.0	103.8	0.635
4	19.0	105.4	0.612	5.8	101.8	0.668	8.0	103.5	0.641
7	17.8	111.0	0.531	13.7	101.6	0.673	8.8	105.0	0.618
11.8	16.1	111.9	0.519	11.5	102.0	0.668	10.9	107.4	0.584
16.9	15.9	113.2	0.505	6.6	107.2	0.588	6.5	109.6	0.554
25.9	13.7	116.2	0.468	5.9	109.6	0.556	6.8	111.8	0.525
37.2	12.8	122.1	0.400	7.2	114.2	0.496	7.6	116.9	0.462
50	12.6	120.6	0.423	9.4	117.6	0.456	9.4	123.4	0.388
75	16.9	108.2	0.591	14.5	109.5	0.571	13.7	114.9	0.497
100	20.3	100.4	0.727	16.0	108.4	0.595	14.3	111.8	0.546

Appendix D: Limiting Silt Content

The limiting silt content of a sand may be calculated using the following equation:

Limiting Silt Content $= G_{sm}e_s/G_{ss}(1+e_m)$

Where:

G_{sm} = Specific gravity of the silt
G_{ss} = Specific gravity of the sand
e_m = void ratio of the silt
e_s = Maximum index void ratio of the silt

Table D-1: Limiting silt content based on Yatesville silt

Soil	D60 mm	D10 mm	Cu	Gs	emax	Gamma Dmin (pcf)	Limiting Silt Content (%)	Reference
Yatesville Silt	0.039	0.009	4.3	2.77	1.72	63.5	N.A.	Polito and Martin (1999)
Medium Gravel	13.00	9.50	1.37	2.65	0.90	87.0	34.6	Lee and Singh (1971)
Pea Gravel	12.00	8.00	1.50	2.65	0.98	83.5	37.6	Lee and Singh (1971)
Coarse Sand	4.50	3.00	1.50	2.65	0.99	83.1	38.0	Lee and Singh (1971)
Medium Sand	0.84	0.70	1.20	2.65	1.05	80.7	40.3	Lee and Singh (1971)
Fine Sand	0.28	0.20	1.40	2.65	1.08	79.5	41.5	Lee and Singh (1971)
Silty Sand	0.12	0.08	1.50	2.65	1.17	76.2	44.9	Lee and Singh (1971)
Ottawa Std Sand	0.72	0.60	1.20	2.65	0.71	96.7	27.3	Lee and Singh (1971)
Loch Aline Sand	0.21	0.16	1.31	2.65	0.80	91.9	30.7	Lee and Singh (1971)
Folkstone Bed Sand	0.29	0.14	2.07	2.65	0.85	89.4	32.6	Lee and Singh (1971)
Fine Sand	0.16	0.06	2.67	2.74	0.85	92.4	31.6	Lee and Singh (1971)
Fine Sand	0.85	0.65	1.31	2.65	0.79	92.4	30.3	Lee and Singh (1971)
Fine Sand	0.85	0.75	1.13	2.65	0.86	88.9	33.0	Lee and Singh (1971)
River Sand	0.18	0.12	1.50	2.65	0.90	87.0	34.6	Lee and Singh (1971)
Fine Sand	0.12	0.08	1.50	2.65	0.92	86.1	35.3	Lee and Singh (1971)
Medium Sand	0.45	0.15	3.00	2.65	1.05	80.7	40.3	Lee and Singh (1971)
Fine Gravel	4.50	1.20	3.75	2.65	1.10	78.7	42.2	Lee and Singh (1971)
Coarse Sand	0.90	0.10	9.00	2.65	0.85	89.4	32.6	Lee and Singh (1971)
Chattahooche River Sand	0.48	0.20	2.40	2.65	1.10	78.7	42.2	Lee and Singh (1971)
Crushed Granite Gneiss	125.00	50.00	2.50	2.62	0.77	92.4	29.9	Lee and Singh (1971)
Leighton Buzzard Sand	0.20	0.15	1.33	2.60	0.72	94.3	28.2	Lee and Singh (1971)
Leighton Buzzard Sand	0.90	0.15	6.00	2.70	0.85	91.1	32.0	Lee and Singh (1971)
Ham River Sand	0.45	0.35	1.29	2.70	0.83	92.1	31.3	Lee and Singh (1971)
Brasted Sand	0.30	0.14	2.14	2.70	0.79	94.1	29.8	Lee and Singh (1971)
Wing Beach Sand	0.20	0.07	2.86	2.65	0.93	85.7	35.7	Lee and Singh (1971)
Bolsa Island Sand	0.40	0.20	2.00	2.70	0.85	91.1	32.0	Lee and Singh (1971)
Silica Sand	45.00	30.00	1.50	2.65	1.08	79.5	41.5	Lee and Singh (1971)
Sacramento River Sand	0.24	0.15	1.60	2.68	1.03	82.4	39.1	Lee and Singh (1971)
Tarbella Dam Med Sand	0.40	0.10	4.00	2.86	0.83	97.5	29.5	Lee and Singh (1971)
Antioch Sand	0.24	0.15	1.60	2.74	1.14	79.9	42.3	Lee and Singh (1971)
Monterey Sand	0.66	0.54	1.22	2.64	0.83	90.0	32.0	Lee and Singh (1971)
Thumb Island Sand	0.55	0.22	2.50	2.70	0.91	88.2	34.3	Lee and Singh (1971)
Lake Michigan Sand	0.18	0.06	3.00	2.65	0.87	88.4	33.4	Lee and Singh (1971)
Marsa El Brega Sand	0.27	0.10	2.70	2.65	0.72	96.1	27.6	Lee and Singh (1971)
Marsa El Brega Sand	0.16	0.08	1.90	2.65	0.88	88.0	33.8	Lee and Singh (1971)
Uniform Spheres	-	-	1.00	2.65	0.91	86.6	34.9	Lee and Singh (1971)
Yatesville Sand	0.22	0.09	2.45	2.72	0.97	86.1	36.4	Polito and Martin (1999)
Montery #0/30 Sand	0.46	0.31	1.48	2.65	0.82	90.8	31.5	Polito and Martin (1999)

N.R. = Not Reported
N.A. = Not Applicable

Table D-2: Limiting silt content based on uniform inorganic silt

Soil	D60 mm	D10 mm	Cu	Gs	emax	Gamma Dmin (pcf)	Limiting Silt Content (%)	Reference
Uniform Inorganic Silt	N.R.	N.R.	N.R.	2.67	1.10	79.3	N.A.	Lade and Lee (1976)
Medium Gravel	13.00	9.50	1.37	2.65	0.90	87.0	43.2	Lee and Singh (1971)
Pea Gravel	12.00	8.00	1.50	2.65	0.98	83.5	47.0	Lee and Singh (1971)
Coarse Sand	4.50	3.00	1.50	2.65	0.99	83.1	47.5	Lee and Singh (1971)
Medium Sand	0.84	0.70	1.20	2.65	1.05	80.7	50.4	Lee and Singh (1971)
Fine Sand	0.28	0.20	1.40	2.65	1.08	79.5	51.8	Lee and Singh (1971)
Silty Sand	0.12	0.08	1.50	2.65	1.17	76.2	56.1	Lee and Singh (1971)
Ottawa Std Sand	0.72	0.60	1.20	2.65	0.71	96.7	34.1	Lee and Singh (1971)
Loch Aline Sand	0.21	0.16	1.31	2.65	0.80	91.9	38.4	Lee and Singh (1971)
Folkstone Bed Sand	0.29	0.14	2.07	2.65	0.85	89.4	40.8	Lee and Singh (1971)
Fine Sand	0.16	0.06	2.67	2.74	0.85	92.4	39.4	Lee and Singh (1971)
Fine Sand	0.85	0.65	1.31	2.65	0.79	92.4	37.9	Lee and Singh (1971)
Fine Sand	0.85	0.75	1.13	2.65	0.86	88.9	41.3	Lee and Singh (1971)
River Sand	0.18	0.12	1.50	2.65	0.90	87.0	43.2	Lee and Singh (1971)
Fine Sand	0.12	0.08	1.50	2.65	0.92	86.1	44.1	Lee and Singh (1971)
Medium Sand	0.45	0.15	3.00	2.65	1.05	80.7	50.4	Lee and Singh (1971)
Fine Gravel	4.50	1.20	3.75	2.65	1.10	78.7	52.8	Lee and Singh (1971)
Coarse Sand	0.90	0.10	9.00	2.65	0.85	89.4	40.8	Lee and Singh (1971)
Chattahooche River Sand	0.48	0.20	2.40	2.65	1.10	78.7	52.8	Lee and Singh (1971)
Crushed Granite Gneiss	125.00	50.00	2.50	2.62	0.77	92.4	37.4	Lee and Singh (1971)
Leighton Buzzard Sand	0.20	0.15	1.33	2.60	0.72	94.3	35.2	Lee and Singh (1971)
Leighton Buzzard Sand	0.90	0.15	6.00	2.70	0.85	91.1	40.0	Lee and Singh (1971)
Ham River Sand	0.45	0.35	1.29	2.70	0.83	92.1	39.1	Lee and Singh (1971)
Brasted Sand	0.30	0.14	2.14	2.70	0.79	94.1	37.2	Lee and Singh (1971)
Wing Beach Sand	0.20	0.07	2.86	2.65	0.93	85.7	44.6	Lee and Singh (1971)
Bolsa Island Sand	0.40	0.20	2.00	2.70	0.85	91.1	40.0	Lee and Singh (1971)
Silica Sand	45.00	30.00	1.50	2.65	1.08	79.5	51.8	Lee and Singh (1971)
Sacramento River Sand	0.24	0.15	1.60	2.68	1.03	82.4	48.9	Lee and Singh (1971)
Tarbella Dam Med Sand	0.40	0.10	4.00	2.86	0.83	97.5	36.9	Lee and Singh (1971)
Antioch Sand	0.24	0.15	1.60	2.74	1.14	79.9	52.9	Lee and Singh (1971)
Monterey Sand	0.66	0.54	1.22	2.64	0.83	90.0	40.0	Lee and Singh (1971)
Thumb Island Sand	0.55	0.22	2.50	2.70	0.91	88.2	42.9	Lee and Singh (1971)
Lake Michigan Sand	0.18	0.06	3.00	2.65	0.87	88.4	41.7	Lee and Singh (1971)
Marsa El Brega Sand	0.27	0.10	2.70	2.65	0.72	96.1	34.5	Lee and Singh (1971)
Marsa El Brega Sand	0.16	0.08	1.90	2.65	0.88	88.0	42.2	Lee and Singh (1971)
Uniform Spheres	-	-	1.00	2.65	0.91	86.6	43.7	Lee and Singh (1971)
Yatesville Sand	0.22	0.09	2.45	2.72	0.97	86.1	45.4	Polito and Martin (1999)
Montery #0/30 Sand	0.46	0.31	1.48	2.65	0.82	90.8	39.4	Polito and Martin (1999)

N.R. = Not Reported
N.A. = Not Applicable

Table D-3: Limiting silt content based on Iowa silt

Soil	D60 mm	D10 mm	Cu	Gs	emax	Gamma Dmin (pcf)	Limiting Silt Content (%)	Reference
Iowa Silt	0.004	0.003	1.37	2.67	1.12	78.6	N.A.	Arman and Thorton (1973)
Medium Gravel	13.00	9.50	1.37	2.65	0.90	87.0	42.8	Lee and Singh (1971)
Pea Gravel	12.00	8.00	1.50	2.65	0.98	83.5	46.6	Lee and Singh (1971)
Coarse Sand	4.50	3.00	1.50	2.65	0.99	83.1	47.1	Lee and Singh (1971)
Medium Sand	0.84	0.70	1.20	2.65	1.05	80.7	49.9	Lee and Singh (1971)
Fine Sand	0.28	0.20	1.40	2.65	1.08	79.5	51.3	Lee and Singh (1971)
Silty Sand	0.12	0.08	1.50	2.65	1.17	76.2	55.6	Lee and Singh (1971)
Ottawa Std Sand	0.72	0.60	1.20	2.65	0.71	96.7	33.7	Lee and Singh (1971)
Loch Aline Sand	0.21	0.16	1.31	2.65	0.80	91.9	38.0	Lee and Singh (1971)
Folkstone Bed Sand	0.29	0.14	2.07	2.65	0.85	89.4	40.4	Lee and Singh (1971)
Fine Sand	0.16	0.06	2.67	2.74	0.85	92.4	39.1	Lee and Singh (1971)
Fine Sand	0.85	0.65	1.31	2.65	0.79	92.4	37.5	Lee and Singh (1971)
Fine Sand	0.85	0.75	1.13	2.65	0.86	88.9	40.9	Lee and Singh (1971)
River Sand	0.18	0.12	1.50	2.65	0.90	87.0	42.8	Lee and Singh (1971)
Fine Sand	0.12	0.08	1.50	2.65	0.92	86.1	43.7	Lee and Singh (1971)
Medium Sand	0.45	0.15	3.00	2.65	1.05	80.7	49.9	Lee and Singh (1971)
Fine Gravel	4.50	1.20	3.75	2.65	1.10	78.7	52.3	Lee and Singh (1971)
Coarse Sand	0.90	0.10	9.00	2.65	0.85	89.4	40.4	Lee and Singh (1971)
Chattahooche River Sand	0.48	0.20	2.40	2.65	1.10	78.7	52.3	Lee and Singh (1971)
Crushed Granite Gneiss	125.00	50.00	2.50	2.62	0.77	92.4	37.0	Lee and Singh (1971)
Leighton Buzzard Sand	0.20	0.15	1.33	2.60	0.72	94.3	34.9	Lee and Singh (1971)
Leighton Buzzard Sand	0.90	0.15	6.00	2.70	0.85	91.1	39.6	Lee and Singh (1971)
Ham River Sand	0.45	0.35	1.29	2.70	0.83	92.1	38.7	Lee and Singh (1971)
Brasted Sand	0.30	0.14	2.14	2.70	0.79	94.1	36.9	Lee and Singh (1971)
Wing Beach Sand	0.20	0.07	2.86	2.65	0.93	85.7	44.2	Lee and Singh (1971)
Bolsa Island Sand	0.40	0.20	2.00	2.70	0.85	91.1	39.6	Lee and Singh (1971)
Silica Sand	45.00	30.00	1.50	2.65	1.08	79.5	51.3	Lee and Singh (1971)
Sacramento River Sand	0.24	0.15	1.60	2.68	1.03	82.4	48.4	Lee and Singh (1971)
Tarbella Dam Med Sand	0.40	0.10	4.00	2.86	0.83	97.5	36.6	Lee and Singh (1971)
Antioch Sand	0.24	0.15	1.60	2.74	1.14	79.9	52.4	Lee and Singh (1971)
Monterey Sand	0.66	0.54	1.22	2.64	0.83	90.0	39.6	Lee and Singh (1971)
Thumb Island Sand	0.55	0.22	2.50	2.70	0.91	88.2	42.4	Lee and Singh (1971)
Lake Michigan Sand	0.18	0.06	3.00	2.65	0.87	88.4	41.3	Lee and Singh (1971)
Marsa El Brega Sand	0.27	0.10	2.70	2.65	0.72	96.1	34.2	Lee and Singh (1971)
Marsa El Brega Sand	0.16	0.08	1.90	2.65	0.88	88.0	41.8	Lee and Singh (1971)
Uniform Spheres	-	-	1.00	2.65	0.91	86.6	43.2	Lee and Singh (1971)
Yatesville Sand	0.22	0.09	2.45	2.72	0.97	86.1	45.0	Polito and Martin (1999)
Montery #0/30 Sand	0.46	0.31	1.48	2.65	0.82	90.8	39.0	Polito and Martin (1999)

N.R. = Not Reported
N.A. = Not Applicable

Table D-4: Limiting silt content based on Danby silt

Soil	D60 mm	D10 mm	Cu	Gs	emax	Gamma Dmin (pcf)	Limiting Silt Content (%)	Reference
Danby Silt	N.R.	N.R.	N.R.	2.67	0.86	89.6	N.A.	Clukey et al. (1983)
Medium Gravel	13.00	9.50	1.37	2.65	0.90	87.0	48.8	Lee and Singh (1971)
Pea Gravel	12.00	8.00	1.50	2.65	0.98	83.5	53.1	Lee and Singh (1971)
Coarse Sand	4.50	3.00	1.50	2.65	0.99	83.1	53.6	Lee and Singh (1971)
Medium Sand	0.84	0.70	1.20	2.65	1.05	80.7	56.9	Lee and Singh (1971)
Fine Sand	0.28	0.20	1.40	2.65	1.08	79.5	58.5	Lee and Singh (1971)
Silty Sand	0.12	0.08	1.50	2.65	1.17	76.2	63.4	Lee and Singh (1971)
Ottawa Std Sand	0.72	0.60	1.20	2.65	0.71	96.7	38.5	Lee and Singh (1971)
Loch Aline Sand	0.21	0.16	1.31	2.65	0.80	91.9	43.3	Lee and Singh (1971)
Folkstone Bed Sand	0.29	0.14	2.07	2.65	0.85	89.4	46.0	Lee and Singh (1971)
Fine Sand	0.16	0.06	2.67	2.74	0.85	92.4	44.5	Lee and Singh (1971)
Fine Sand	0.85	0.65	1.31	2.65	0.79	92.4	42.8	Lee and Singh (1971)
Fine Sand	0.85	0.75	1.13	2.65	0.86	88.9	46.6	Lee and Singh (1971)
River Sand	0.18	0.12	1.50	2.65	0.90	87.0	48.8	Lee and Singh (1971)
Fine Sand	0.12	0.08	1.50	2.65	0.92	86.1	49.8	Lee and Singh (1971)
Medium Sand	0.45	0.15	3.00	2.65	1.05	80.7	56.9	Lee and Singh (1971)
Fine Gravel	4.50	1.20	3.75	2.65	1.10	78.7	59.6	Lee and Singh (1971)
Coarse Sand	0.90	0.10	9.00	2.65	0.85	89.4	46.0	Lee and Singh (1971)
Chattahooche River Sand	0.48	0.20	2.40	2.65	1.10	78.7	59.6	Lee and Singh (1971)
Crushed Granite Gneiss	125.00	50.00	2.50	2.62	0.77	92.4	42.2	Lee and Singh (1971)
Leighton Buzzard Sand	0.20	0.15	1.33	2.60	0.72	94.3	39.8	Lee and Singh (1971)
Leighton Buzzard Sand	0.90	0.15	6.00	2.70	0.85	91.1	45.2	Lee and Singh (1971)
Ham River Sand	0.45	0.35	1.29	2.70	0.83	92.1	44.1	Lee and Singh (1971)
Brasted Sand	0.30	0.14	2.14	2.70	0.79	94.1	42.0	Lee and Singh (1971)
Wing Beach Sand	0.20	0.07	2.86	2.65	0.93	85.7	50.4	Lee and Singh (1971)
Bolsa Island Sand	0.40	0.20	2.00	2.70	0.85	91.1	45.2	Lee and Singh (1971)
Silica Sand	45.00	30.00	1.50	2.65	1.08	79.5	58.5	Lee and Singh (1971)
Sacramento River Sand	0.24	0.15	1.60	2.68	1.03	82.4	55.2	Lee and Singh (1971)
Tarbella Dam Med Sand	0.40	0.10	4.00	2.86	0.83	97.5	41.7	Lee and Singh (1971)
Antioch Sand	0.24	0.15	1.60	2.74	1.14	79.9	59.7	Lee and Singh (1971)
Monterey Sand	0.66	0.54	1.22	2.64	0.83	90.0	45.1	Lee and Singh (1971)
Thumb Island Sand	0.55	0.22	2.50	2.70	0.91	88.2	48.4	Lee and Singh (1971)
Lake Michigan Sand	0.18	0.06	3.00	2.65	0.87	88.4	47.1	Lee and Singh (1971)
Marsa El Brega Sand	0.27	0.10	2.70	2.65	0.72	96.1	39.0	Lee and Singh (1971)
Marsa El Brega Sand	0.16	0.08	1.90	2.65	0.88	88.0	47.7	Lee and Singh (1971)
Uniform Spheres	-	-	1.00	2.65	0.91	86.6	49.3	Lee and Singh (1971)
Yatesville Sand	0.22	0.09	2.45	2.72	0.97	86.1	51.3	Polito and Martin (1999)
Montery #0/30 Sand	0.46	0.31	1.48	2.65	0.82	90.8	44.5	Polito and Martin (1999)

N.R. = Not Reported
N.A. = Not Applicable

Table D-5: Limiting silt content based on Prices Fork silt

Soil	D60 mm	D10 mm	Cu	Gs	emax	Gamma Dmin (pcf)	Limiting Silt Content (%)	Reference
Prices Fork Silt	N.R.	N.R.	N.R.	2.68	1.01	83.2	N.A.	Mullen (1991)
Medium Gravel	13.00	9.50	1.37	2.65	0.90	87.0	45.3	Lee and Singh (1971)
Pea Gravel	12.00	8.00	1.50	2.65	0.98	83.5	49.3	Lee and Singh (1971)
Coarse Sand	4.50	3.00	1.50	2.65	0.99	83.1	49.8	Lee and Singh (1971)
Medium Sand	0.84	0.70	1.20	2.65	1.05	80.7	52.9	Lee and Singh (1971)
Fine Sand	0.28	0.20	1.40	2.65	1.08	79.5	54.4	Lee and Singh (1971)
Silty Sand	0.12	0.08	1.50	2.65	1.17	76.2	58.9	Lee and Singh (1971)
Ottawa Std Sand	0.72	0.60	1.20	2.65	0.71	96.7	35.7	Lee and Singh (1971)
Loch Aline Sand	0.21	0.16	1.31	2.65	0.80	91.9	40.3	Lee and Singh (1971)
Folkstone Bed Sand	0.29	0.14	2.07	2.65	0.85	89.4	42.8	Lee and Singh (1971)
Fine Sand	0.16	0.06	2.67	2.74	0.85	92.4	41.4	Lee and Singh (1971)
Fine Sand	0.85	0.65	1.31	2.65	0.79	92.4	39.8	Lee and Singh (1971)
Fine Sand	0.85	0.75	1.13	2.65	0.86	88.9	43.3	Lee and Singh (1971)
River Sand	0.18	0.12	1.50	2.65	0.90	87.0	45.3	Lee and Singh (1971)
Fine Sand	0.12	0.08	1.50	2.65	0.92	86.1	46.3	Lee and Singh (1971)
Medium Sand	0.45	0.15	3.00	2.65	1.05	80.7	52.9	Lee and Singh (1971)
Fine Gravel	4.50	1.20	3.75	2.65	1.10	78.7	55.4	Lee and Singh (1971)
Coarse Sand	0.90	0.10	9.00	2.65	0.85	89.4	42.8	Lee and Singh (1971)
Chattahooche River Sand	0.48	0.20	2.40	2.65	1.10	78.7	55.4	Lee and Singh (1971)
Crushed Granite Gneiss	125.00	50.00	2.50	2.62	0.77	92.4	39.2	Lee and Singh (1971)
Leighton Buzzard Sand	0.20	0.15	1.33	2.60	0.72	94.3	36.9	Lee and Singh (1971)
Leighton Buzzard Sand	0.90	0.15	6.00	2.70	0.85	91.1	42.0	Lee and Singh (1971)
Ham River Sand	0.45	0.35	1.29	2.70	0.83	92.1	41.0	Lee and Singh (1971)
Brasted Sand	0.30	0.14	2.14	2.70	0.79	94.1	39.0	Lee and Singh (1971)
Wing Beach Sand	0.20	0.07	2.86	2.65	0.93	85.7	46.8	Lee and Singh (1971)
Bolsa Island Sand	0.40	0.20	2.00	2.70	0.85	91.1	42.0	Lee and Singh (1971)
Silica Sand	45.00	30.00	1.50	2.65	1.08	79.5	54.4	Lee and Singh (1971)
Sacramento River Sand	0.24	0.15	1.60	2.68	1.03	82.4	51.3	Lee and Singh (1971)
Tarbella Dam Med Sand	0.40	0.10	4.00	2.86	0.83	97.5	38.7	Lee and Singh (1971)
Antioch Sand	0.24	0.15	1.60	2.74	1.14	79.9	55.5	Lee and Singh (1971)
Monterey Sand	0.66	0.54	1.22	2.64	0.83	90.0	41.9	Lee and Singh (1971)
Thumb Island Sand	0.55	0.22	2.50	2.70	0.91	88.2	45.0	Lee and Singh (1971)
Lake Michigan Sand	0.18	0.06	3.00	2.65	0.87	88.4	43.8	Lee and Singh (1971)
Marsa El Brega Sand	0.27	0.10	2.70	2.65	0.72	96.1	36.2	Lee and Singh (1971)
Marsa El Brega Sand	0.16	0.08	1.90	2.65	0.88	88.0	44.3	Lee and Singh (1971)
Uniform Spheres	-	-	1.00	2.65	0.91	86.6	45.8	Lee and Singh (1971)
Yatesville Sand	0.22	0.09	2.45	2.72	0.97	86.1	47.7	Polito and Martin (1999)
Montery #0/30 Sand	0.46	0.31	1.48	2.65	0.82	90.8	41.3	Polito and Martin (1999)

N.R. = Not Reported
N.A. = Not Applicable

Vita For Carmine Polito

Carmine Paul Polito was born in Hudson, New York, on July 19th, 1963, the second child of Carmine and Wynette Polito, and the fourth in a relatively long line of Carmine Polito's. After living in Claverack, Malone, and Pomona, New York, he moved to Westminster, California in 1973.

He graduated from Westminster High School before attending California Polytechnical State University at San Luis Obispo. He earned his Bachelors of Science in Civil Engineering from Cal Poly in 1986. He received his Masters of Science in Civil Engineering with a concentration in Geotechnical Engineering from Virginia Polytechnic Institute and State University in 1989.

After his Bachelors degree, Carmine spent 2 years working for the US Army Corps of Engineers in Los Angeles, Ca. He has also worked for CH2M Hill in Santa Ana, California for 3 years as a geotechnical engineer. He is registered as a Professional Engineer in the Sate of California.

In the little spare time allotted to a graduate student, Carmine enjoys golf, fishing, sports, literature, epic poetry, music, Latin, good friends and good beer.