LIQUEFACATION OF SILTS AND SILT-CLAY MIXTURES

SHAMSHER PRAKASH[*]
Emeritus Professor,
Department of Civil Engineering,
University of Missouri
Rolla, MO- 65409 USA
Emails Prakash@umr.edu

Vijay K. Puri
Professor
Department of Civil Engineering
Southern Illinois University
Carbondale, IL- 62901, USA
Puri@engr.siu.edu

ABSTRACT

Low plasticity silts and silty clays occur extensively in the Central United States, India and China. For evaluating their liquefaction potential during an earthquake, no accepted guidelines are available based on their density, void ratio, plasticity index, standard penetration values, or any other simple soil property. Their liquefaction behavior is not properly understood at present (2003) and is often confused with that of sand-silt mixtures.

The liquefaction behavior of clean sands has been studied extensively. If fines are added to sands, their resistance to liquefaction decreases if the soils are tested at the same void ratio. However if a sand-fines mixture has the same standard penetration value $(N_1)_{60}$, the addition of fines increases the liquefaction resistance.

In the case of silts and silt-clay mixtures, not enough studies have been performed. It is hypothesized that clay imparts plasticity to the silt, which may itself be non-plastic. A small amount of clay in a matrix of non-plastic silt imparts a little plasticity to it. It is generally believed, though erroneously, that addition of clay or plasticity to the silt increases the resistance of silts against liquefaction. Based on limited laboratory test data on both reconstituted and undisturbed samples it has been observed that for a plasticity index (PI) in the range of 2-4%, the liquefaction resistance of silt decreases with increasing plasticity. Critical examination of relevant data suggests that there is a threshold PI value of soil sample below which the liquefaction resistance decreases with increasing PI and above which the liquefaction resistance starts increasing. Specific values for this PI are not known at this time. Also, other relevant data from literature on both undisturbed and reconstituted samples has been critically examined and this also substantiates the above hypothesis.

Pore water pressures and liquefaction in silt and silt-clay mixtures are discussed, and the influences of plasticity index on their cyclic strength are reviewed critically. It is concluded that considerable additional work is needed to fully understand the liquefaction behavior of these soils.

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Key Words: Liquefaction, Mixtures, Silt, Clay, Fine Content, Plasticity

[*] Corresponding auhthor

INTRODUCTION

Cyclic shear stresses develop in the soil deposits as a result of passage of seismic waves through them. These stresses may result in progressive build up of pore water pressure in certain types of soils in a saturated state. Cohesionless soils of loose and medium density have a tendency to compact under vibrations leading to decrease in the inter-granular space. This tendency for volume decrease gives rise to increase in pore water pressure. The progressive build up of pore water pressure may eventually become large enough resulting in complete loss of shear strength accompanied by large deformations and failure. The evaluation of liquefaction potential of soils at any site requires a combination and interaction of two sets of parameters namely the cyclic loads due to seismic action and soil properties which describe the soil resistance under those loads.

The phenomenon of liquefaction has been extensively studied for the case of cohesionless soils under seismic loading conditions. The state of the art on liquefaction behavior of cohesionless soils has progressed to a stage that reasonable estimates of liquefaction potential can be made based on laboratory investigations or on simple in-situ test data such as standard penetration values (N_1 or $(N_1)_{60}$) or on cone penetration data, and the experience during the past earthquakes,(Arulanandan et al , 1986; Mitchell and Tseng, 1990; Robertson, 1990; Robertson and Campnella ; 1985 Prakash,1981; Seed,1976,1979; Seed and Idriss, 1967,1971, 1981; Seed and DeAlba, 1986; Seed , Idriss and Arango , 1983; Seed and Lee, 1966; Seed and Harder, 1990; Seed et al , 1985,1988; Idriss'1991;Youd and Idriss ,2001;Idriss et al,2001).

The cyclic stress approach (Seed and Idriss ,1981) and the cyclic strain approach (Dobry et al, 1982) are commonly used for evaluation of liquefaction potential of sands.

LIQUEFACTION OF SILTS AND SILT -CLAY MIXTURES

Most earlier studies on liquefaction phenomenon were on sands and fine grained soils such as silts, clayey silts and even sands with fines were considered non-liquefiable. However, the observations following the Haicheng (1975) and Tangshan (1976) earthquakes indicate that many cohesive soils had liquefied (Wang, 1979, 1981, 1984). These cohesive soils had clay fraction less than 20%, liquid limit between 21-35%, plasticity index between 4 and 14 and water content more than 90% of their liquid limit (Wang, 1979).

Kishida (1969) reported liquefaction of soils with upto 70 % fines and clay fraction of 10% during Mino-Owar, Tohankai and Fukui earthquakes. Tohno and Yasuda (1981) reported that soils with fines up to 90% and clay content of 18 % exhibited liquefaction during Tokachi –Oki earthquake of 1968. Miura et al (1995) noted liquefaction of soils with up to 48 % fines and 18 % clay content during the Hokkaido Nansai –Oki earthquake of 1993. Gold mine tailings liquefied during the Oshima- Kinkai earthquake in Japan (Ishihara, 1984). These tailings had silt sized particles and liquid limit of 31%, plasticity index of 10 % and water content of 37 %.

The upstream flow slide failure of Lower San Fernado Dam during the San Fernado earthquake of 1971 occurred due to liquefaction of 'silt like' fill sands (Seed et al, 1989). Youd et al (1985) reported liquefaction of silt –clay materials at the Whiskey Spring during the Idaho earthquake of 1983. There is evidence of extensive liquefaction in the Mississippi embayment during the 1811-12, New Madrid earthquake (Wesnousky et al, 1989).

Following the observation of liquefaction of soils with fines, some research activity started in this direction, of course at a slow pace and some liquefaction criteria were also developed based on the experience during the Chinese earthquakes. Wang (1981) suggested that three types of soils may be susceptible to liquefaction. These are: (1) Saturated sands with SPT values lower than a critical value as a function of earthquake intensity and effective overburden pressure; (2) saturated slightly cohesive silty soils with water content lower than 90 % of their liquid limit and liquidity index greater than 0.75; and (3) soft clays with liquidity index of more than 0.75, SPT values ‹ 4 and sensitivity › 4. Major criticism of this criteria has been that ground motion characteristics triggering liquefaction are nor specified and any soil meeting these criteria will be considered susceptible to liquefaction irrespective of the level of ground shaking.

Seed et al (1983) found that some soils with fines may be susceptible to liquefaction. Such soils (based on Chinese criteria) appear to have the following characteristics:

Percent finer than 0.005 mm (5 microns) ‹15%
Liquid limit ‹ 35 %
Water content › 90 % of liquid limit.

Seed et al (1983) suggested that the liquefaction characteristics of soils meeting the above criteria should be ascertained by tests.

It is thus seen that even though silts and silt-clay mixtures occur in several parts of the world including China, India and United States, their liquefaction characteristics have not received enough attention. Soils with fines were generally, though erroneously, considered to possess higher resistance to liquefaction compared to clean sands. In fact when fines are added to sand, the liquefaction potential decreases if soils are tested at the same void ratio (Troncoso, 1990). However, if the sand-fines mixture has the same standard penetration test value $(N_1)_{60}$, the addition of fines, results in a decrease in liquefaction potential (Seed et al, 1985). The liquefaction susceptibility of a soil with fines should be expected to depend not only on the amount of fine but also on the nature of the fines (Ishihara, 1993). A small amount of clay in a matrix of non-plastic silt imparts a slight plasticity to silt. The behavior under cyclic loading may then be different depending on the increase in the plasticity. Ishihara and Koseki (1989a,b) observed that the cyclic strength does not change much for low plasticity range (PI ‹10) but increases thereafter. The behavior of silts and silt clay mixtures in the low plasticity range is of particular interest and should be ascertained to see if these soils are vulnerable to liquefaction.

SOME CONFLICTING OPINIONS ABOUT EFFECT OF FINES ON LIQEFACTION

There are several research findings worth mentioning on the effect of fines on liquefaction potential of soils. Some of these opinions are conflicting and some time may be confusing.

1. Seed et al (1985) have recommended that for sands containing less than 5% fines, the effect of fines may be neglected. For sands containing more than 5% fines the liquefaction potential increases as shown in Fig. 1. Neglecting the effect of fines should therefore be expected to lead to conservative estimates of liquefaction potential.

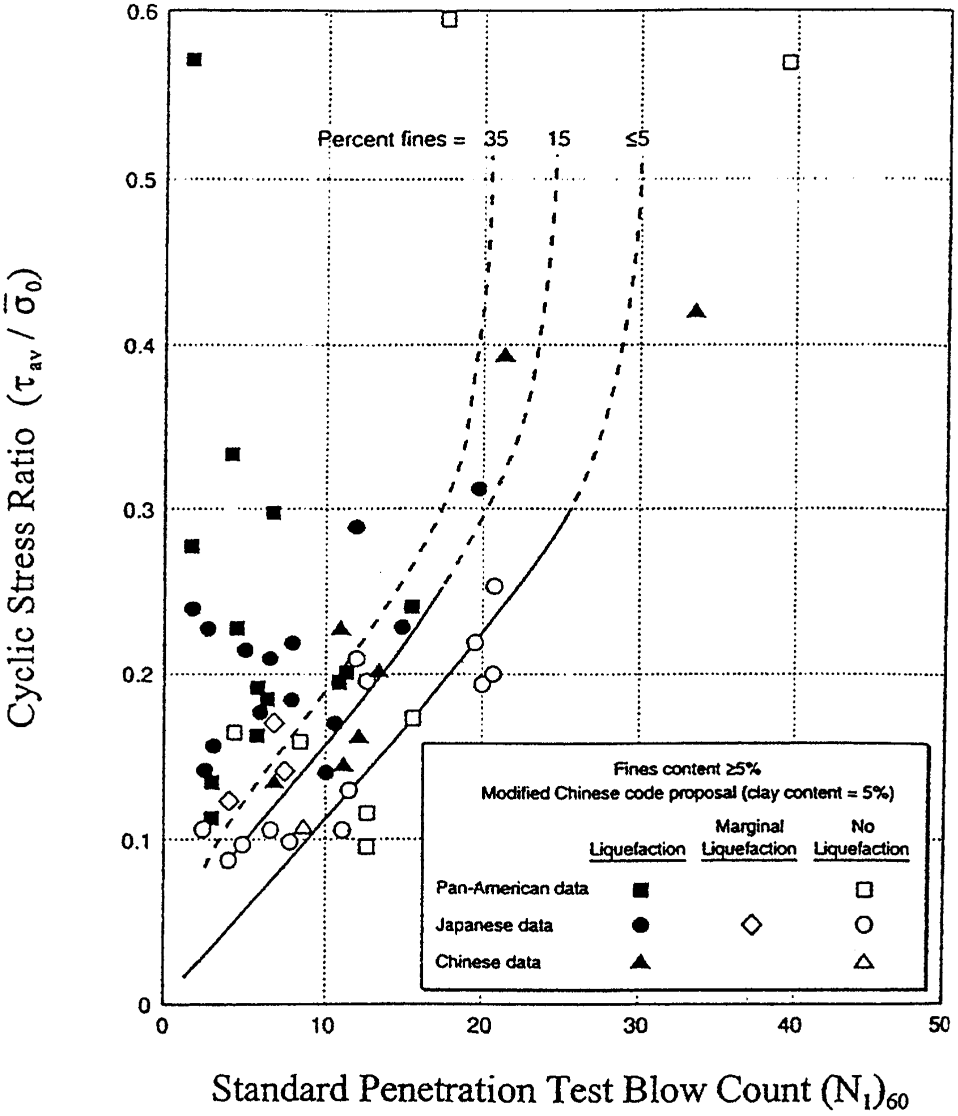

Figure 1 Relationship between Stress Ratio Causing Liquefaction and $(N_1)_{60}$ values for Silty Sand for M = 7.5 (after Seed et al. 1985)

2. Zhou (1981) made an interesting observation based on CPT tests on silty sands at one site and clean sands at another site that an increase in the fines content in sand decreases the CPT resistance but increases the cyclic resistance of the soil. No explanation is given for this peculiar behavior.

3. Zhou (1987) observed that if the clay content P_c in a soil is more than the critical percentage P_c^*, the soil will not liquefy. The value of P_c^* are related to the intensity of earthquake I as follows:

Intensity I	7	8	9
P_c^* (%)	10	13	16

4. Ishihara and Koseki (1989) had suggested that low plasticity fines (PI‹ 4) do not influence the liquefaction potential. They did not consider the effect of the void ratio in their analysis.

5. Finn (1991) made the following observation about the effect of fines in sand in developing equivalent clean sand behavior. If the void ratio of silty sand and clean sand is the same the liquefaction resistance decreases. If the comparison is made at the same $(N_1)_{60}$, the effect of fines is to increase the liquefaction resistance. If comparison is made using the "the same void ratio in sand skeleton" as the criteria, then there is no effect on the cyclic strength provided the fines can be accommodated in the sand voids.

It is thus seen that there are different conclusions about the effect of fines on liquefaction resistance arising from different comparison criteria. It may be worth while to elaborate the 'Chinese criteria' for liquefaction of fine grained soils here. According to Wang (1979), the following criteria are recommended by the Chinese Code for Aseismic Design of Hydraulic Structures. According this criteria any silty soil which contains less than 15 % to 20% clay particles (smaller than 5 μm size) and has plasticity index more than 3, can liquefy during a strong motion earthquake if its water content is greater than 90 % of its liquid limit.

The Chinese practice of determining the liquid and plastic limits, water content and clay fraction differs somewhat from the ASTM procedures followed in USA and some other countries. Finn (1991, 1993) and Perlea et al (1999) suggested the following adjustments of the index properties as determined using the US standards, prior to applying the Chinese criteria:

1. decrease the fines content by 5%
2. increase the liquid limit by 1% and
3. increase the water content by 2

Figure 2, further illustrates the Chinese criteria modified as discussed above and applied to the index properties determined following the US or similar standards. The soils that fall below the line defined by w = 0.87 LL and LL= 33.5 in Fig. 2 will be considered as susceptible to liquefaction.

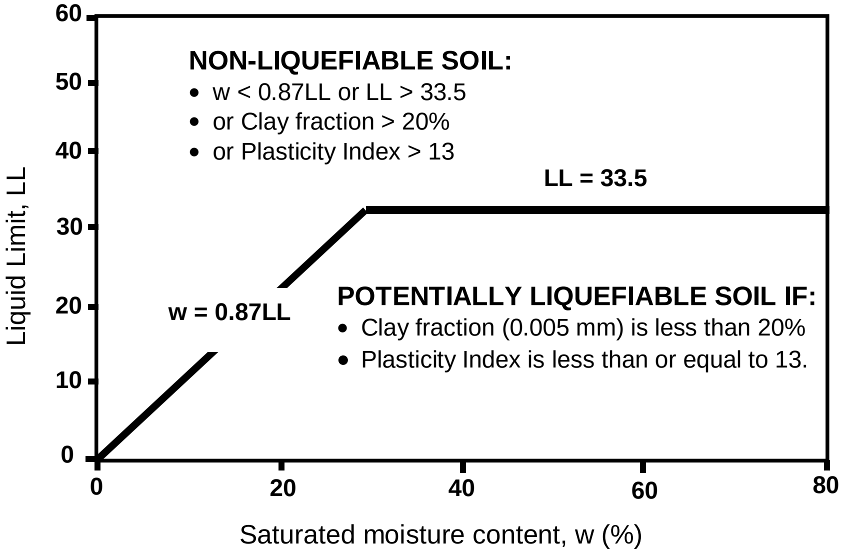

Figure 2. Chinese Criteria Adapted to ASTM Definitions of Soil Properties (Perlea, Koester and Prakash, 1999)

SIGNIFICANT DEVELOPMENTS :LIQUEFACTION OF FINE GRAINED SOILS

We will now discuss and evaluate the present (2003) information on liquefaction of sand and sand-silt mixtures and silt and silt-clay mixtures.

Sands and Sand Silt Mixtures

Lee and Fitton (1968) tested sands and silts with up to 95 % fines and confirmed the susceptibility of fine grained soils to liquefaction.

Seed et al. (1985) developed a liquefaction assessment chart relating normalized cyclic stress ratio $\tau_{av} / \bar{\sigma}_0$ versus $(N_1)_{60}$ based on field data, where τ_{av} is average cyclic shear stress at a point in the soil mass, $\bar{\sigma}_0$ is effective overburden pressure at the same point, and $(N_1)_{60}$ is normalized SPT blow count at the same point.

Fig. 1 shows the boundary line between liquefiable and non-liquefiable level sandy sites with less than 5%, and with 15 and 35% fines for an earthquake of magnitude of 7.5. A detailed study of Fig. 1 suggests that (Guo and Prakash, 1999) :

1. The changes of "CSR increase" imply changes in the pore water pressure build up in the soil. At lower SPT values, i.e., loose sand, fines in the soil leads to higher pore pressure than in the pure sand. When the sand is dense with higher fines content, plasticity is introduced. This imparts cohesive character to soil, and therefore the resistance to liquefaction increases rapidly.
2. CSR increase is the lowest with $(N_1)_{60}$ for soils containing fines of about 10%. For $(N_1)_{60}$ greater than 15, the rate of increase of CSR is substantially higher in

sands with higher fines content. This indicates that both the content and nature of fines (such as plasticity index) control the value of CSR.

However, there are not enough data to generalize the above observations. Also Seed et al. (1985) do not describe plasticity characteristics (plasticity index) of the fines in Fig. 1.

An earlier review of the same issue (NRC 1985) concluded the following:

1. It is clear that, for soils with the same $(N_1)_{60}$, ignoring the presence of fines can be conservative, and that the fine content should be noted in evaluating the liquefaction susceptibility of a sand deposit.
2. However, it is still not possible to evaluate the likelihood of liquefaction of a silty sand with the same confidence as for clean sand.

Troncoso (1990) compared the cyclic strength of tailing sands with different silt contents ranging from 0 to 30 % at a constant void ratio of 0.85 (Fig. 3). It was found that the cyclic strength decreased with the increase of fine content. This conclusion is in apparent contradiction with that of Seed et al. (1985). Note, however, that the difference of results between Seed et al. (1985) and Troncoso (1990) is due to the different criteria used in their studies. In Troncoso's study, the cyclic shear strength of tailing sands was examined at a constant void ratio, while in the study of Seed et al., cyclic stress ratio was investigated at the same SPT values.

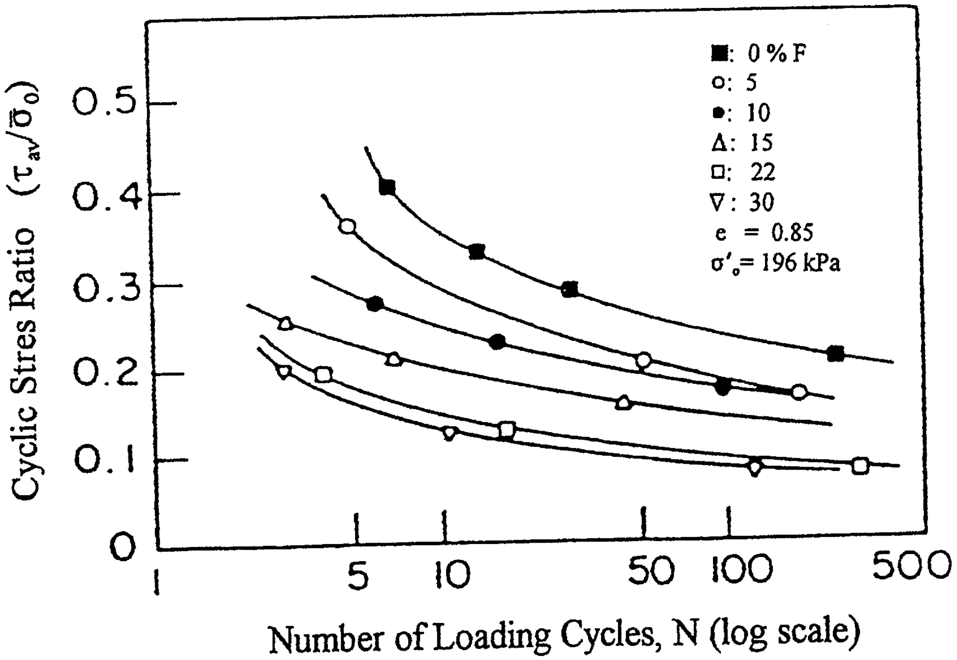

Figure 3 Variation of Cyclic Strength with Fine Content at Constant Void Ratio (after Troncoso, 1990)

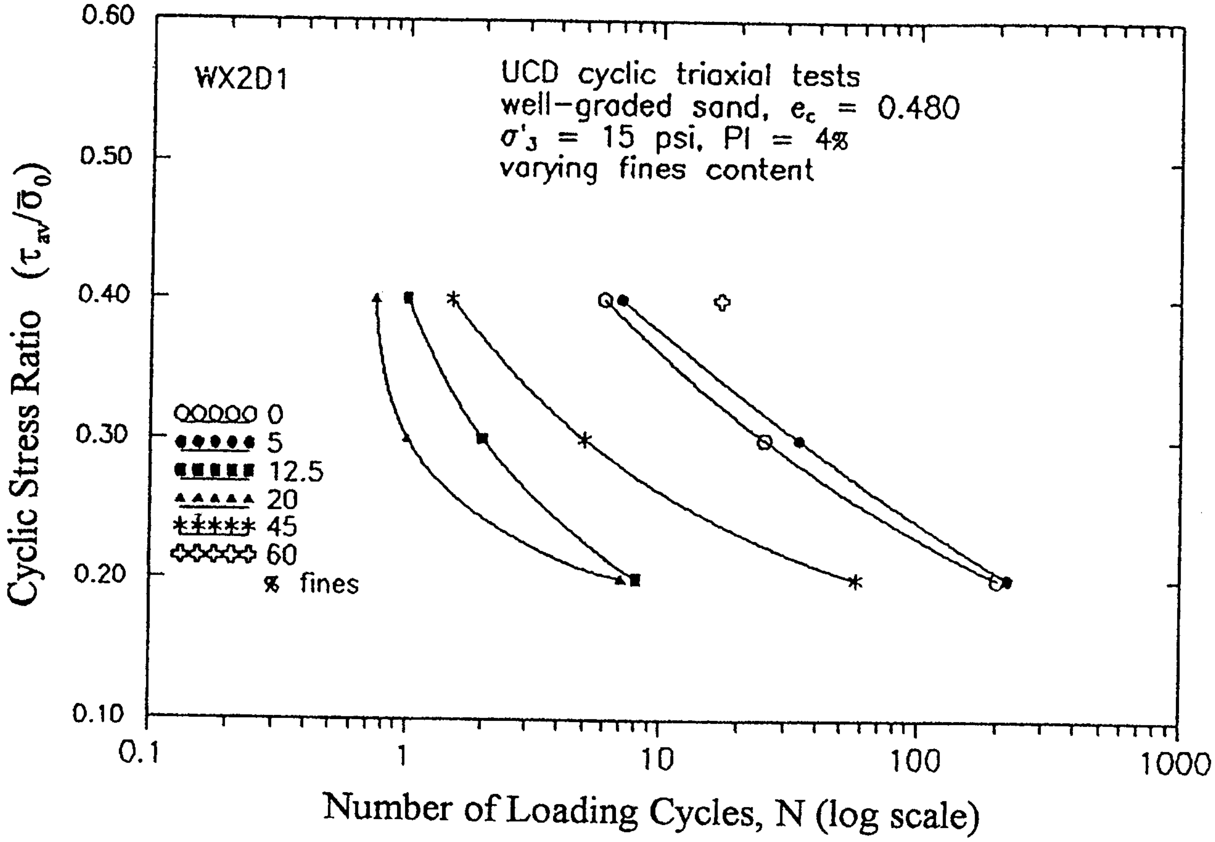

Figure 4 Cyclic Stress Ratio for Well-Graded Sand Mixtures, with Index Properties and Test Conditions Shown (after Chang 1990)

The influence of gradation and index properties of the fine fraction on liquefaction resistance and pore pressure were analyzed by Koester (1993) using the data from Chang (1990) on reconstituted samples of sand silt mixtures. Fig. 4 shows the test results on samples of medium sand, with varying fines content at a void ratio of 0.48. These results are similar to those of Troncoso (1990) for specimens containing up to 20% fines in the sand. If the fine content exceeds 20%, the cyclic stress ratio increases.

Prakash and Sandoval (1992) examined the data of Ishihara and Koseki (1989). Sixteen reconstituted samples of Toyoura sand were prepared with a different percentage of kaolinite and Kanto loam and tailings sand. Only three samples, with plasticity index of 2-4%, are of interest. Detailed properties and test results are summarized in Table 1. The CSR varies only marginally (0.20 or 0.22-0.24) with increase in PI of the material from 2 to 4%. However, the void ratios after consolidation (e_c) of sample 1 with PI = 4% is very low (0.58-0.62), while that of samples 2 and 3 with PI = 2% is 0.73-0.78. If samples 2 and 3 were tested at the void ratio of 0.60, its CSR will be higher than 0.22-0.24. Therefore, the void ratio effects are also reflected in these results. Ishihara and Koseki (1989) stated that, in their investigation, void ratio is not considered as an independent parameter, and it might be desirable to investigate the effects of fines content on the cyclic strength under identical void ratios.

Table 1: Properties of Different Low-Plasticity Soil Samples (after Ishihara and Koeski 1989)

Sl. No.	Soil	Composition	PI	e_c	CSR
1	Kaolin (20/48)	Kaolin-tailings-sand	4	0.60	0.240
2	Kaolin (18/66)	Kaolin-tailings-sand	2	0.78	0.220
3	Loam (10/47)	Kanto loam-tailings-sand	2	0.73	0.200
4	Tail (9/44)	Tailings-sand	0	0.56	0.190

Note: e_c = void ratio after consolidation; CSR = cyclic stress ratio causing 5% strain in 20 cycles.

Discussion

The following can be concluded from the above discussion (Guo and Prakash 1999)

1. Ignoring the presence of fines in sand can be erroneously interpreted as conservative according to Seed's (1985) liquefaction assessment chart. The fines content should not be neglected in evaluating the liquefaction susceptibility of a sand deposit.
2. Troncoso (1990) and Koester (1993) indicated that the cyclic strength of sand decreased with increasing silt content up to 20-30% by weight. If the fine content goes beyond 20%, cyclic stress ratio of sand increases with fines. There should be a lowest value of cyclic stress ratio between fines content of 20-30% of the soil's weight.
3. There is more scatter in Koester's (1993) data than in that of Troncoso (1990). Therefore, no quantitative conclusions can be drawn relating the decrease in CSR with fines content.
4. Further systematic investigations are needed to study these effects.

Silts and Silt Clay Mixtures

For clean non-plastic saturated silts , the behavior under cyclic loading and nature of generation and buildup of pore-pressure should be expected to be about the same as that for clean sands. If, however a small fraction of highly plastic material is added to non-plastic silt, one of two things may happen:

1. The rate of buildup of pore water pressure may increase because the addition of clay fraction will reduce the hydraulic conductivity of the soil , which may lead to higher pore water pressures.
2. Plasticity of clay fraction will impart some cohesion to the soil which may increase the resistance of the soil to liquefaction.

It is the interplay of these two factors that will determine whether the liquefaction resistance of silt-clay mixtures increases or decreases compared to that of the pure silts.

Typical test data on liquefaction of undistributed samples of silts and silt-clay mixtures has been critically examined by Guo and Prakash (1999).

El Hosri et al. (1984) performed tests on six undisturbed samples of silt obtained from depths of 20 and 40 m from two sites. The soils tested were mainly silts with traces of clay or clayey silt (ML-Cl or ML-MH) with PI of 5-15%, except for one sample that consisted of silty sand. Table 2 lists the values of cyclic stress ratios (CSR) causing liquefaction in 20 cycles. The void ratios of the samples are also given in Table 2. .

The following observations were made based on data in Table 2: (Guo and Prakash 1999):

Table 2: Characteristics of the Specimens and Test Results (El Hosri et al. 1984)

Sample No.	Gradation <2 µm%	PI	e_o of sample	Soil type	Number of cycles	CSR
Site (I) A	0	-	0.644	SM	20	0.295
B	19	5	0.478	ML-CL	20	0.32
C	21	8	0.548	ML	20	0.265
Site (II) D	17	9	0.654	ML	20	0.305
E	28	15	0.914	ML-MH	20	0.326[a]
F	15	6.5	0.600	ML-CL	20	0.335

[a] Extrapolated Value

1. Sample B, with e_0 of 0.478 and PI of 5%, failed at CSR of 0.32. CSR of Sample A (PI of 0) with e_0 of 0.644 is 0.295. Although Sample B is much denser than Sample A, CSR for Sample B is only slightly higher than that of Sample A. Therefore, it can be inferred that if the void ratio of Sample B is close to that of Sample A, CSR at failure for Sample B is likely to be much less than the CSR for Sample A. This shows that introduction of a small amount of plasticity lowers the CSR at liquefaction.
2. Sample C, with e_0 of 0.548 and PI of 8%, failed at CSR of 0.265. Again, even though Sample C is denser than Sample A, the resistance to liquefaction of this sample is smaller than that of Sample A. Also, the PI of Sample C is more than the PI of Sample B, but CSR at liquefaction of Sample C is less than that of Sample B. However, this comparison is not reasonable, because the void ratio of Sample C is greater than that of Sample B.

It is seen that there are two variables, PI and e_0, that control the CSR for initial liquefaction. Therefore, in order to study the effect of one variable, PI, it was decided to normalize the CSR for all samples for a common void ratio of 0.644. It was assumed that the CSR for initial liquefaction is inversely proportional to the void ratio (Guo, 2003). Other relationships between CSR and void ratio were also considered, but their effect on the CSR-number of cycles relationship was found to be insignificant.

In Table 3, cyclic stress ratios of all the samples are normalized to the initial void ratio of 0.644 for inducing initial liquefaction in 20 cycles. The listing was rearranged according the values of the plasticity index of the samples. In Fig. 5, this normalized CSR is plotted against PI for different values of number of cycles inducing initial liquefaction . It is seen from the plots that the cyclic stress ratio of undisturbed samples first decreases with an increase in the plasticity index up to a PI of about 5%, and then increases with an increasing PI. For Sample E with PI = 15, the cyclic stress ratio is even higher than that for Sample A with no plasticity.

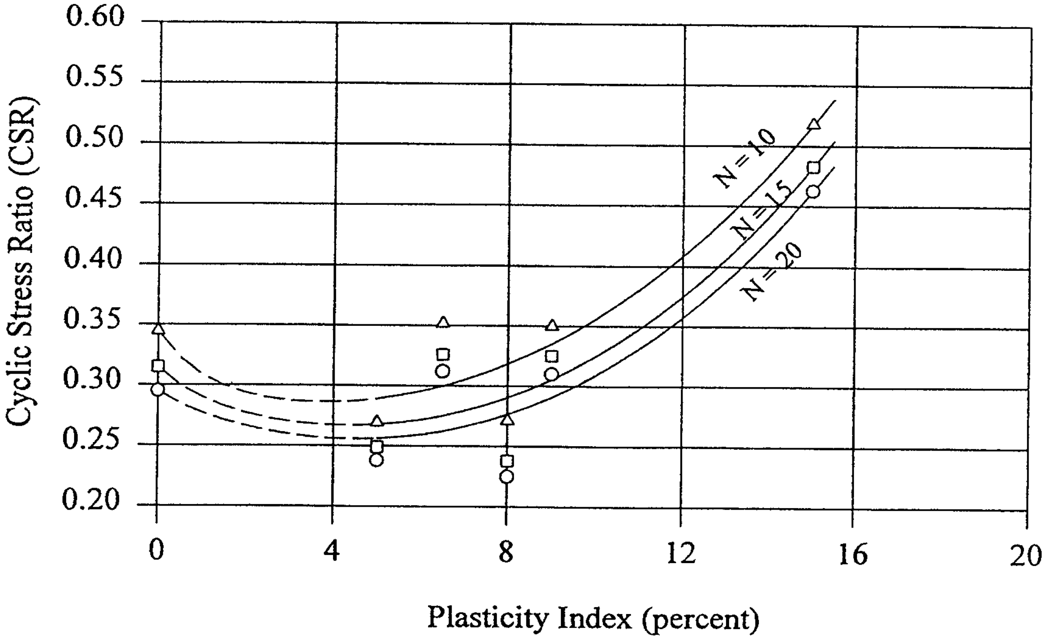

Figure 5 Normalized cyclic Stress Ratio versus plasticity Index on Undisturbed samples (Data of El Hosri et al 1984)

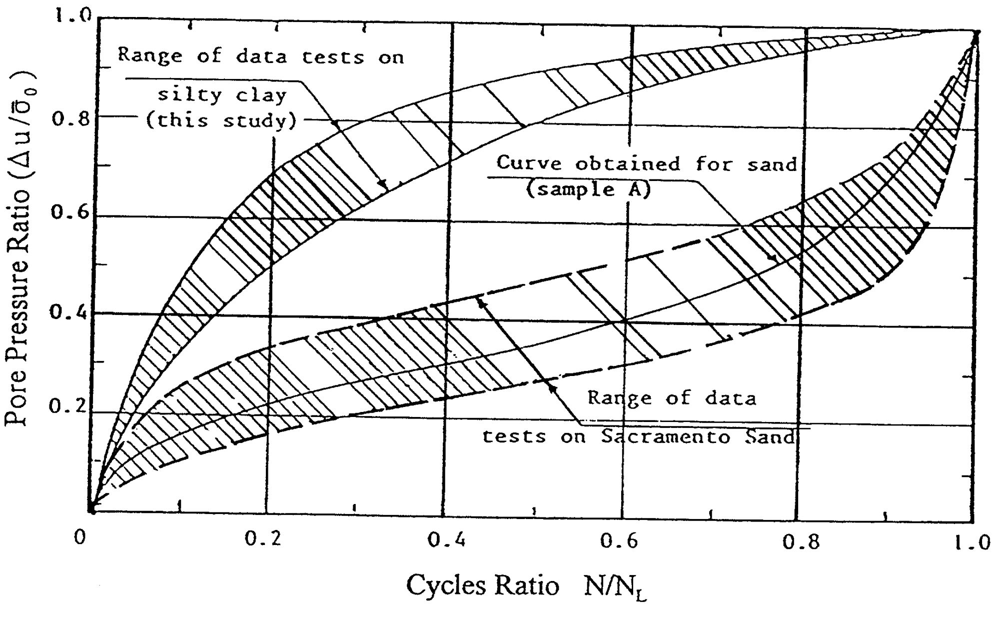

Figure 6 Rate of Pore Pressure Buid up in Cyclic Triaxial Tests on Undisturbed Samples (After El-Hosri et al. 1984)

. The test results presented above clearly show that the plasticity index has a definite influence on liquefaction resistance of silt-clay mixtures.

Table 3: Normalized Test Results for Various Numbers of Cycles

Sample No.	PI	Number of Cycles	CSR	Number of Cycles	CSR	Number of Cycles	CSR
A	-	20	0.295	15	0.315	10	0.345
B	5	20	0.238	15	0.249	10	0.269
F	6.5	20	0.312	15	0.326	10	0.352
C	8	20	0.225	15	0.238	10	0.271
D	9	20	0.310	15	0.325	10	0.350
E	15	20	0.463	15	0.483	10	0.518

Note: CSR normalized to initial void ratio $e_0 = 0.644$

In Table 3, the normalized CSR for liquefaction with number of cycles N = 10, N = 15 are also listed and plotted in Fig. 5. It is clear that an increase in the plasticity index results in a decrease in liquefaction resistance of silty soil in the low plasticity index range. In the middle and/or high plasticity index range, an increase of the plasticity index increases the liquefaction resistance for undisturbed silt-clay mixtures.

Figure 6 shows the relationship between excess pore-pressure ratio $\Delta u/\bar{\sigma}_0$ and cycle ratio N/N_1 for five undisturbed silt samples (Samples B-F in Tables 2 and 3) and Sacramento sand (Sample A), where N_L is the number of cycles required to cause initial liquefaction, and N is the number of cycles required to develop excess pore water pressure Δu. This figure shows that the excess pore-pressure ratio increases rapidly at the beginning of cyclic load application on the clayey silt specimens. This increase in pore-pressure is at a much faster rate than that observed for the Sacramento Sand (Sample A). The above test results of El Hosri et al. (1984) show that silty soils are vulnerable to a pore water pressure buildup substantially different from that for sands. The pore-pressure buildup is much faster at the beginning of cyclic loading in a silt-clay mixture than in sand. This is similar to the observations of Puri (1984) presented next.

Discussions

On the basis of studies on undisturbed samples, the following was concluded: (Guo and Prakash 1989)

1. Tests indicate that the pore water pressure buildup in silt-clay mixtures are remarkably different from that for sands.

2. The increase of the PI decreases the liquefaction resistance of silt-clay mixtures in the low range of plasticity. In the high plasticity range, the liquefaction resistance increases with an increasing PI.

3. For silt-clay mixtures, the criteria used to define the stage of initial liquefaction for sands may not be applicable, because of the difference in pore pressure buildup and deformation relationship as compared with those of sand.

The authors conducted an experimental investigation of the cyclic strength of undisturbed and reconstituted samples of a loessial soil (Puri ,1984, 1990: Prakash and Puri ,1982). Dynamic triaxial tests on 73.65 mm (diameter) and 147.3 mm (high) were conducted for this purpose. Based on its grain size and plasticity, the soil used for tests may be classified as clayey silt. The index properties of the soil are given below:

Percent finer than 75 μ (0.075 mm) = 93.0 – 98.0 %
Natural water content = 18 -26 %
Liquid limit = 32.0 – 36.0 %
Plastic limit = 21.0 – 25.0 %
Plasticity index = 9 -14 (mostly ≈ 10)
Clay content (‹ 2μ) = 2.0 – 7.2 %
Dry unit weight = 14.7 – 15.2 kN/m^3
(93.5 – 96.5 lb/ft^3)
Specific gravity of soil particles = 2.71
Particle size D_{50} = 0.06 mm
Uniformity coefficient ≈ 1

The effect of plasticity index on cyclic strength was also investigated on reconstituted samples by altering the clay fraction and samples were made with plasticity index values 10, 15 and 20. The plasticity index in this study refers to plasticity of the entire soil sample.

Typical results of cyclic stress ratio versus number of cycles of loading for tests on undisturbed samples are shown in Fig. 7. This figure shows four different plots corresponding to different failure / deformation conditions. Plot R is for $u = \overline{\sigma}_3$ condition which is the commonly used criteria to define initial liquefaction for sands. Plots P, Q and S (Fig. **7**) depict the relationship between the cyclic stress ratio and number of cycles for inducing 5, 10 and 20 % D.A. (double amplitude) axial strains in the sample. It is seen that the failure defined by 5 or 10 % strains occurs even before (at a smaller cyclic stress ratio) the condition of initial liquefaction defined by $u = \overline{\sigma}_3$ develops for a given number of cycles.

A typical data for the tests on reconstituted samples is shown in Fig. 8. It is seen form this figure that for the case of reconstituted samples also the failure defined by 5 % or 10 % double amplitude axial strains occurs before the condition of initial liquefaction defined by $u = \overline{\sigma}_3$ occurs.

A comparison of the cyclic strength of undisturbed and reconstituted samples of given in Fig.9. All the results in figures 7 through 9 are on sample with plasticity index of 10% It may be observed that both undisturbed and reconstituted samples of this clayey silt, have a tendency to develop the condition of initial liquefaction and large deformations under cyclic loading. The cyclic strength behavior of this soil is markedly different from that of loose sands. In case of loose saturated sands, the initial liquefaction defined by u $=\bar{\sigma}_3$ condition generally occurs before the condition of 5 or 10 % double amplitude strain (Seed, 1967; Seed and Lee, 1966; Silver, 1977).

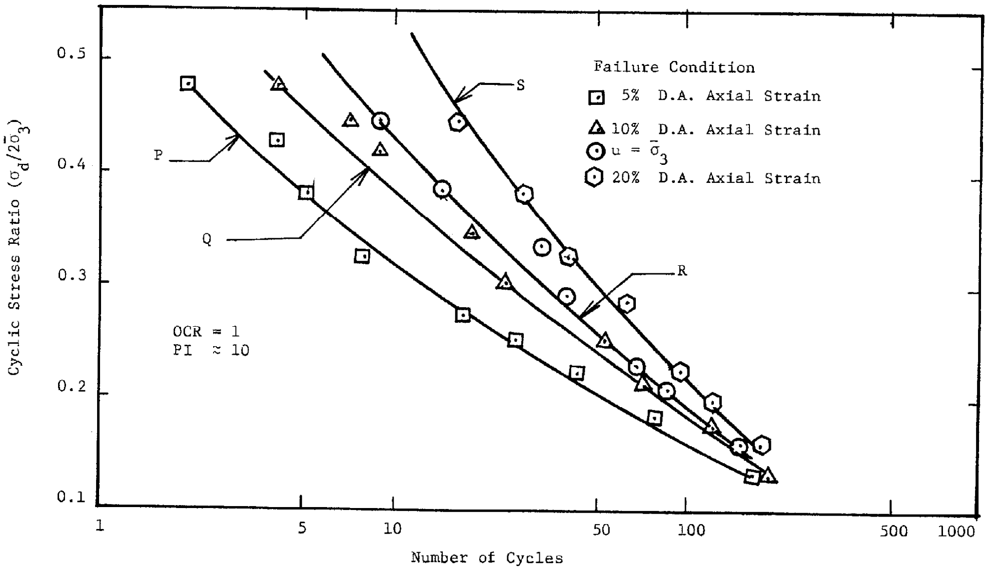

Figure 7. Cyclic Stress ratio Versus number of Cycles for Undisturbed Saturated Samples for σ_3 = 10.0 psi (Puri, 1984)

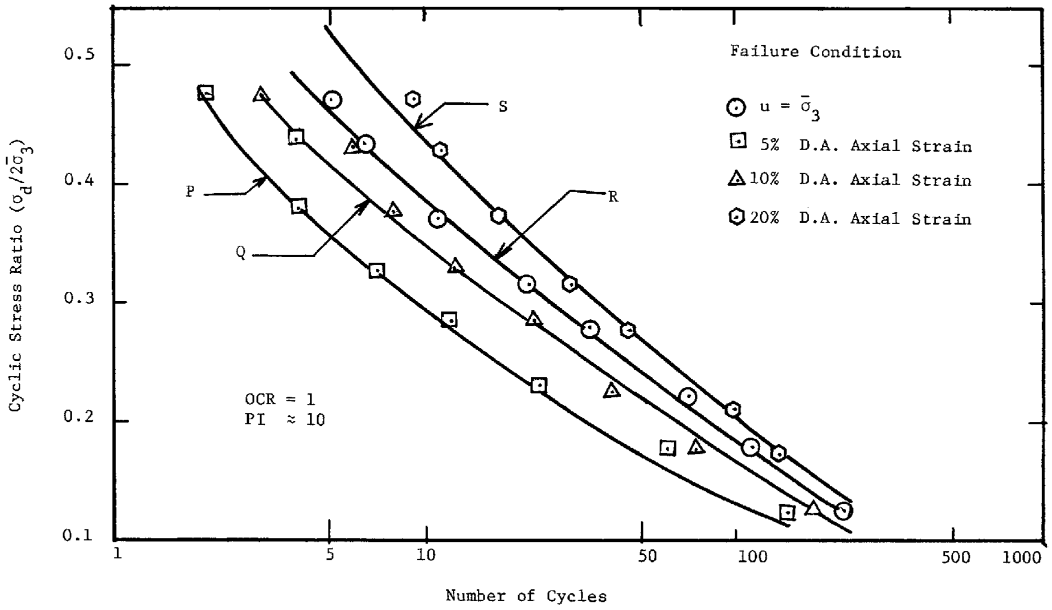

Figure 8.Cyclic Stress ratio Versus number of Cycles For Reconstituted Saturated Samples For σ_3 = 10.0 psi (Puri, 1984)

In case of the soil used in the present case the condition of 5 or 10 % double amplitude axial strain developed before $u = \bar{\sigma}_3$ occurred. Similar behavior was observed by Singh (1994) during tests on undistributed samples of silt.

As mentioned earlier the effect of variation in plasticity index from 10-20% on the cyclic strength was investigated by tests on reconstituted samples. Figure 10 shows the effect of plasticity index on the cyclic stress ratio versus number of cycles plot for 5% double amplitude strain condition for PI =10%,15% and 20%. It is noted from this figure that cyclic stress ratio causing the 5% double amplitude condition increases with an increase in the plasticity index.

Figure 11 shows the effect of PI values on cyclic stress ratio causing 5% double amplitude axial strain in 10 and 30 cycles. It was also observed that for PI values of 15% and more the $u = \bar{\sigma}_3$ condition did not develop within the range of number of loading cycles applied in this investigation whereas it did develop for the case of PI =10%. This behavior suggests that the development of pore water pressure in silt-clay mixtures depends on the plasticity index and the silt-clay mixtures (with plasticity index below 15) need further investigation. The slope of plots in Fig. 11 also indicates that if theses plots are extended in the low plasticity range (PI ‹ 10), the cyclic stress ratio causing 5% double amplitude strain will be much smaller. The silts and silt -clay mixture with PI ‹ 10 may be prone to liquefaction unless proved otherwise

Sandoval (1989) and Prakash and Sandoval (1992) investigated the effect plasticity index on the liquefaction potential of silty soils of low plasticity. Low plasticity was achieved by adding kaolinite clay to silt. The index properties of the soil tested by them are as follows:

Specific gravity of soil solids	2.725
Particle size data	
D_{50} mm	0.022
D_{10} mm	0.013
Uniformity coefficient	3.5
Percent finer than # 200 (wet sieving)	96-98
Percent finer than # 200 (dry sieving)	83-87
Liquid limit (distilled water)	24.2-26.6
Plastic limit (distilled water)	21.0 – 25.2
Liquid limit (tap water)	24.0-26.0
Plastic limit (tap water)	22.5 – 23.0
Plasticity index	1.7± 0.1
Proctor compaction test	
Optimum water content	16.5- 17.5 %
Maximum dry unit weight	106.0-107.2 pcf

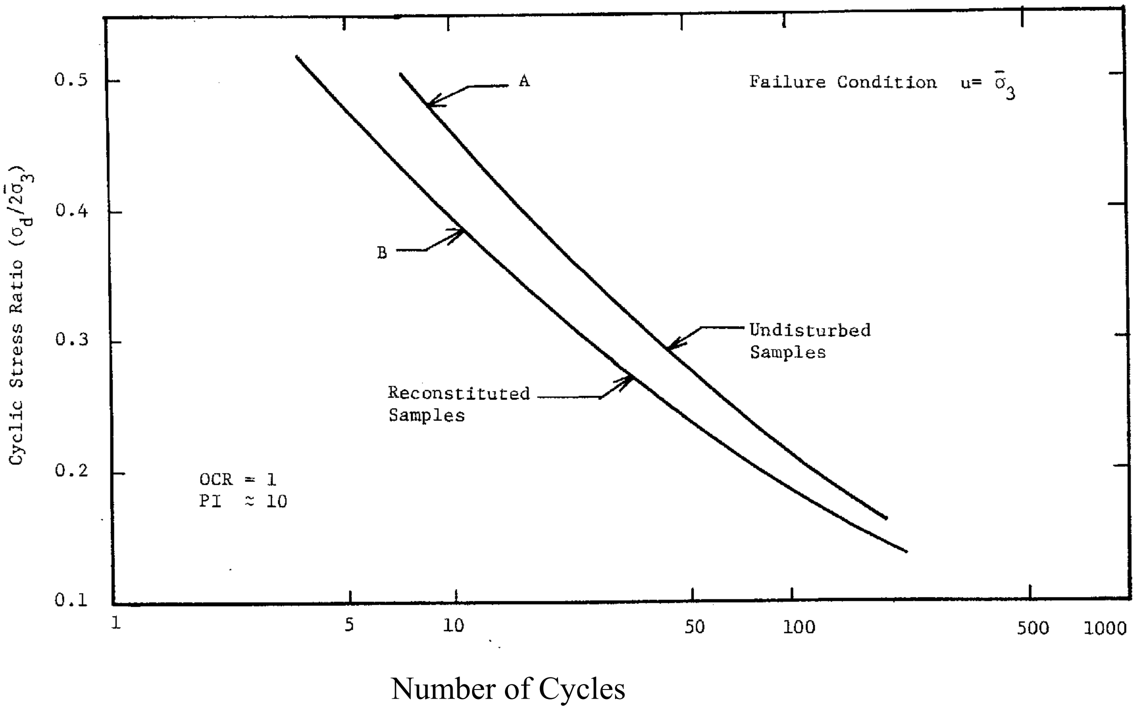

Figure 9 Comparison of Cyclic Stress Ratios for Undisturbed and Reconstituted Saturated Samples For Inducing u = σ_3 Condition in a Given Number of Cycles (Puri, 1984)

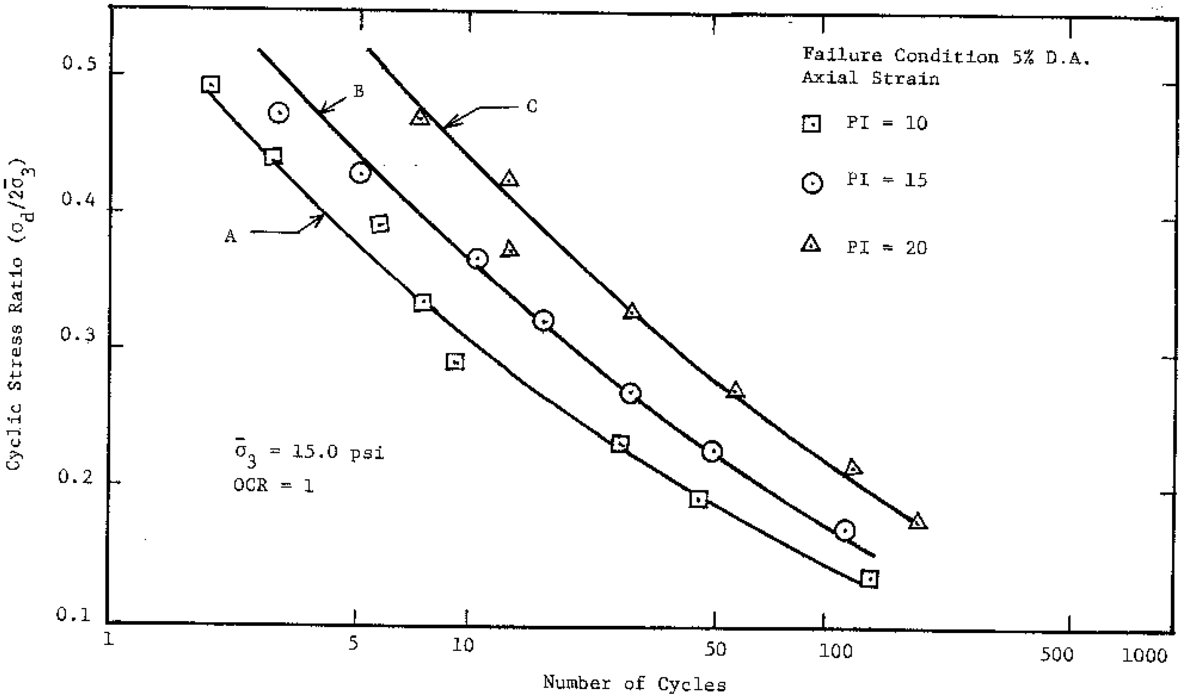

Figure 10. Cyclic Stress Ratio Versus Number of Cycles for Reconstituted Saturated Samples for Different PI Values, Inducing 5 % D.A Axial Strain (Puri, 1984)

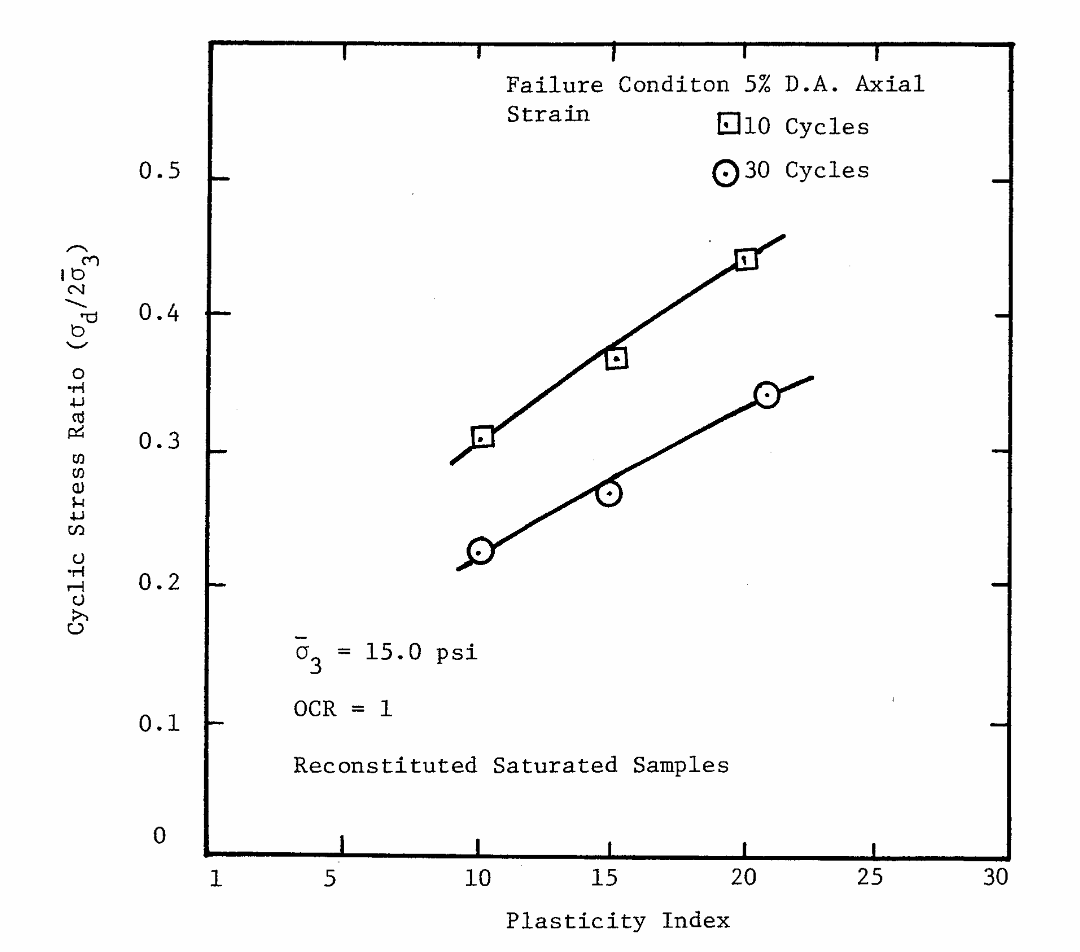

Figure 11 Effect of Plasticity Index on Cyclic Stress Ratio Inducing Failure Number of Cycles (Puri, 1984)

Typical results of their study on reconstituted samples are shown in figures 12 and 13. Figure 12 shows the cyclic stress ratio versus number of cycles causing initial liquefaction (u $=\bar{\sigma}_3$) and Fig. 13 shows the same information for 20 % double amplitude axial strain. Figure 14 shows the effect of plasticity index (PI = 1.7,2.6 and 3.4) on the cyclic stress ratio causing initial liquefaction in any given number of cycles. It is clear from this figure that the cyclic stress causing liquefaction in a given number of cycles decreases with the increase in plasticity index. Prakash and Sandoval (1992) further observed that cyclic loading of plastic silts results in pore pressure build up which becomes equal to the initial effective confining pressure resulting in development of the initial liquefaction. An increase in the clay fraction up to 10%, decreased the cyclic stress ratio to cause a given failure condition Sandoval (1989) has also shown that the axial deformation of low plasticity silt increases with an increase in the clay content. For 5% clay content, the condition of initial liquefaction was attained after 5% double amplitude strain but before 10 % double amplitude strain condition. For 10% clay content, the condition of initial liquefaction was attained after 10% double amplitude axial strain condition but before 20 % double amplitude strain condition.

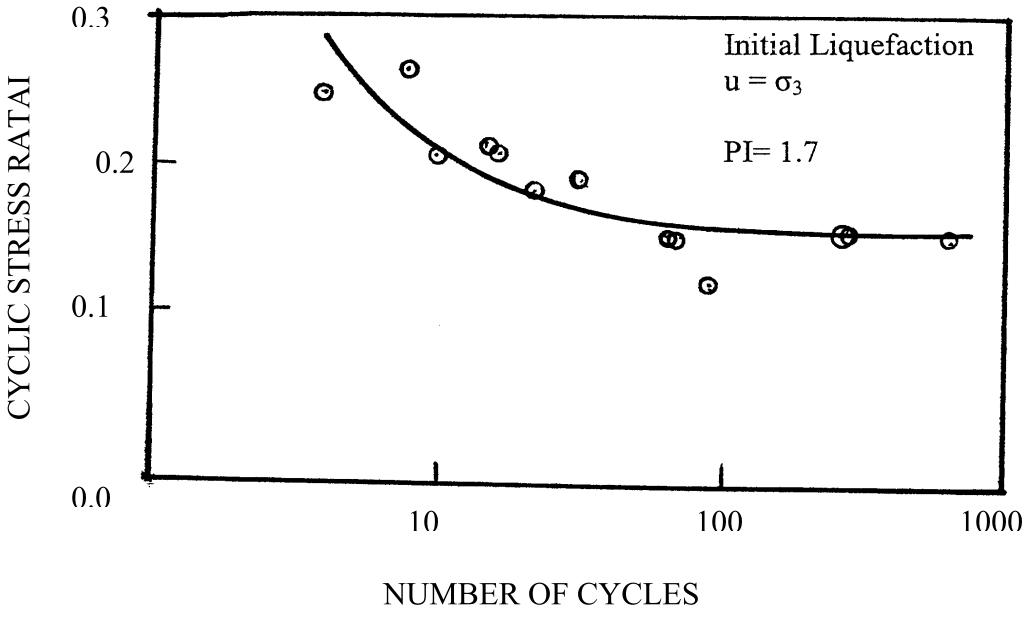

Figure 12 Cyclic Stress Ratio versus Number of Cycles for Low Plasticity Silts, for Inducing Initial Liquefaction Condition at 15 psi Effective Confining Pressure , for Density =95.4 – 98.5 pcf and w = 8 % (Sandoval, 1989; Prakash and Sandoval, 1992)

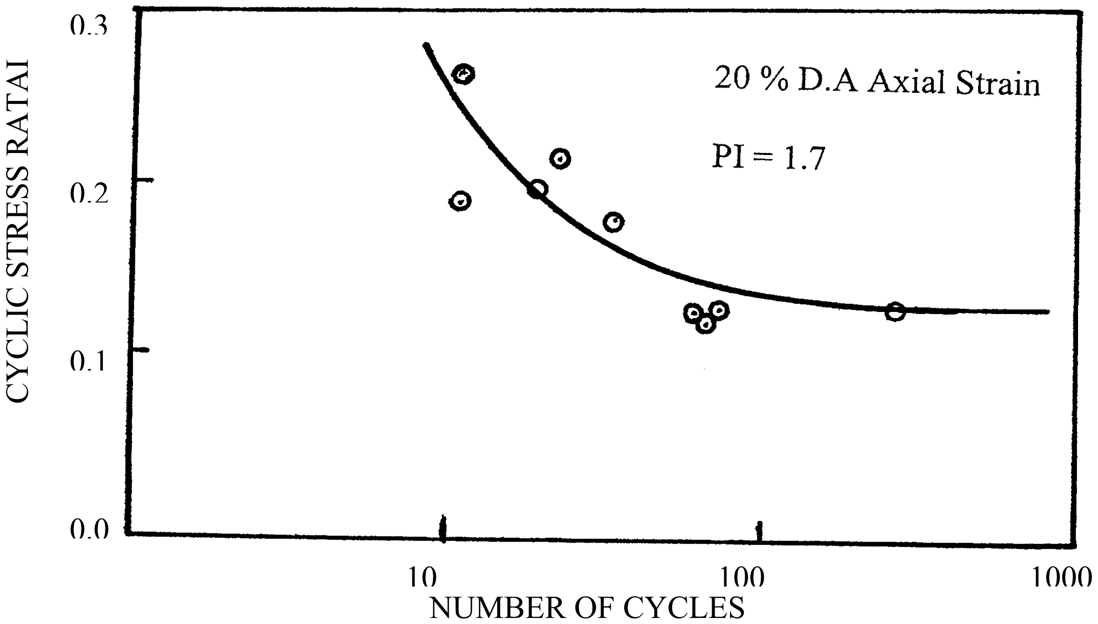

Figure 13 Cyclic Stress Ratio versus Number of Cycles for Low Plasticity Silts, for Inducing 20 % DA Axial Strain Condition at 15 psi Effective Confining Pressure, for Density= 95.4 – 98.5 pcf and w = 8 % (Sandoval, 1989; Prakash and Sandoval, 1992)

Comparing the results of these studies of Puri (1984) and Sandoval (1989) brings out an interesting fact that the cyclic strength behavior of silts and silt clay mixtures depends on the clay fraction and the plasticity index and the behavior shows change as the plasticity index changes in the low plasticity range. From liquefaction point of view, the cyclic strength of silts and silt clay mixtures with PI‹ 10 or may be 15 ,is rather important. Guo and Prakash (1999) further examined this aspect . They combined the data obtained by the authors (Puri,1984,1990; and Prakash and Puri, 1982) and Sandoval (1989) on the cyclic stress ratio versus number of cycles causing initial liquefaction in a given number of cycles for different values of plasticity index. The results are shown in figure 15. It is observed from Fig. 15 that for PI values in the range of 1.7 to 4, the cyclic stress ratio causing liquefaction in any given number of cycles decreases with an increase in PI values. For PI values beyond about 4, the cyclic stress ratio causing initial liquefaction in any given number of cycles increases with an increase in the PI values. Based on these results , it may be inferred that there is a threshold or critical value of PI. In this case, this critical value of PI is about 4, at which saturated samples of silt –clay mixtures have a minimum resistance to cyclic loading or highest susceptibility to liquefaction. For PI value below the critical value , liquefaction susceptibility increases with an increase in the PI value. If PI value of soil is above the critical value , the liquefaction susceptibility decreases with an increase in the PI value.

The authors will , however, will like to point that the critical value of PI = 4, mentioned above is based on the results of limited studies. Puri (1984,1990) observed initial liquefaction for samples of clayey silt with PI≈10 for both undisturbed and reconstituted samples. However , initial liquefaction was not observed for reconstituted samples with PI of 15 and 20. Sandoval(1989) only tested samples with PI = 1.7, 2.5 and 3.4 . The index properties of the soils tested by Puri (1984) and Sandoval(1989) also had some differences. The results presented in Fig. 15 may therefore be considered only qualitative in nature . Nevertheless, the results do indicate that the plasticity of the soil plays a very important role in influencing the liquefaction behavior of clayey silt. The critical values of PI, and factors affecting this value need further investigation.

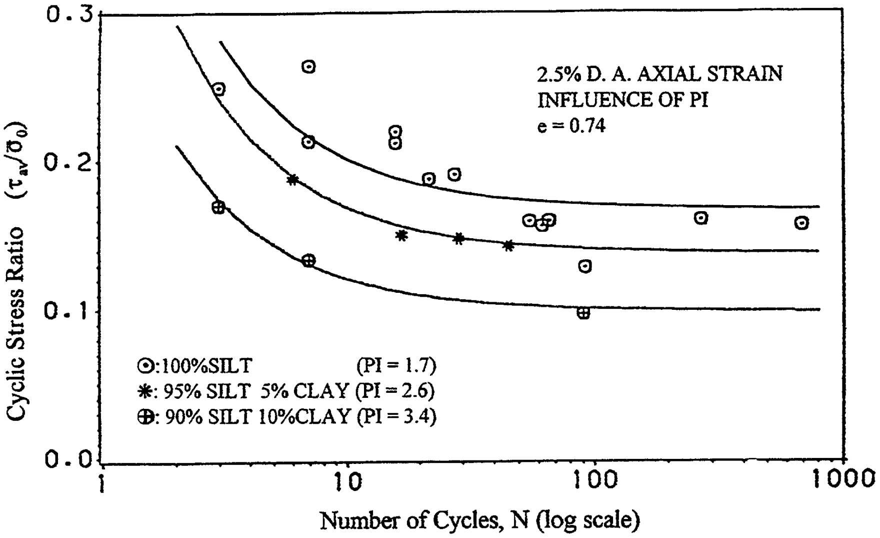

Figure 14 Cyclic Stress Ratio versus Number of Cycles for Low Plasticity Silts for Inducing Initial Liquefaction Condition at 15 psi Effective Confining Pressure; PI = 1.7, 2.6, and 3.4, for Density 97.2-99.8 pcf, and w = 8% (Sandoval 1989; Prakash and Sandoval 1992)

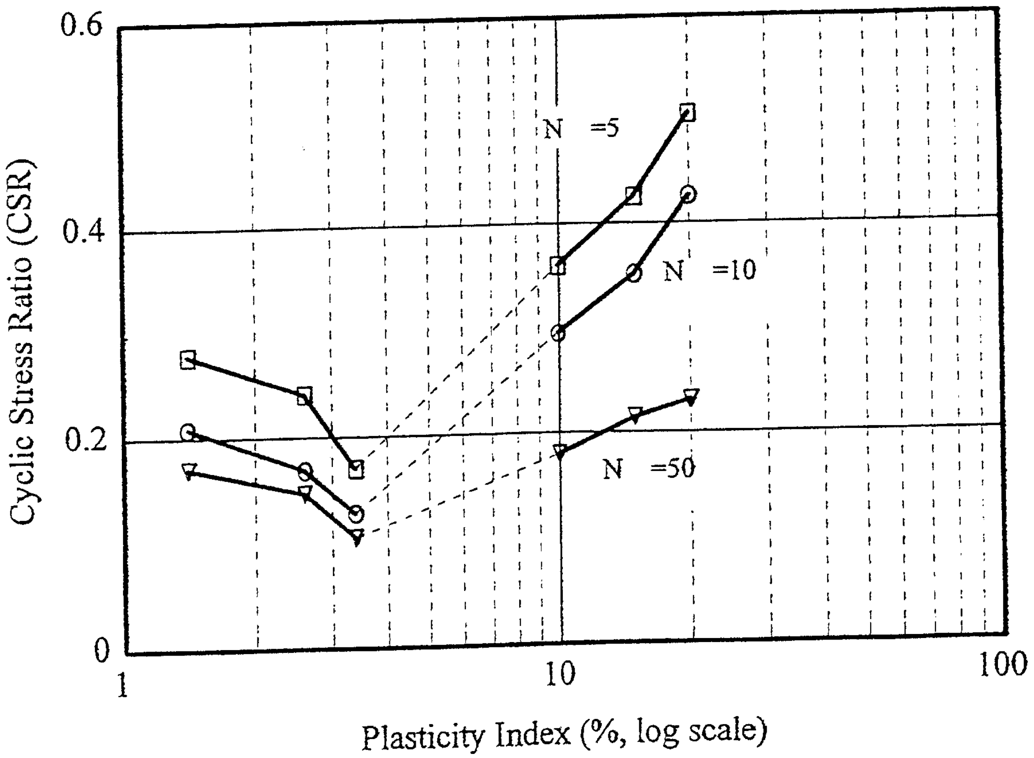

Figure 15 Cyclic Stress Ratio versus Plasticity Index for Silt-Clay Mixtures (CSR Normalized to initial Void Ration e_0 = 0.74) (Prakash and Guo, 1999)

CONCLUSIONS AND FUTHER WORK

It may be concluded that :

(1) The silts and silt –clay mixtures behave differently from sands, both with respect to development and build up of pore water pressures, and deformations under cyclic loading. This has been observed by the authors and several other investigators during the laboratory studies on undisturbed as well as reconstituted samples of silts with low to medium plasticity .

(2) There are several gaps in the existing literature and no guidelines are available and there is no definite criterion to ascertain the liquefaction susceptibility of silts and silt-clay mixtures from simple index properties or simple field tests.

(3) There is also some confusion about the influence of clay content, plasticity index, and void ratio. These factors need further study.

(4) The effects of soil fabric, aging, and other factors are not quite clear. It appears that the soil fabric and aging may slow down the pore pressure generation. Clearly the importance of soil fabric in fine-grained soils such as

silts needs to be recognized when evaluating the pore-pressure generation and strain developments.

The team of investigators at UMR (Luna, 2003) will be conducting a research investigation to provide answers to some of the important questions on the liquefaction behavior of silts and silt-clay mixtures. The aim of this investigation is to establish relationship between in-situ and laboratory resistance to liquefaction of silts consolidated under K_o condition from a slurry.

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