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LABORATORY STUDY OF LIQUEFACTION TRIGGERING CRITERIA

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SUMMARY

The term "liquefaction" has been used to describe a wide range of phenomena associated with loss of strength of saturated soil due to dynamic loading. Historically, liquefaction initiation criteria are commonly based on either pore-pressure or strain amplitude within a sample. This has led to numerous definitions of "soil liquefaction" and inevitable inconsistencies within the geotechnical community over the years. In a recent laboratory study conducted at the University of California at Berkeley, the liquefaction "triggering" threshold criteria is selected as the first occurrence of either 6% double amplitude (DA) or 6% single amplitude (SA) shear strain, whichever occurs first. Of particular interest is the relationship between the threshold shear strain and the corresponding peak excess pore pressure, $r_{u,max}$. In addition, it appears that the occurrence of 6% DA (or 6% SA) threshold shear strain is well correlated the onset of flow-type deformation behavior in liquefiable soils.

INTRODUCTION

Since the inception of modern studies of soil liquefaction, numerous definitions of "soil liquefaction" have been proposed. While it is clearly understood that the phenomenon of soil liquefaction depends on three principal factors: excess pore pressure, shear strength, and shear strain/deformation of the soil, many of these definitions are based solely on one factor instead of all three. Unfortunately, these various definitions have led to inconsistent (sometime even confusing) situations within the geotechnical community, and despite considerable efforts that have been devoted to rectifying this problem of inconsistent definitions and semantics, the situation has not improved much and the phenomenon of soil liquefaction remains variably defined.

In 1978, the Committee on Soil Dynamics of the Geotechnical Engineering Division of ASCE recommended the definition of liquefaction as: "The act or process of transforming any substance into a liquid" [1]. It was hoped that this definition was general enough to be universally applicable and error free. Unfortunately, it avoided any quantitative criteria, and thus was too loose to be useful for either

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researchers or practitioners. In the meantime, the U.C. Berkeley definition of soil liquefaction continues to be "significant reduction of strength and stiffness of a soil, principally as a result of pore pressure increase and corresponding reduction in effective stress." This definition is deliberately broad and general, and avoids taking a stand with regard to the specific question regarding how much strength and stiffness reduction is required to satisfy the "significant" threshold.

Most other definitions that have been used in academic research and engineering practice are based on either pore pressure criteria, shear strength, or shear strain/deformation criteria. Recently, an innovative energy-based liquefaction criterion was proposed by Obermeier et al. [2] in which normalized energy capacities are computed as the area bounded by the stress-strain hysteresis loops during cyclic loading and used as the criterion for liquefaction. This new approach appears promising in evaluation of the liquefaction potential of reconstituted soil samples; however, its practical application has yet to be developed.

Recently, a series of high quality, uni-directional, simple shear tests were performed on fully saturated samples of Monterey 0/30 sand. These tests incorporate a variety of relative densities and confining pressures, and are part of a comprehensive cyclic simple shear testing database developed at UC Berkeley [3, 4]. In these tests, the liquefaction "triggering" threshold criteria is selected as the first occurrence of either 6% double amplitude (DA) or 6% single amplitude (SA) shear strain, depending on the static driving shear stress conditions. This definition is schematically illustrated in Figure 1. The results of this testing program provided insight into some fundamental aspects of the behavior of liquefiable soils. It appears that this threshold shear strain to be a reliable indicator of the onset of flow-type deformation behavior in liquefiable soils. Also of particular interest is the relationship between the threshold shear strain and the corresponding peak excess pore pressure, $r_{u,max}$.

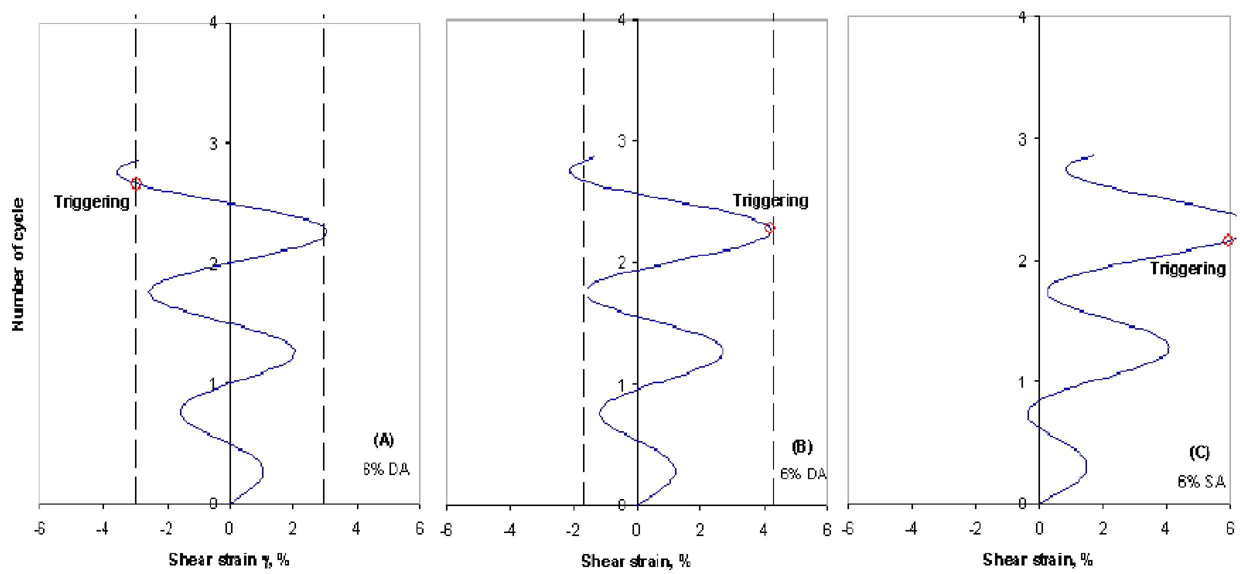

Figure 1: Strain-based liquefaction triggering criteria in this study for (A) symmetrical cyclic loading and, (B) and (C) asymmetrical cyclic loading

BACKGROUND

Pore Water Pressure-Based Criteria

When based on pore pressure-related criteria, soil liquefaction has often been defined as the state at which the excess pore water pressure ratio (r_u) equals 1.0. This occurs when the pore water pressure increase (Δu) becomes equal to the initial vertical effective overburden stress ($r_u = \Delta u / \sigma'_{v,o} = 1.0$) in simple shear tests and in field studies, or when Δu equals the initial effective minor principal stress ($r_u = \Delta u/\sigma'_{3c} = 1.0$) in triaxial compression tests. Sometimes, the terms "initial liquefaction" and "partial liquefaction" are used to describe the occurrence of r_u =1.0.

Pore water pressure criteria capture some vital elements of the mechanism of liquefaction; and as a result, it is appealing to scholars and researchers who actually measure the change of pore pressure in the laboratory. Unfortunately, pore pressure based liquefaction definitions suffer from some major drawbacks. Perhaps the most important of these is the ambiguity regarding whether full pore pressure ratio (r_u =1.0) is truly achievable in many situations. While liquefaction researchers routinely report unity pore pressure ratios in laboratory tests, those ratios may not necessarily be exactly 1.0; instead, they can be values approximately equal to 1.0. For instance, Ishihara [5] suggested that the pore water pressure ratio in silty sands or sandy silts containing some amount of fines is observed not to develop fully, but instead may stop building up when it has reached a value equal to about 90 to 95 percent of the initial confining stress. If liquefaction was strictly defined as the occurrence of the full pore pressure ratio of 1.0, then these soils would never "liquefy" despite of the fact that they may behave as liquefiable materials for all practical engineering purposes.

In light of recent laboratory studies, it seems doubtable whether a full pore pressure ratio is achievable in dense, clean sands. Similarly, soils with initial static "driving" shear stresses cannot approach r_u of 1.0. Instead, they "fail" as they approach a lesser pore pressure ratio of $r_{u,lim}$. $r_{u,lim}$ is the "limiting" pore pressure ratio at which the effective normal stress on the most critical plane has been reduced enough that the frictional shear strength that can be mobilized is met or exceeded by the initial static "driving" shear stress. However, determination of $r_{u,lim}$, rather than r_u, is not yet widely practiced and is considerably more difficult in many engineering applications. Lastly, recent multi-directional simple shear testing has shown that in tests where stresses are not completely removed during each cycle of loading (e.g. circular loading), the r_u values can remain well below 1.0 while still producing very large strains [3].

Another major drawback of this type of definition is that it does not reflect nor assess the seismic performance of liquefiable soils, particularly denser soils. It has long been understood that recognizing "liquefaction" in loose sands is relatively straightforward; the development of state of initial liquefaction (r_u~1) is accompanied immediately by accumulation of large shear strains. Conversely, denser soils may generate high pore pressures under severe cyclic loading, but they have limited shear strain potential due to their strong dilative behavior upon continuous shear deformation. Therefore, pore pressure parameter alone provides little (i.e.: incomplete) information about the soils' performance unless the deformations developed are also discussed.

Pore water pressure based criteria also suffer from the fact that seismically-induced excess pore pressures are almost never recorded in-situ. The few exceptions include the Wildlife Site, California and the Lotung Site, Taiwan. Consequently, in most field liquefaction cases, investigators instead rely on manifestations of liquefaction observable at the ground surface (e.g.: ground deformations, sand boil ejecta, etc.) to determine the occurrences of soil liquefaction.

Finally, as a practical consideration in laboratory experimentation, it can be difficult to measure Δu near the center of soil specimens when the rate of loading is high relative to the rate at which pore pressures equalize across the specimen. This can be problematic in samples with relatively low permeability, and was a reason for adoption of some deformation threshold as defining occurrence of "liquefaction" in many early laboratory studies. In samples with significant fines content, pore pressure lag may require very slow rates of cyclic loading if pore pressure based criteria are to be used to define liquefaction. Unfortunately, these same soil types are vulnerable to rate effects, so that testing at loading rates much slower than those of real earthquakes produces biased results.

Strength-Based Criteria

Seismically-induced soil strength loss is often of great concern for the overall stability of structures and earth slopes or embankments. Therefore, strength-based criteria and liquefaction definitions also exist. This type of definition commonly requires that liquefied soils behave like a viscous fluid, which implies that they must nearly completely lose their shear strength and cannot regain strength. Because soils gain strength as a result of an effective confining stress, they will lose strength completely if and only if the effective confining stress becomes zero. It is therefore obvious that this type of liquefaction definition is essentially equivalent to pore water pressure criteria based definitions. Consequently, it inherits many same shortcomings discussed in the previous section. For instance, all but the loosest soils tend to dilate as they begin to develop significant shear strains (or deformations), and thus do not long sustain a condition of zero, or even negligible, strength upon continued shearing.

Strain/Deformation-Based Criteria

The other criteria available for defining soil liquefaction are those associated with shear strain or shear deformation of soils. These criteria were not initially as popular as pore pressure or shear strength criteria, in part because many tended to regard shear strain/deformation in soils as a consequence of liquefaction. This misconception quickly became obsolete as it is now accepted that shear deformations and pore pressure (and thus effective stress and strength) are all closely interrelated.

Dobry et al. [6] suggested that a threshold of shear strain exists below which no permanent excess pore pressure generation will occur in soils. Their study showed that this "threshold strain", which is insensitive to grain size of soils, sample preparation technique and effective confining stress but varies with soil type and plasticity, is between 0.01% and 0.04% for confining stresses of between 24 and 192kPa. They concluded that the attainment of threshold strain may be used to assess the potential for triggering of liquefaction. This is, however, an overly conservative threshold for most engineering applications.

The National Research Council (1985) recommended using the term "liquefaction" to encompass all phenomena involving excessive deformation of saturated cohesionless soils; while Seed and Lee [7] separated those phenomena into "liquefaction" and "failure". They suggested that a soil may be considered to have liquefied when its resistance to deformation is zero over a wide range of strain amplitude, and a soil may be considered to have failed when excessive deformations are induced by applied stresses. Ishihara [5] examined many laboratory cyclic triaxial tests and proposed the use of 5% double amplitude (DA) axial strain in cyclic triaxial testing as a criterion to define the occurrence of liquefaction and cyclic softening for both clean sands and sands with fines. For field studies and simple shear tests, Ishihara also suggested that in the tests modeling level ground, the condition of failure in sands with any density should be taken as 3% single amplitude (SA) shear strain.

Shear strain and deformation serve as good criteria for seismic performance assessment purposes. The seismic performances of most earth structures, slopes, underground structures and pipelines, and conventional buildings are strongly correlated with the performance of soils of which they were built, built in or built upon. Therefore, shear strain/deformation based liquefaction definitions implicitly provide vital information about the performance levels of soils if liquefaction occurs.

While the shear strain/deformation criterion seems to be the ideal basis for performance based engineering, it unfortunately has some potentially serious limitations. One limitation is that the measurement of shear strain or deformation depends on the deformation mode that the soil is subjected to. As a result, different strain criteria must be selected for different deformation modes (e.g.: triaxial or simple shear). For example, in undrained conditions the axial strain (ε_a) in the triaxial test (TX) is related to the shear strain γ as $\gamma = 1.5\ \varepsilon_a$. Another limitation of this type of definition lies in the non-uniqueness of choosing a certain strain level as the liquefaction criterion. Ishihara recommended 3% to 3.5% SA shear strain and 5% DA axial strain based on approximate correlations between strains and pore pressure ratios. Others have used different strain levels in the range of 2 to 10 percent. In practice, strain-based liquefaction criteria are sometimes used interchangeably with strain-based failure criterion, such as the 20% DA suggested by Seed and Lee. The influence of selection of different strain levels on the measured liquefaction resistance or cyclic strength varies from relatively insignificant in loose soils to highly important in dense soils. Consequently, dense soils' liquefaction resistances are very sensitive to the level of strain selected as the criteria for "triggering" or initialization of liquefaction. Variations of strain levels used in liquefaction definitions raise problems of compatibility and lead to confusion in the application of these definitions. The laboratory study presented herein partly serves the purpose of identifying the proper shear strain level that best delineate the initialization of liquefaction in a clean, uniform sand.

LABORATORY TESTING PROGRAM

The laboratory testing program undertaken consisted primarily of a series of stress-controlled, uni-directional, undrained, cyclic simple shear tests performed on saturated Monterey No. 0/30 sand samples formed by pluviation through water.

The U.C. Berkeley bi-directional simple shear apparatus is the primary testing device used in this study. This device was initially designed in the late 1980's, based on three objectives: (1) the capability of uni- and bi-directional monotonic and cyclic loading, (2) low mechanical compliance and (3) a sealable chamber for lateral support and saturation purposes. Several upgrades of this apparatus were later performed in 1998. In this study, the system was reconditioned and the device frame was further air-tightened.

Monterey No. 0/30 sand is a tan, sub-angular to sub-rounded, medium sized, uniform and clean, quartz beach sand with small amount of feldspar and mica minerals. It is similar to the Monterey No. 0 sand. It has a minimum void ratio of 0.541 determined by the modified Japanese method and a maximum void ratio of 0.885 determined by dry tipping method.

All samples in this study were prepared by wet pluviation. This method was selected because it produces a fabric that is similar to the fabric which occurs from natural sedimentation. Dry sand was first placed with water in a flask and the materials were de-aired under full vacuum. A wire-reinforced membrane was secured to the bottom cap by O-rings and a custom mold was applied. The assembly was then filled with de-aired water and a screen was placed in the mold. The saturated sand is then pluviated into the mold through the water column. Gentle vibration to the mold was applied to prepare denser samples. Then the surface of the sample was carefully leveled by means of a small vacuum suction. The top cap was carefully placed on the sand and the membrane was secured to the top cap through two O-rings. A small vacuum was then applied to the sample before the mold was removed. At this stage, the dimensions of the samples were measured and the density of sample was calculated. Finally the assembled sample was mounted on the top table of the simple shear apparatus and carefully flushed with de-aired water to yield a higher initial degree of saturation. The sample then underwent backpressure saturation until a B-value of 0.94 or greater was achieved.

The sample was then subjected to K_o-consolidation until the target vertical effective (i.e.: overburden) stress was reached. In the consolidation process the K_o boundary condition was maintained by the means of a wire-reinforced membrane. In a test that had 80 or 180 kPa overburden stress, the effective confining stress was raised to 30 or 100 kPa (respectively) at this stage to avoid the risk of leakage caused by elevated pore water pressure during the liquefaction process. While this increment of effective confining stress slightly violated the K_o-consolidation conditions, it was necessary because the risk of leakage was found unacceptably high when the outward differential pressure on the membrane exceeded approximately 80 kPa. It also helped offset the tensional stress in the circumferential wires. When a sample was consolidated to the target overburden pressure, the density of the sample was again calculated using the height of consolidated sample. This density was typically within ±3% of the target value. At this stage, the drainage valves were closed and a cyclic shear test was started. During a test, a cyclic load with 0.1 Hertz frequency and constant amplitude was applied superimposed over the static shear force. No initial shear was imposed in level ground tests. The constant vertical load is approximately maintained by a pressure regulator. Additional information on the boundary condition is provided by Kammerer [4]. The cyclic loading stopped when the sample reached a certain threshold of shear deformation, or when a maximum number of loading cycles had been completed.

A total of 81 uni-directional cyclic simple tests with level ground conditions were included in the final data set. These tests were performed with three levels of effective overburden pressure and four levels of relative density, as summarized in the following table. In this study, no test with 35% relative density was performed under a 180 kPa effective overburden pressure because even the loosest sample (which was prepared by wet pluviation method and had a typical initial relative density of 30% or more) was consolidated to at least 40% relative density under this stress level. This observation suggests that the minimum relative density achievable by wet pluviation method for Monterey No. 0/30 sand under 180 kPa effective overburden pressure is approximately 40 percent. This is in good agreement with the observation of a minimum relative density over 33% existing in wet pluviated Toyoura sand sample under negligible confining stress, as reported by Ishihara [5]. Detailed information on the testing program is provided in a report [2].

Table 1: Uni-directional cyclic simple shear testing program (level ground conditions)

Number of tests		**Initial vertical effective stress σ'_{vi}, kPa**		
		40	80	180
Relative density, D_r (%)	35	8	5	
	45	7	7	6
	60	7	8	6
	80	8	11	8

FINDINGS OF THIS STUDY

Shear Strain Amplitude at Liquefaction Triggering

Shear strain or deformation based liquefaction triggering criteria are becoming increasingly popular despite the difficulties of choosing a unique strain level as the liquefaction criterion and of selecting different strain criteria for different deformation modes (e.g. triaxial and simple shear). While triaxial testing is the most widely performed dynamic soil testing method, its applicability in fundamental liquefaction studies is severely hampered by an inherent stress path problem that is widely discussed in the literature [3, 11]. As a result, sophisticated simple shear type testing methods, including torsional shear and direct simple shear, have become the standard tool in more recent fundamental liquefaction studies. This effectively resolves the need to select different strain criteria for different deformation modes. In other word, only simple-shear-strain-based liquefaction triggering criteria inherent to simple shear should be used. The corresponding strain amplitudes in other deformation modes may be deduced when needed.

In recent years, a number of laboratory investigations of high technical caliber have been conducted by various researchers to improve our understanding of the stress-strain behaviors and the post-liquefaction deformation mechanism of liquefied soils. With no exceptions, these studies demonstrated that the post-liquefaction stress-strain behavior of cohesionless soils is characterized by a prominent shear strain range exhibiting extremely low shear stiffness, which is absent in the pre-liquefaction stress-strain behaviors. The strain associated with this low shear stiffness is accompanied by nearly full pore pressure ratio (r_u) and nearly zero effective vertical confining stress in uni-directional simple-shear type tests. The range of this low shear stiffness strain increases steadily with additional cycles of shear loading and can reach tens of percent in laboratory testing. An example of this type of stress-strain behavior is shown in Figure 2.

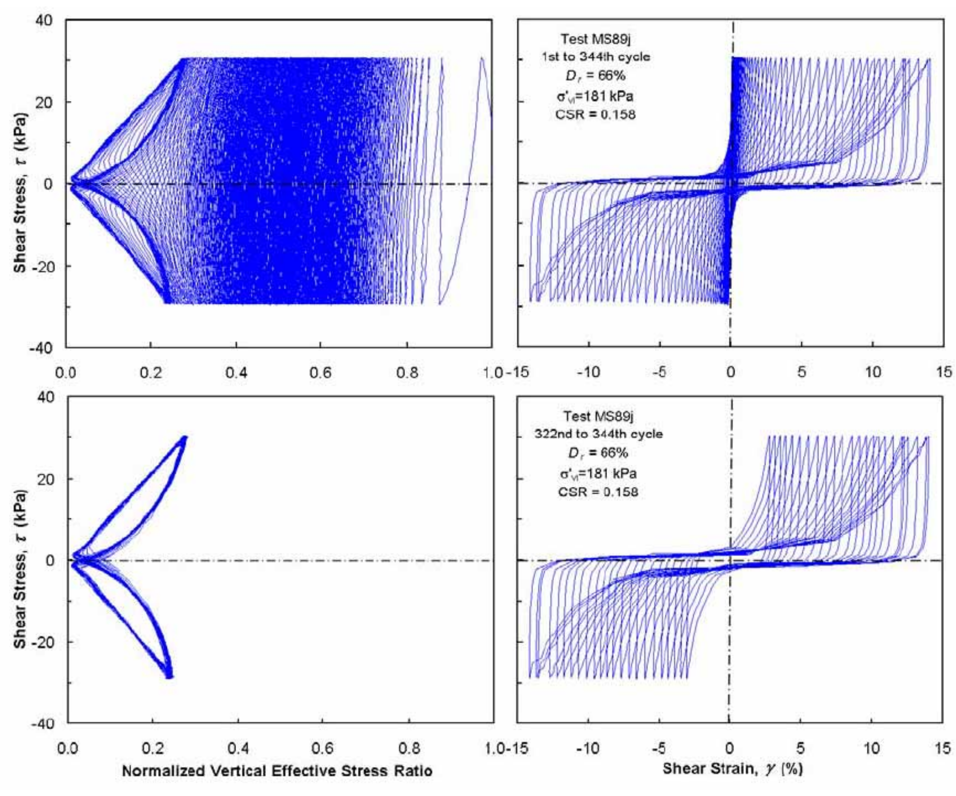

Figure 2: An example of post-liquefaction behavior of Monterey No. 0/30 sand

The strain component associated with very low shear stiffness is usually termed flow strain; however, it should to be noted that flow strain not only exists in "flow" type liquefaction phenomena, but also in "cyclic mobility" type liquefaction. The other post-liquefaction strain component is the strain-hardening component that is characterized by a sharp drop in pore pressure due to dilation upon shearing and a corresponding sharp increase in shear stiffness. The strain-hardening component evolves gradually in pre-liquefaction stage, and essentially stops evolving in post-liquefaction process. It is clear that the presence of the flow strain component distinguishes the pre- and post-liquefaction behavior of cohesionless soils, and thus may serve as an indicator of liquefaction initiation.

Flow strain, however, is not directly measured in laboratories. Instead, the typically measured strain is the maximum shear strain amplitude (γ_{max}) which is often of the primary concern in engineering practice. Figure 3 shows the relationship between the maximum shear strain amplitude, the flow strain component, and the strain-hardening component.

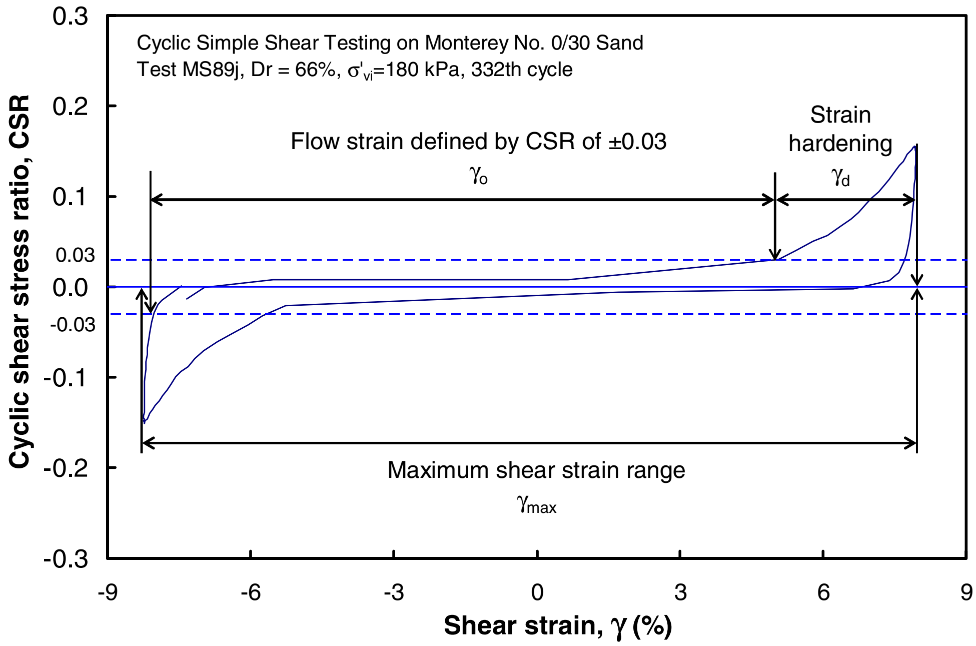

Figure 3: Relationship between flow strain amplitude and maximum shear strain in this study

Vaid and Thomas [8] were among the first to quantitatively study the relationship between the flow strain component and the maximum shear strain amplitude by conducting monotonic triaxial compression tests on Fraser River sand that had been liquefied during initial cyclic loading phases. Figure 4 shows the data from Vaid and Thomas, in which the flow strain component ($\Delta\varepsilon$) was defined as the axial strain (ε_d) required to mobilize 5 kPa deviatoric stress. Shamoto et al. [9] conducted a similar study of the flow strain in cyclic torsional shear tests on Toyoura sand with various densities, and their data is also shown in Figure 4.

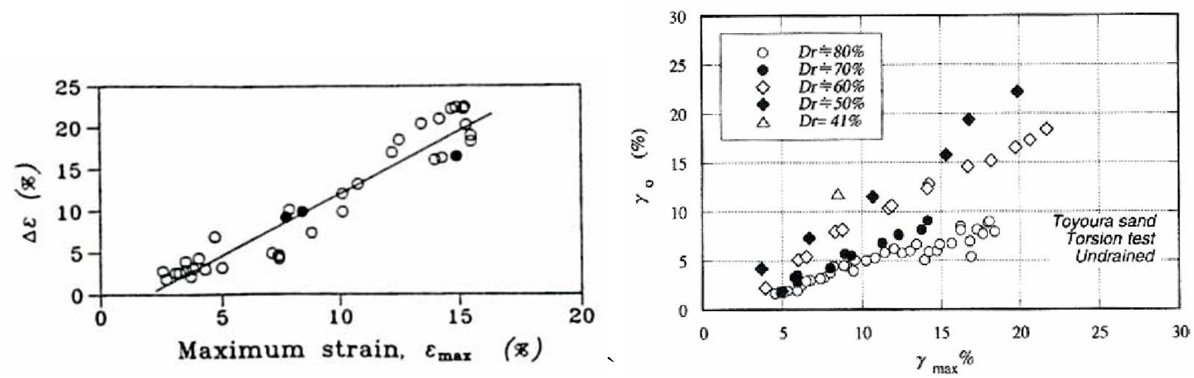

Figure 4: Relationship between flow strain component and the preceding maximum cyclic strain in triaxial tests (left: Vaid and Tomas [8]), and in torsional tests (right: Shamoto et al. [9])

In the study presented herein, the relationship between flow shear strain and the maximum shear strain was investigated. Figure 5 presents the data of flow strain and limiting shear strain in the "level ground" series of tests. In this study, the limiting shear strains were defined as the maximum shear strain amplitude in either the 15th cycle or the last stress cycle if the total number of stress cycles were fewer than 15. The flow strain (γ_0) is defined as the shear strain required to mobilize a shear stress equal to 3% of the initial effective vertical stress (equivalent to a *CSR* of 0.03). As shown in Figure 5, the flow strain component was insignificant when the single amplitude limiting shear strain was less than 3%. For larger limiting shear strain amplitude, the flow strain (γ_0) was well correlated with the limiting shear strain in a linear function. These results show that the 3% SA (equivalent of 6% DA in symmetric loading) shear strain marks the threshold at which sands starts to exhibit the flow strain type of behavior. This is comparable with the "entrance shear strain" (γ_{entry}) of 2.65% in Toyoura sand. The similarity suggests that the 3% SA shear strain threshold may be relatively independent of the sand tested.

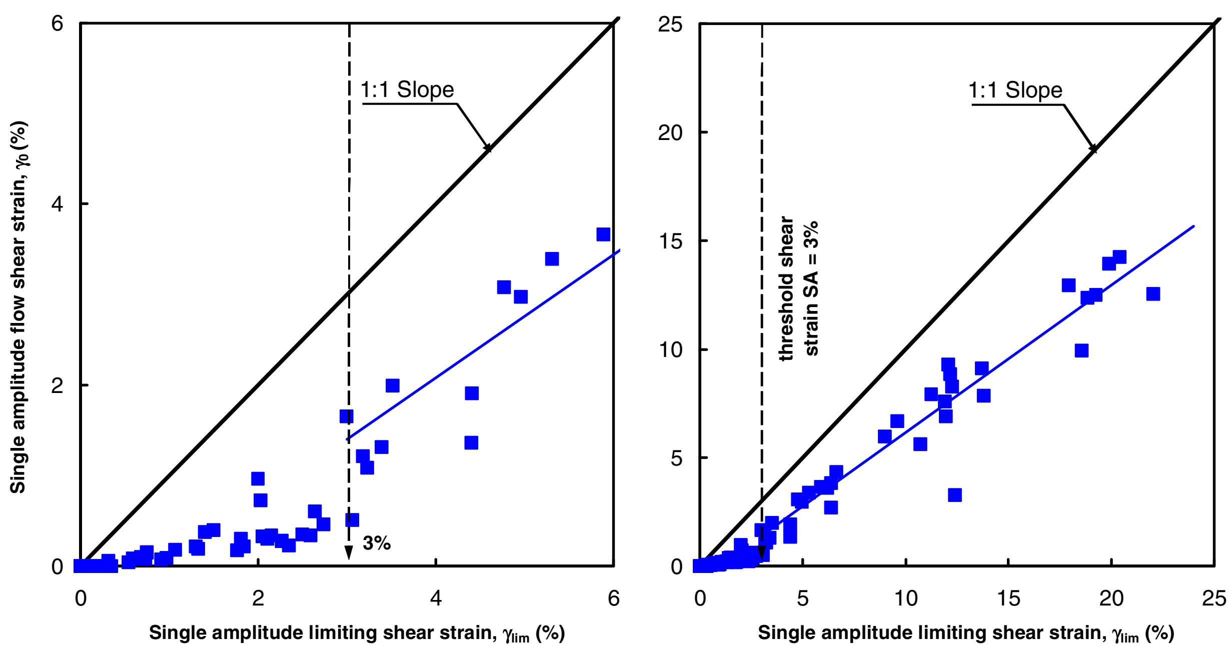

Figure 5: Correlations between the flow strain and the limiting shear strain

Pore Water Pressure at Liquefaction Triggering

As previously discussed, both pore-pressure-based and strain-based liquefaction criteria have been used extensively in presenting laboratory liquefaction data. However, quantitative studies of relationships between these two sets of definitions have been scarce. The liquefaction criteria selected in this study is 6% double amplitude cyclic shear strain in tests under "level ground conditions". Thus it is of interest to present the corresponding pore pressure ratios (r_u) at the point of liquefaction triggering as defined by this shear strain criterion.

Figure 6 shows the recorded pore pressure buildup in samples under 80 kPa initial effective vertical stress, compared with the pore pressure "envelope" reported by Lee and Albaisa [10]. Among the 81 samples tested, the lowest value of r_u at liquefaction triggering was 0.78, and the highest value of r_u was 0.99. The overall occurrence rate of pore pressure ratios r_u is presented in Figure 7, which shows most samples (approximately 80 percent) liquefied with r_u of 0.95 or higher, but a non-trivial number of samples "liquefied" with r_u of less than 0.95. This indicates that although in most samples the difference between pore-pressure-based and shear-strain-based criteria is relatively small, these two categories of criteria can show poor agreement for others. Figure 8 presents the occurrence rates of r_u of samples with different densities. From which it can be seen that the lower values of r_u occurs primarily in denser samples (e.g. *Dr* of 60 % or higher) while looser samples always liquefy with high values of r_u (0.9 or higher).

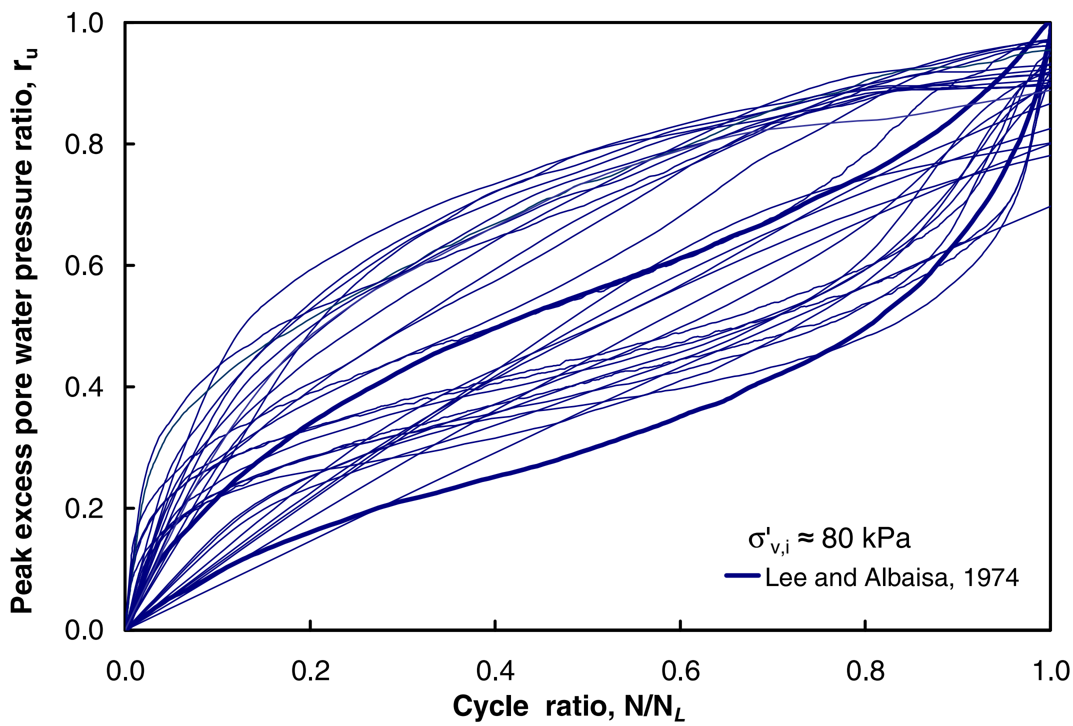

Figure 6: Pore water pressure buildup in cyclic simple shear tests

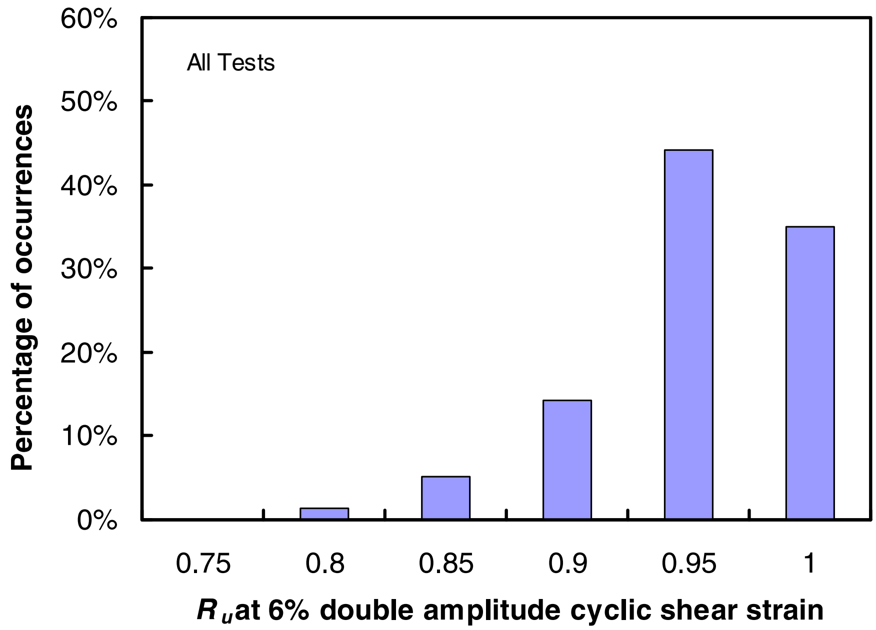

Figure 7: Distribution of pore water pressure ratio at liquefaction triggering

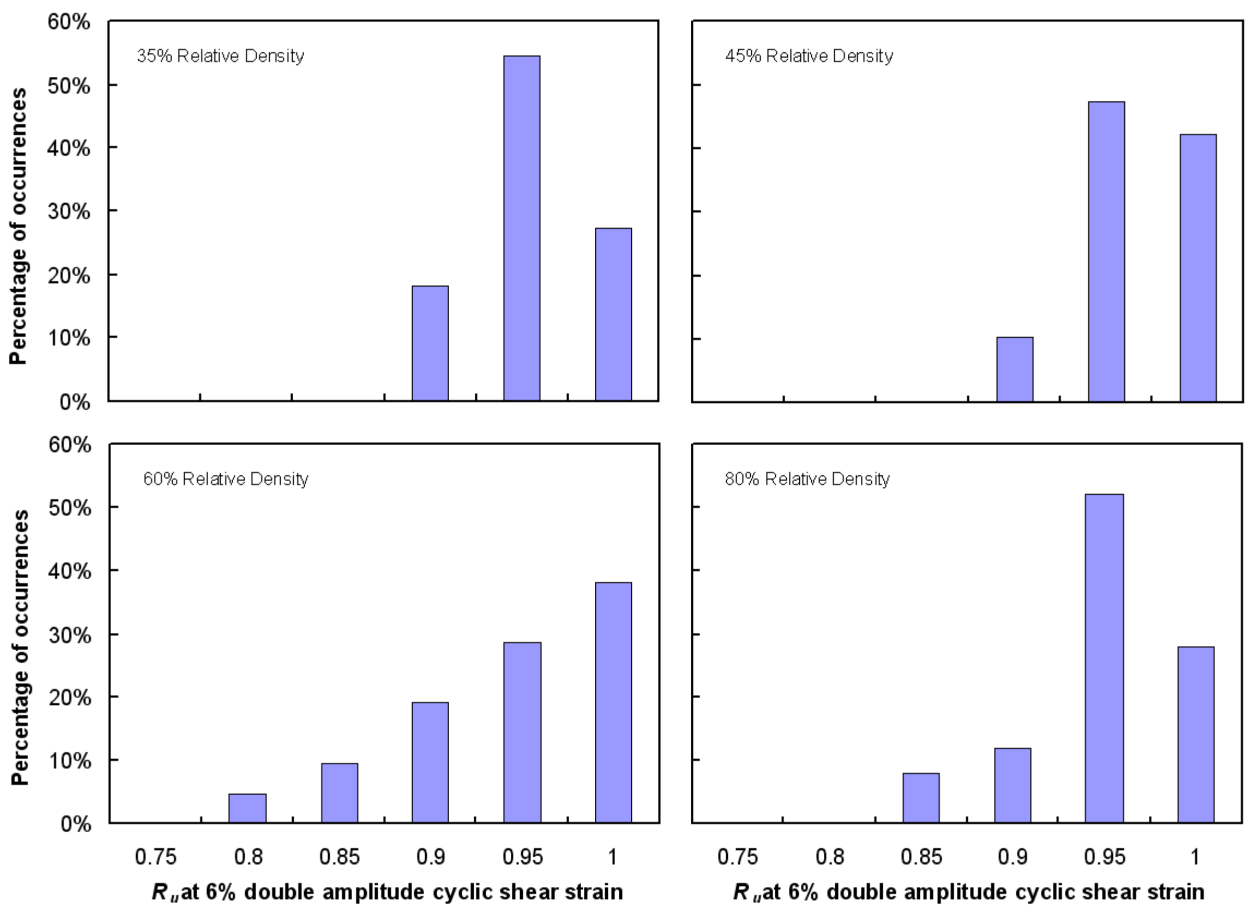

Figure 8: Occurrence rates of pore water pressure ratio at liquefaction triggering

Figure 9 summarizes the recorded r_u at liquefaction triggering for individual tests performed in this experimental study. Despite considerable scatter among the data, there is a persistent trend that the r_u at onset of liquefaction increases as the number of cycles required to reach the 6% DA shear strain amplitude increases (if all other conditions were identical). The correlations shown in Figure 9 may be used to approximately estimate r_u at the occurrence of 6% DA cyclic shear strain amplitude in Monterey No. 0/30 sand.

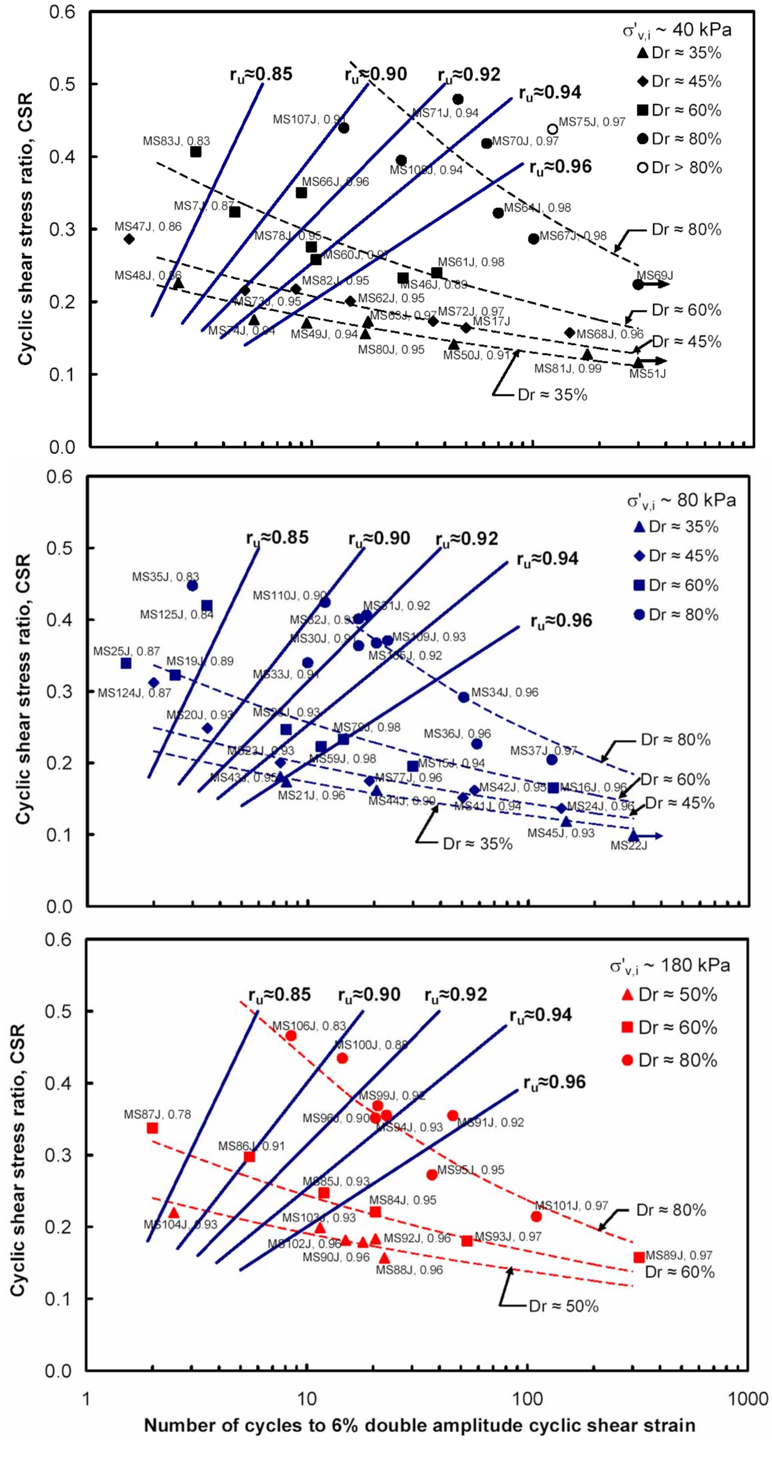

Figure 9: Relationships between pore water pressure ratio, relative density, and cyclic stress ratio

If the occurrence of maximum pore pressure ratio ($r_{u,,max}$) of 1.0 is used as the criterion for initial liquefaction, it naturally eliminates the issue of post-liquefaction pore water pressure generation. In this and other studies that use shear-strain-based liquefaction triggering criteria, however, excess pore water pressure can keep increasing after the threshold shear strain has been reached (and liquefaction is considered to have triggered). In fact, in all the tests that liquefied with r_u of less than 0.95, the excess pore pressure ratios continued increasing with additional loading cycles until reaching the maximum asymptotic value of the pore pressure ratio known as $r_{u,lim}$, the limiting pore pressure ratio. In this study, the limiting pore pressure ratio was found to be at least 0.95 or higher, regardless of sample densities and shear stress levels. This phenomenon should be distinguished from that under "sloping ground conditions", in which the limiting pore pressures in sands were suppressed by the static "driving" shear stress and may not reach full pore pressure ratios (r_u = 1.0) no matter how many additional loading cycles are applied after liquefaction triggering. The limiting pore pressure in cyclic simple shear tests under "sloping ground conditions" were discussed extensively by Boulanger et al. [11], and more recently by Kammerer et al. [12,13]. In a companion bi-directional simple shear testing study, several sand samples liquefied with pore pressure ratio, $r_{u,\,lim}$ as low as 0.6 under multi-directional loading. Details information on that testing program and its results is provided in [12,14].

CONCLUSIONS

Several conclusions regarding the liquefaction triggering criteria may be drawn on the basis of this laboratory experimental study:

1) The flow shear strain condition distinguishes the pre-liquefaction and post-liquefaction deformation behavior of cohesionless soils, and may serve as an indicator of initiation of liquefaction. The flow shear strain takes place only in a state that is characterized by very low shear stiffness and an excess pore water pressure ratio of approximately 1.0.

2) The flow shear strain behavior is initialized as the maximum single amplitude shear strain approaches approximately 3%. This threshold strain amplitude appears relatively insensitive to material properties (e.g.: sand types and relative densities) and to the testing conditions (e.g.: loading modes and consolidation stresses).

3) When strain-base liquefaction triggering criteria, such as the 6% DA maximum shear strain used in this study are selected, soils do not necessarily liquefy with a full excess pore water pressure. Instead, soil liquefaction may be triggered with r_u of 0.8 or even lower under certain conditions. It was found that the value of $r_{u.lim}$ at liquefaction trigging varies with soil densities and loading conditions. Denser sands subjected to larger shear stress ratios appear to generate the lowest pore pressure ratios r_u at the onset of liquefaction.

4) After soil liquefaction triggering (defined by reaching a threshold shear strain amplitude), the excess pore pressure may continue rise until reaching a limiting pore pressure ratio $r_{u.lim}$. The value of $r_{u.lim}$ was found to be approximately 1.0 under uni-directional, "level ground" loading conditions. However, under bi-directional, "sloping ground" conditions, the value of $r_{u.lim}$ could be significantly less than 1.0.

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