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UNDRAINED RESPONSE AND LIQUEFACTION BEHAVIOUR OF NON-PLASTIC SILTY SANDS UNDER CYCLIC LOADING

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ABSTRACT

The undrained response and liquefaction behaviour of fine sand and silty sands consisting of 30% and 50% non-plastic fines were investigated in the present study. The effect of consolidation stress level, cyclic stress amplitude and amount of non-plastic fines on soil liquefaction have been studied through a systematically planned stress controlled cyclic triaxial tests. To examine the effect of consolidation pressure on liquefaction behaviour, the sand-silt specimens were prepared at 40% relative density and consolidated at pressures of 50, 100 and 200 kPa. To investigate the effect of fines content on liquefaction resistance of sands at constant post-consolidation void ratio, the sand-silt specimens were prepared at different relative densities of 20, 40 and 70%. The cyclic loading with CSR in the range of 0.075 to 0.275 was applied at a frequency of 1Hz in different tests. Test results show the influence of the addition of non-plastic fines to the sand on the reduction of liquefaction resistance. Soil specimens had exhibited higher resistance to liquefaction at low consolidation pressures and lower resistance at high consolidation pressures.

INTRODUCTION

The soil liquefaction is one of the major earthquake hazards and an important research topic in earthquake geotechnical engineering. The main mechanism associated with soil liquefaction is the generation of excess pore pressure in saturated soil mass when it is subjected to cyclic loading or ground shaking. In recent years the liquefaction resistance of silty soils has become an increasingly serious problem during large earthquakes. Earlier, it was considered that poorly graded sands were most susceptible to liquefaction and the fines present in the silty soils would prevent liquefaction. However, the occurrence of tremendous damages to soil structures due to liquefaction was observed in silty soils during past earthquakes (1988 Saguenay earthquake in Quebec; 1999 Kocaeli and Chi-Chi earthquakes; 2001 Bhuj earthquake). The soils in seismic regions of India contain significant amounts of silt fraction. These soil deposits have greater chance to undergo liquefaction during earthquakes. But very limited studies were carried out on the liquefaction resistance of silty sands of these regions (Sitharam et al. 2004).

In the recent past, lot of research work was undertaken by different researchers to study the effect of stress level and amount of non-plastic fines on undrained response and liquefaction behaviour of sand-silt mixtures. However, contradictory results are reported in the literature as follows.

Based on the cyclic liquefaction tests, several studies showed that the liquefaction resistance decreases with increase in the consolidation pressure (Seed and Harder 1990, Amini and Qi 2000). Some other studies are reported that the resistance to liquefaction increases with increase in consolidation pressure (Naeini and Bazir 2004, Tao et al. 2004). Wijewickreme et al. (2005) found that the cyclic strength of copper-gold-zinc tailings is insensitive to consolidation pressure. Based on the static liquefaction tests, the investigations showed that sands become more contractive with increase in consolidation pressure (Lade and Yamamuro 1997, Hyodo et al. 1998). Some other studies found that the silty sands become dilative at high consolidation pressures (Yamamuro and Lade 1998, Yamamuro and Covert 2001).

Earlier research works on liquefaction resistance of sand-silt mixtures have been concluded that the presence of fines decreases the liquefaction resistance (Chien et al. 2002 and Carrao et al. 2003). Some other studies have revealed that the presence of fines decreases the liquefaction resistance up to certain percentage of fines and then increases with fines content (Polito & Martin 2001; Xenaki and Athanasopoulos 2003). It is also reported that there is no effect of the presence of fines on liquefaction resistance of silty sand (Finn et al.

1994), while Amini and Qi (2000) reported increase of liquefaction resistance by addition of fines. However, several studies have shown that the liquefaction resistance of silty sand is more closely related to its sand skeleton void ratio than to its silt content (Thevanayagam et al. 2002, Tao et al. 2004). Based on static liquefaction tests, contradictory observations were also reported on silty sands in the literature. Some research findings showed that the silty sands are less dilative and more susceptible to liquefaction than sands (Yamamuro and Kovert 2001, Thevanayagam 2002). However, other findings are contradictory to the above (Kuerbis et al. 1988, Pitman et al. 1994).

Therefore it is essential to re-investigate the liquefaction phenomenon in silty soils for preventing the disasters and to confirm the specific patterns of liquefaction response.

EXPERIMENTAL PROGRAM

A series of laboratory cyclic triaxial tests was performed on prepared specimens of sand and silty sands using an advanced two-way cyclic triaxial testing device. A brief description on soils tested, sample preparation, test equipment and procedure is given in the following sections.

Soils tested

For the present investigation, natural, non-plastic silty sand was collected at a depth of 2 m from the power plant site at Dhuvaran in Gujarat, India. The soil consisting of 14-18% non-plastic silt which is almost similar to the soils presents in the Bhuj regions.

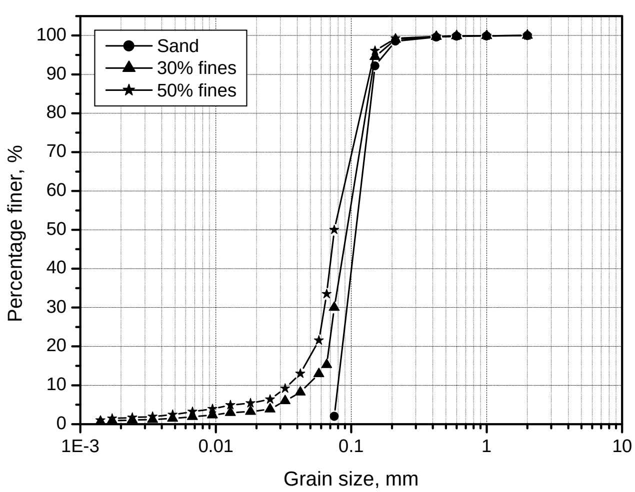


Fig. 1 Grain size distribution of soils

Clean sand (>75 microns) and fines fraction (<75 microns) were separated from the natural silty sand by wet and dry sieving methods. Silty sands were created by adding 30% and 50% fines to the clean sand. In order to produce a particle size distribution curves for the samples, sieve analysis and hydrometer tests were performed according to ASTM D422-63. The grain size distribution curves of soils are plotted based on the sieve analysis and hydrometer test results and are presented in Fig. 1. It is observed from Fig. 1 that, clean sand consists of almost 92% of soil particles are less than 0.15 mm size indicating very fine sand. Maximum and minimum void ratios for sand and sand-silt mixtures were determined as per ASTM procedures (D 4254-91 and D 4253-93). For the determination of the maximum void ratio, e_{max}, a thin-wall cylindrical tube (method B) was used to pour the dry soil material into a standard 153.5 mm inside diameter compaction mould with a volume of 3331 cm^3. The loosest state was achieved by lifting the tube while releasing the soil material into the mould. The minimum void ratio, e_{min} (maximum density) was achieved by densifying the dry soil material in a compaction mould by subjecting it to a surcharge of 14 kPa, and then it is vibrated using electromagnetic shaker at a frequency of 60 Hz for 15 minutes.

Even though these methods were recommended for soils with silt content less than 15%, but in this case they can still indicate the probable range of densities that could be obtained in soils with silt content above 15%. In order to get a consistent value, standard proctor test method was also used in this study to determine the minimum void ratio. Each of the above tests was run three times and average values of void ratios were used in this study. The index properties of the sand and silty soils are shown in Table. 1. According to Unified soil classification system, the soils are classified as poorly graded uniform soils. The shape of the soil particles were mostly rounded to sub-rounded and the soils are in brownish colour. The silt fraction of soil was found to be non-plastic because it has no discernible liquid or plastic limit.

Table 1. Index properties of soils

Index properties	Soils tested		
	Sand	30% fines	50% fines
G_s	2.69	2.71	2.71
D_{50} (mm)	0.112	0.09	0.075
C_u	1.543	2.127	2.428
C_c	0.891	1.197	1.420
e_{max}	0.975	1.015	1.014
e_{min}	0.675	0.620	0.569
Soil classification	SP	SM	ML

Sample preparation

Triaxial soil specimens were prepared by the moist tamping method using the undercompaction procedure reported by Ladd (1978). This procedure avoids the particle segregation in case of silty sands and enables the preparation of loose soil specimens at high void ratios. This method was selected because the material used in this study is sand mixed with various amounts of fines content. An important aspect of soil specimen preparation is the achievement of homogeneous and uniform distribution of void ratio within the specimen. As such consideration has been given to minimize the non-uniform distribution of void ratio, the moist tamping method was conveniently modified using undercompaction procedure reported by Ladd (1978) as illustrated in Fig. 2.

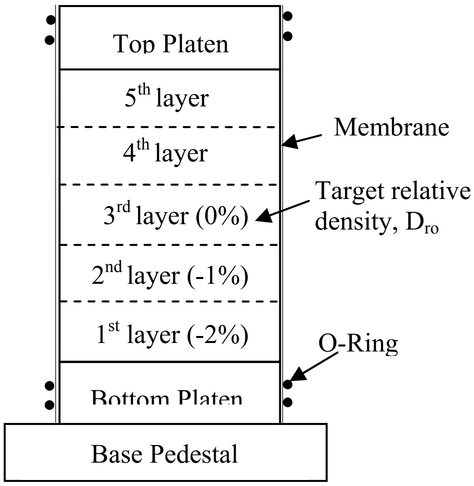


Fig. 2 Schematic view of undercompaction procedure

As per the procedure, air-dried soil sample was divided into five parts, with each part determined by the weight corresponding to desired relative density of soil. In order to attain a uniform density throughout the height of the specimen, the soil was placed at different relative densities in the sand former. Figure 2 shows the target layer densities for a moist-tamped specimen with an undercompaction ratio of 1%. The weight of the soil used to form the different layers corresponds to relative densities of D_{ro}-2, D_{ro}-1, D_{ro}, D_{ro}+1 and D_{ro}+2% from bottom to top of soil specimen. Herein, initial relative density D_{ro} is the target density of the specimen at the time of sample preparation. However, the volume of soil placed in each layer is the same for the equal height of five layers corresponds to 20 mm. The soil to be placed in each layer was mixed with 6% water content and kept in a sealed container for 10 hours to ensure uniform water content distribution throughout the soil. The capillary force due to this moisture enables the silt particles to hold together to obtain the loosest possible condition.

Each of the five parts of the sample was placed one by one in layers into the membrane-lined split former and compacted to the specified density with tamping rod. Tamping rod consist rubber tamping disc of 25 mm diameter. Tamping of soil in each layer was continued until the soil was compacted to 20 mm thickness. The tamping rod was marked at every 20 mm vertical distance to monitor the thickness of compacted layers. Top of each compacted layer was slightly scratched before placing the next layer to promote proper bonding. This procedure is continued until the target specimen was achieved. All the specimens of 50.4 mm diameter and 100 mm height were prepared at required relative densities.

Test equipment

Laboratory tests were performed using a servo-pneumatic cyclic triaxial testing device (manufactured by Wykheham Farrance International Ltd, United Kingdom), which is fully automated and software controlled for both operation and data acquisition. The loading system consists of a 100 kN load frame and ± 5 kN double acting actuator capable of generating loading frequencies up to 70 Hz. A special type of submersible load cell of capacity ±5 kN is fitted inside the triaxial cell to measure the applied axial load. An external displacement transducer of ±25 mm capacity was used for measurement of axial displacements. The cell water pressure, pore water pressure and back water pressures were measured through 1000 kPa capacity diaphgram type electronic pressure transducers. A 100cc volume change transducer was used to monitor the volume changes in the sample. An internal LVDT fixed on the actuator (±15 mm) was used to detect the peak to peak deformations during cyclic loading. The data was collected and stored through data acquisition software.

Testing procedure

Undrained stress controlled two-way cyclic triaxial tests were performed on reconstituted specimens as per ASTM-D5311. In the triaxial test, lubricated ends were used to eliminate the effects of end restraints. This lubrication reduces the friction at the end plates and supports the generation of a homogeneous stress field in the specimen. After the specimen was formed as per the procedure described in earlier section, the specimen top cap was placed and sealed with O-rings. Before removing the former from the sample, vacuum pressure of 8 kPa was applied to hold the specimen. After removal of the mould, sample dimensions were measured carefully and then the triaxial cell was filled with water. An isotropic external pressure of 20 kPa was applied to the specimen and vacuum pressure was removed from the specimen before saturation. Initial saturation was performed by purging the specimen with CO_2 before circulating deaired water. After initial saturation, the back pressure and the cell pressure were increased in increments up to 350 kPa by maintaining the constant effective stress acting on the sample. The soil was saturated until the Skempton's pore pressure parameter 'B' reached a value of 0.98. Then the specimens was consolidated isotropically at particular effective confining pressure (σ_c'). The volume of the sample was constantly monitored during the consolidation process. After the constant volume state was reached, the drainage valves were closed electronically.

Then the specimens were subjected to two way cyclic deviator stress, ($\pm\ \sigma_{dc}$) with a frequency of 1.0 Hz. To simulate the earthquake loading, constant amplitude of sinusoidally varying cyclic deviator stress is applied to the top of the specimen. The cyclic deviator stress simulates the change in shear stress induced by the earthquake. For the same initial relative density and effective confining pressure, soil specimens were subjected to different values of cyclic stress ratios, (CSR = $\pm\sigma_{dc}/2\sigma'_c$) in the range of 0.075 and 0.2 to obtain the liquefaction resistance curve. The cyclic loading with a particular CSR was continued for at least 5-10 cycles after triggering the initial liquefaction to arrive both the initial liquefaction and limited strain failure of 2.5% and 5% DA. Initial liquefaction was identified when the excess pore pressure, Δu is equal to the applied effective confining pressure during cycling loading. Double amplitude (DA) axial strain is defined as the total strain that occurs between any two adjacent peak compressive and tensile strains. During the cyclic loading, continuous records of the excess pore water pressure, (Δu), cyclic axial strain, (ε_c), and the cyclic deviator stress ($\pm\sigma_{dc}$) were recorded automatically using the integrated software.

RESULTS AND DISCUSSION

Stress controlled cyclic triaxial tests were performed on sand and silty soil specimens to study the liquefaction during continuous two-way cyclic shearing under constant volume condition. The influence of silt content and stress level on pore pressure generation and development of axial strain responses during load cycles has been investigated.

Typical undrained cyclic response

Typical response of pore pressure buildup and axial strain development obtained from cyclic triaxial test carried out on clean sand specimen (D_{ro}=40%, σ_c'=100 kPa) subjected to a cyclic stress ratio of 0.2 is shown in Fig. 2. The pore pressure ratio in Fig. 2a is defined as the ratio of pore-pressure amplitude to effective consolidation pressure. It can be noticed from Fig. 2a that the pore pressure buildup is rapid in the first and the last 5 cycles and gradually increases during the intermediate load cycles. The specimen has undergone initial liquefaction at about 31 cycles. The initial liquefaction is identified when the excess pore pressure, Δu, reaches the consolidation pressure level, σ_c', i.e. $\Delta u = \sigma_c'$.

Figure 2a also indicates insignificant development of axial strain (DA= 0.1 to 0.5% only) occurs in the soil specimen up to three cycles before the occurrence of initial liquefaction. At the state of initial liquefaction, the increase in axial strain is propagated to 2.0% DA. However, the soil specimen experiences large axial strain (DA= about 6 %) within a two cycles after the initial liquefaction. Sudden increase of such deformation is due to loss of strength of the soil at initial liquefaction. Ueng et al. 2004 also showed the similar pattern of development in axial strain with load cycles.

Stress-strain hysteresis as shown in Fig. 2b demonstrates that the loops of cycles are close to each other and formed in thicker band with insignificant deformation before the onset of initial liquefaction. At and above the state of initial liquefaction, large strains were developed due to loss in applied deviator stress. The loops of cycles are widened with increase in rate of deformation. Followed the thicker band above the state of initial liquefaction, deformation is rapidly increases to 6% DA within two loops of cycles. The sample had experienced both compression and extensional strains and failed in compression side at 3.5 % axial strain. Effective stress path as shown in Fig. 2c indicates that the mean normal effective stress, $p' = (\sigma_1' + 2\sigma_c')/3$ decreases as the pore pressure builds up. Similar types of undrained responses were also reported by previous researchers (e.g. Polito and Martin 2001, Ueng et al. 2004). The stress loops are widened during the start and end of the test because of rapid increase in pore water pressure in the first and the last five cycles of loading.

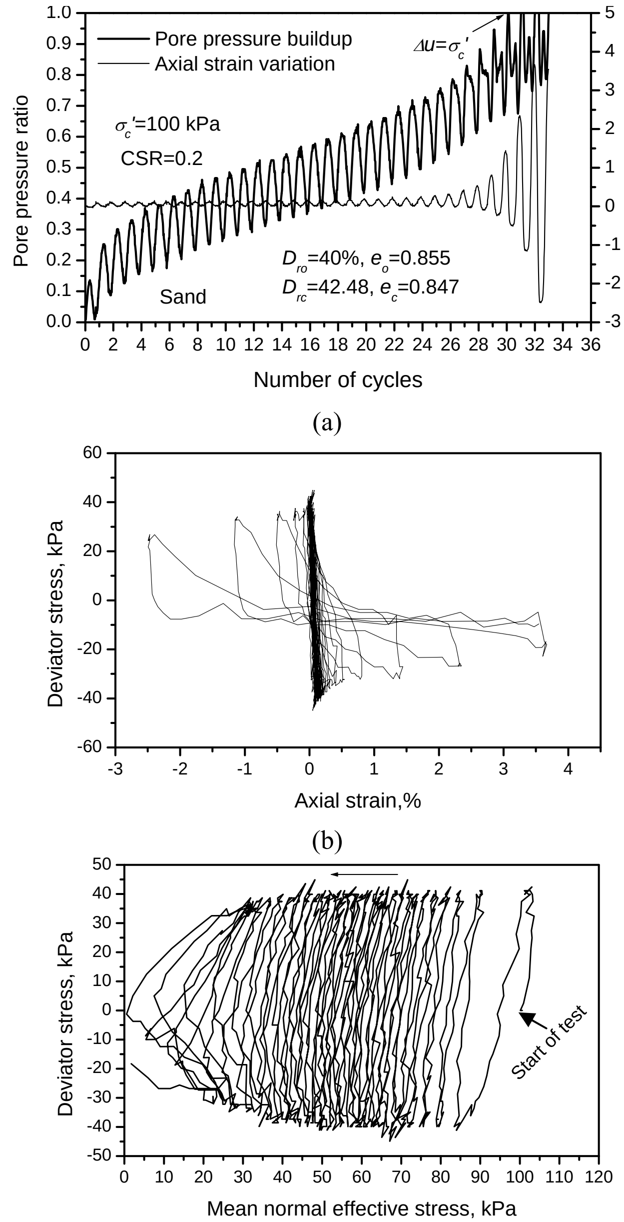
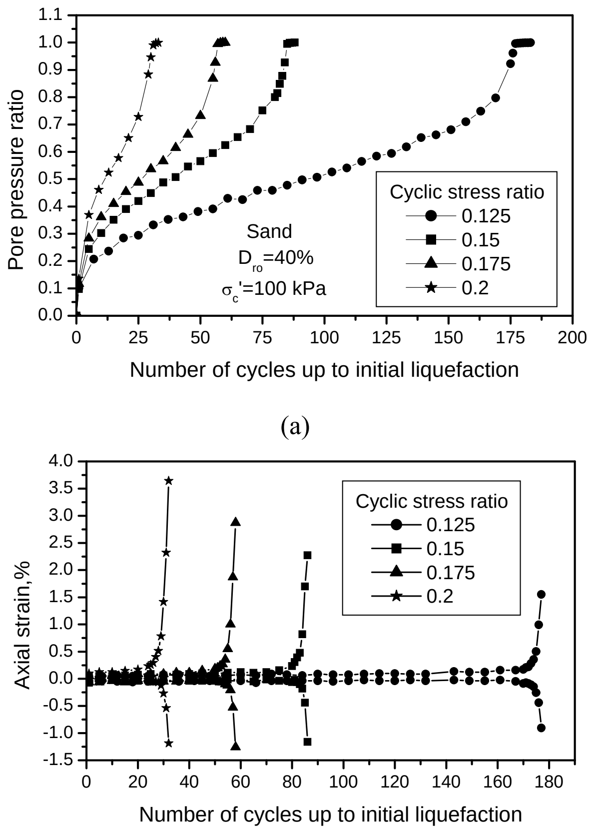


Fig. 2. Undrained response of clean sand: (a) Pore pressure and axial strain responses, (b) Stress-strain hysteresis and (c) Effective stress path.

Effect of cyclic stress ratio

The effect of magnitude of applied cyclic loading i.e. cyclic stress ratio (CSR) on undrained response of clean sand (D_{ro}=40%, σ_c'=100 kPa) is presented in Fig. 3. It can be observed from Fig. 3 that the pore pressure and axial strain is accumulates gradually under cyclic loading with low CSR whereas they develop at faster rate under cyclic loading with high CSR. Fig. 3a shows at high CSR of 0.2 that the initial liquefaction ($\Delta u/\sigma_c'$ = 100%) is triggered at 28 cycles of loading, but at low CSR of 0.125, the development of pore pressure ratio is only about 30% ($\Delta u/\sigma_c' = 30\%$) at the same load cycles. 100% pore pressure ratio is developed for long duration to about 175 cycles when the soil is subjected to low CSR of 0.125. Fig. 3b indicates that the compression strains are accumulated progressively while increasing the magnitude of cyclic loading i.e. increasing CSR from 0.125 to 0.2. Large amplitude of strains occurs at the high CSR values.



(b)

Fig. 3 Effect of Cyclic Stress Ratio (CSR) on (a) pore pressure buildup and (b) axial strain variation with load cycles for clean sand

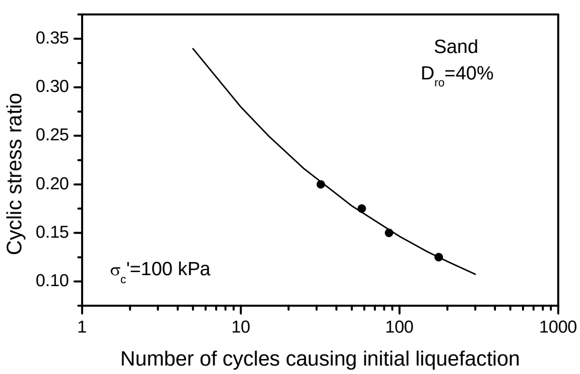


Fig. 4 Typical liquefaction resistance curve for clean sands

The triaxial sand specimens (D_{ro}=40% and σ_c'=100 kPa) were subjected to four different values of cyclic stress ratios to obtain liquefaction resistance curve. Typical liquefaction resistance curve i.e. CSR versus number of cycles causing liquefaction plotted in Fig. 4 indicates the decrease of the cyclic stress level with increase in number of load cycles. The liquefaction resistance is defined in terms of cyclic resistance ratio (CSR_L) required to cause the initial liquefaction at given number of load cycles. In general, the CSR_L at 15 cycles of loading is considered to evaluate the cyclic resistance of soils corresponding to an earthquake magnitude of 7.5 on moment magnitude scale (Seed et al. 1985).

Effect of consolidation pressure level on undrained response

To investigate the effect of consolidation pressure on liquefaction behaviour, the sand-silt specimens were prepared at 40% relative density and consolidated at pressures of 50, 100 and 200 kPa prior to subjecting cyclic axial load.

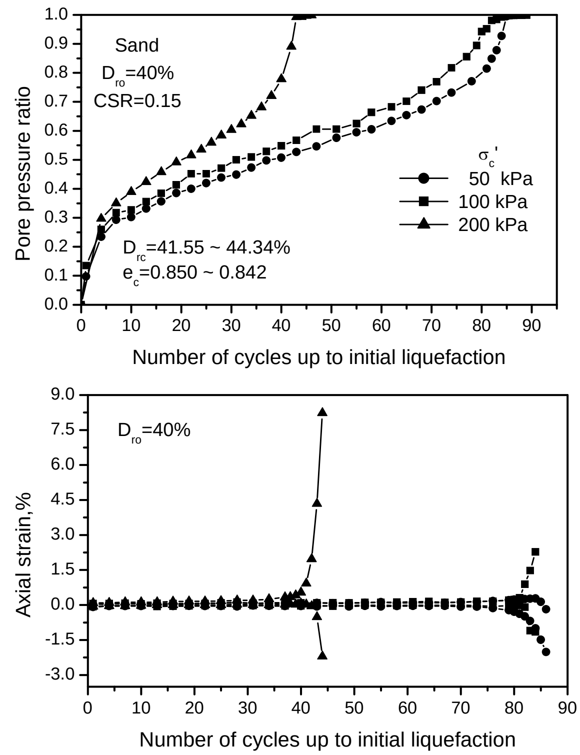


Fig. 5 Effect of consolidation pressure on pore pressure buildup and axial strain variation with load cycles for sand

The response of pore pressure buildup and axial strain accumulations with load cycles for the different consolidation pressures in sand-silt mixtures is depicted in Figs. 5 to7. Figures are indicates that the pore pressure buildup and axial strain accumulation is rapid for specimens consolidated with high pressures. However, the undrained response is gradual for specimens consolidated with low pressures. It is interesting to note from Figs. 5 and 7 that the pore pressure responses are almost similar for sand samples subjected to low consolidation pressures up to 100 kPa, however sand-silt mixtures with 50% fines subjected to high consolidation pressures of 100 and 200 kPa show the similar pore pressure response. It may be due to the role of fines in altering the soil fabric of granular structure under cyclic loading.

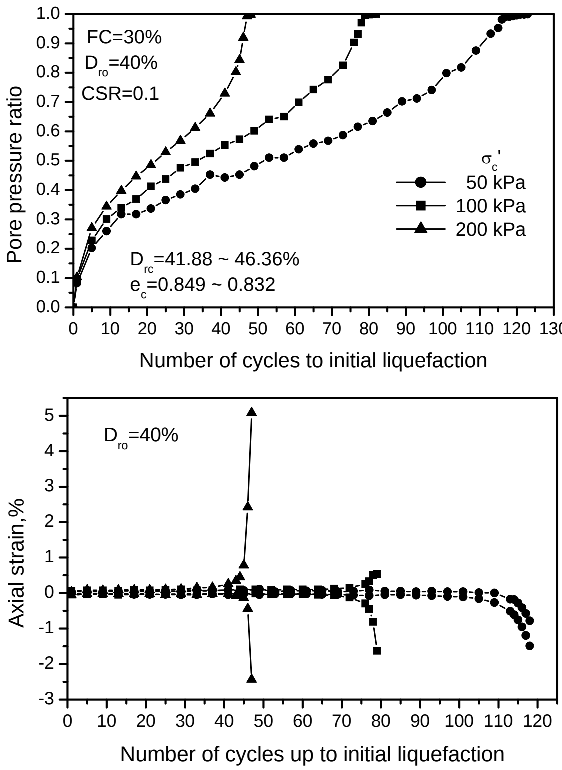


Fig. 6 Effect of consolidation pressure on pore pressure buildup and axial strain variation with load cycles for sand-silt mixture (Fc=30%)

The strain response curves shown in Figs. 5 to 7 indicates that the specimens exhibit both compression and extensional strains for higher consolidation pressures above 50 kPa, however at low confining pressure of 50 kPa the small axial strain occurs only in extension side. Large deformation (DA = 7 to 10%) occurs in sand-silt mixtures when subjected to high consolidation pressures.

<u>Effect of consolidation pressure on liquefaction resistance</u>

The results of effect of consolidation pressure on liquefaction resistance of sand-silt mixtures corresponding to 15 cycles of loading are presented in Fig. 8. It is evident from the figure that the cyclic strength of sand-silt mixtures decreases with increase in consolidation pressures. Similar conclusions were also reported by Amini and Qi (2000) based on the cyclic triaxial tests conducted on Ottawa sand mixed with silt fines. Steep curves of silty samples containing 30% and 50% fines are indicate the more reduction of cyclic strength with increase in consolidation pressure. Sand sample exhibiting higher cyclic strength than the sand-silt mixtures at all pressures and predominant reduction of cyclic strength is observed in between sand and silty soil with 30% fines.

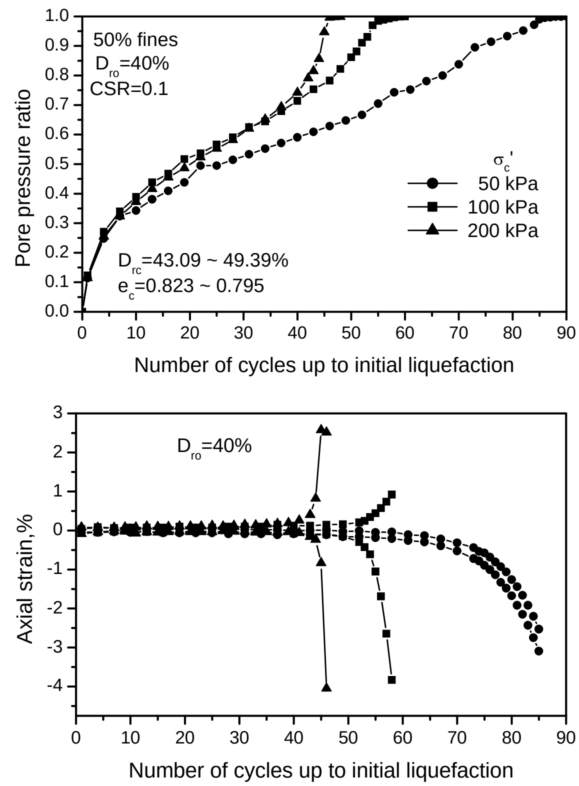


Fig. 7 Effect of consolidation pressure on pore pressure buildup and axial strain variation with load cycles for sand-silt mixture (Fc=50%)

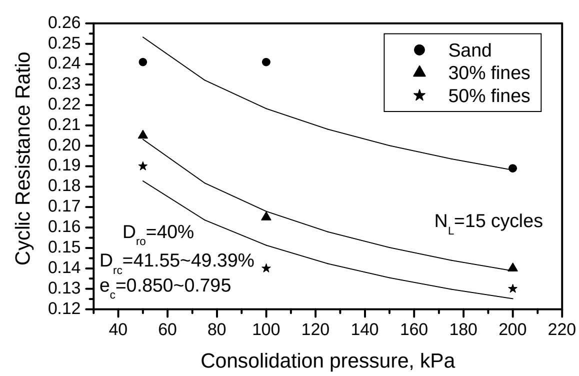


Fig. 8 Effect of consolidation pressure on liquefaction resistance of sand-silt mixtures causing initial liquefaction at 15 cycles of loading

<u>Effect of fines content on undrained response</u>

To investigate the effect of fines content on undrained response and liquefaction resistance of sands at constant post-consolidation void ratio, the sand-silt specimens were prepared at different relative densities of 20, 40 and 70%. The pore pressure buildup and axial strain variation with load cycles for the saturated sand-silt specimens (FC = 0, 30 and

50%) prepared at loose, medium and dense conditions i.e. D_{ro}=20%, 40% and 70% respectively is depicted in Figs. 9 to 11. It can be noticed from the undrained response curves shown in Figs. 9 to 11 that the pore pressure build up and development of axial strain occurs very rapidly for sand-silt mixtures in all the states.

In contrast, gradual development of pore pressure buildup and axial strains occur in clean sand specimens. The pore pressure and axial strain response curves tend to become steeper for sand-silt specimens with increase in fines content, suggesting the increased rate of pore pressure buildup and development of axial strain due to addition of fines to the clean sand. However, the variation in rate of pore pressure buildup is comparatively very less for sand-silt mixtures with 30% and 50% fines than the variation in-between sand and 30% fines.

Soil specimens reconstituted at loose state experience the extensional strains in all cases as shown in Fig. 9 and have negligible compression strains that cause initial liquefaction. On other hand, Fig. 10 shows that the soil specimens at medium dense state experience both compression and extensional strains. Clean sand samples experience high magnitude of compression strains, whereas sand-silt specimens with fines content above 30% experience high magnitude of extensional strains. It may be due to the role of fines in altering the soil fabric of granular structure under cyclic loading.

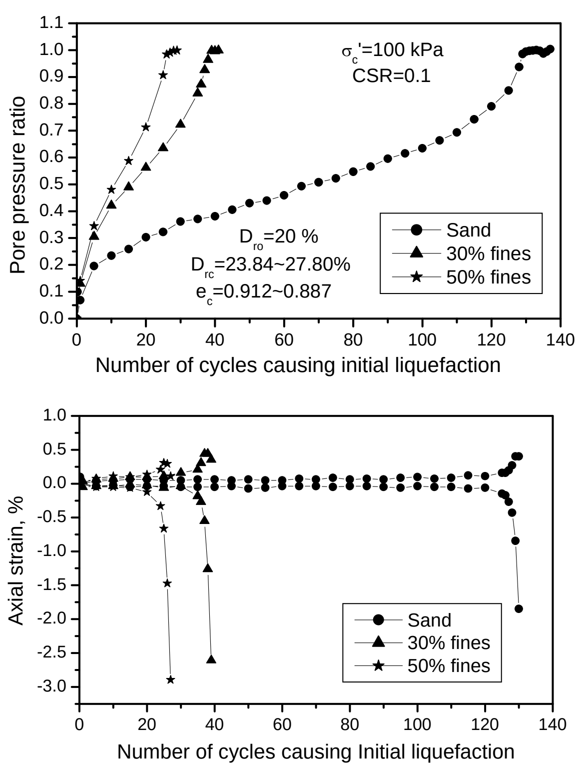


Fig. 9 Effect of fines content on pore pressure buildup and axial strain variation with load cycles at loose state

It can be observed from the pore pressure curves for sand-silt mixtures at dense state shown in Fig. 11 that the pore water pressure buildup is rapid and almost similar for samples with high fines content of 30% and 50%. It indicates that up to a certain value of fines content, the rate of pore pressure increases with increase in fines content for sand-silt mixtures. Samples exhibit pure compression strains with negligible extensional strains. It can be observed from Figs. 9 to 11 that the variation in number of cycles causing initial liquefaction in sand-silt mixtures with fines content above 30% is very less.

<u>Effect of fines content on liquefaction resistance based on initial relative density, D_{ro}</u>

The cyclic strength curves i.e. the relationship between the cyclic stress ratio causing initial liquefaction with the number of cycles for sand-silt mixtures (FC = 0, 30% and 50%) at different dense states are presented in Fig. 12.

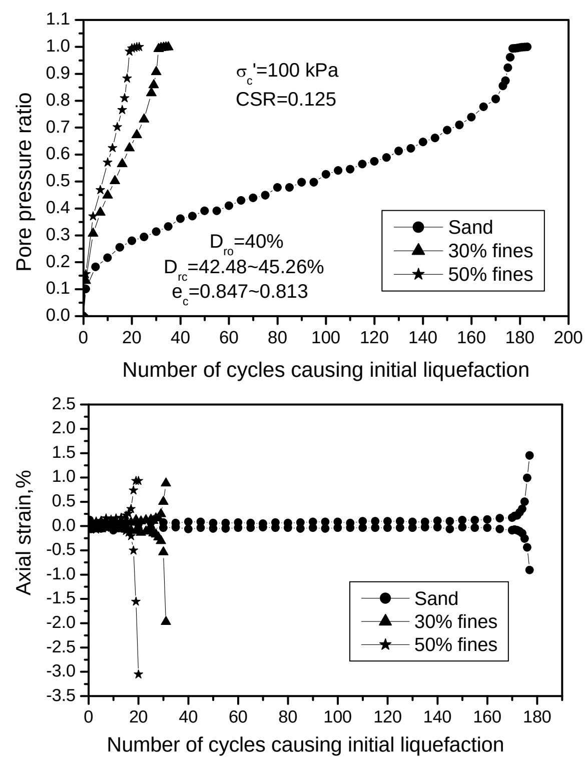


Fig. 10 Effect of fines content on pore pressure buildup and axial strain variation with load cycles at medium dens state

The curves are obtained based on the results of tests carried out on sand-silt specimens prepared at constant relative density, D_{ro}. It is noticed from figure that the number of cycles causing initial liquefaction decreases with increase in cyclic stress ratio for all states. The cyclic strength curves are indicating that the number of cycles required to cause the initial liquefaction of sand at a given CSR decreases with the addition of fines content up to 50%. Liquefaction resistance of clean sand is much higher than the sand-silt mixtures for all states. The reduction in liquefaction resistance is very less in-between sand-silt mixtures with 30% and 50% fines due to buildup of slight variation in rate of pore pressure as discussed in earlier section.

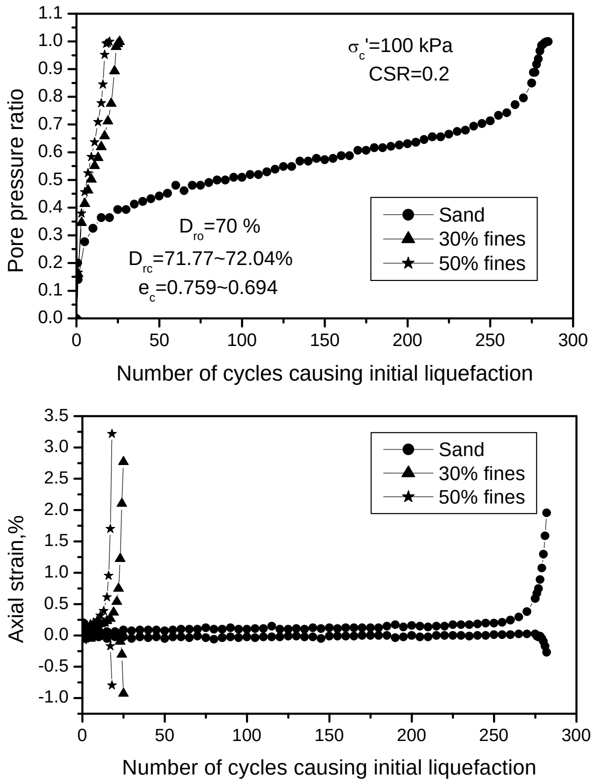
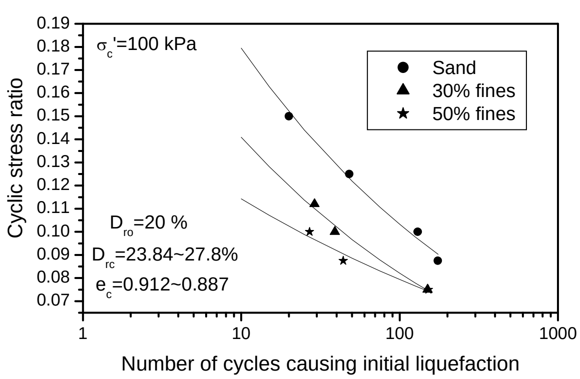
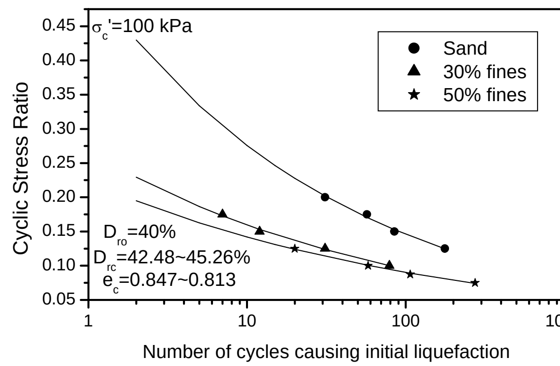


Fig. 11 Effect of fines content on pore pressure buildup and axial strain variation with load cycles at dense state

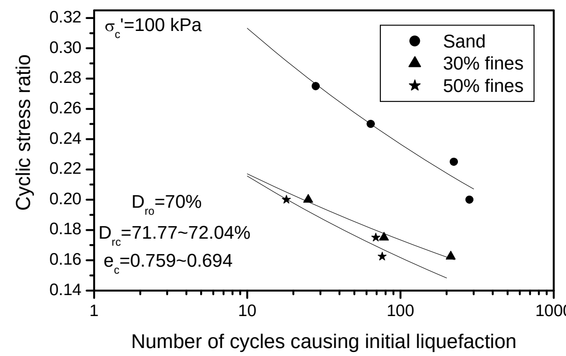
It can be found from the Figs. 12a and 12b that for a soil with loose and medium dense states, the cyclic strength curves converge towards the high number of cycles. But at dense state shown in Fig. 12c, the cyclic strength curves diverge towards the high number of cycles. It indicates the amount of presence of fines in sand affected the liquefaction resistance even at high number of cycles. Fig. 12b also shows that the cyclic strength curves of sand with above 30% fines in moderate dense state are close to each other. It indicates that for further addition of fines above 50%, there is no change in liquefaction resistance of sand.



(a) D_{ro} = 20%



(b) D_{ro} = 40%



(c) D_{ro} = 70%

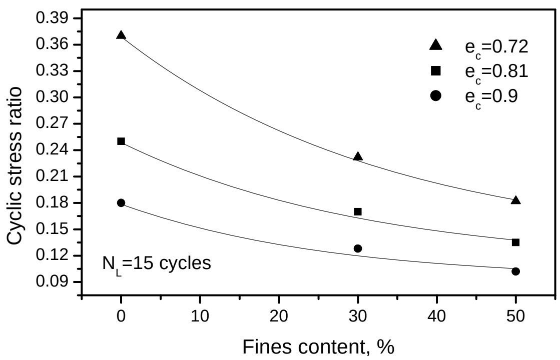
Fig. 12 Cyclic strength curves for sand-silt mixtures causing initial liquefaction in (a) loose,(b) medium dense and (c) dense states

<u>Effect of fines content on liquefaction resistance based on post consolidation relative density / void ratio</u>

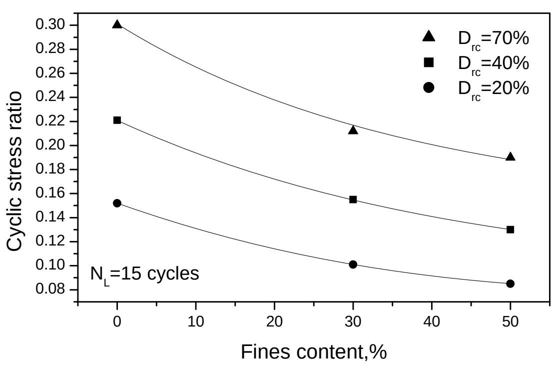
It is evident from the results of cyclic triaxial tests carried on sand and sand-silt specimens that even though a specimen prepared at the same relative density, the post consolidation relative density or void ratio is found to be different and the variation is depend on consolidation pressure and type of soil. Post consolidation void ratio is the best control parameter in determination of liquefaction resistance of silty sands due to the different values of maximum and minimum void ratios for each sample at the sample preparation. The previous research studies have been also reported the results on liquefaction resistance of silty sands based on post-consolidation void ratio (Polito & Martin 2001 and Xenaki and Athnasopoulos 2003).

To determine the effect of fines content on liquefaction resistance of sand at constant post consolidation void ratio, the cyclic stress ratio values corresponding to 15 cycles of loading are marked out from the liquefaction resistance curves as shown in Figs. 12a to 12c. The influence of fines content on cyclic resistance of sand is examined at constant post consolidation void ratio and/or relative density as shown in Fig. 13. It is found that liquefaction resistance of sand decreases with increase in fines content at all states. Similar trend was also observed from the past studies by conducting the cyclic triaxial tests on Nevada & Ottawa sands (Carraro et al. 2003) and Mai Lio sand (Huang et al. 2004).

From Fig. 13a, at dense state of e_c=0.72, the cyclic resistance curve is much steeper and the cyclic resistance of sand decreases rapidly as the silt content increases. However the reduction of cyclic resistance is much lesser for loose and medium dense sands i.e. e_c = 0.9 and 0.81 than the dense sands while increasing the fines content. In contrast based on post consolidation relative density, Fig. 13b show that all the resistance curves of samples are similar and parallel to each other which indicates the reduction in liquefaction resistance of sand is the almost same in all dense states as the silt content increases. It is also interesting to note that the resistance curve is much flatter for the portion above 30% fines indicates less reduction in cyclic strength while adding further fines content to the sand.



(a)



(b)

Fig. 13 Influence of fines content on liquefaction resistance of sand based on constant (a) Post consolidation void ratio and (b) Post consolidation relative density (N_L=15)

Effect of fines content on liquefaction resistance based on sand skeleton void ratio

In addition, the effect of fines content on cyclic resistance of sands is evaluated in terms of sand skeleton void ratio as illustrated in Fig. 14. Sand skeleton void ratio is defined as the void ratio that would exist in a sand-silt mixture if all of the fines particles were removed, leaving only the sand grains and voids to form the soil skeleton. Finn et al. 1994 reported that as long as there is room within the voids produced by the sand to contain all of the silt present, the liquefaction resistance is independent of silt content.

The sand skeleton void ratio, e_{cor} is

$$e_{cor} = \frac{e_c + FC/100}{1 - FC/100} \tag{1}$$

where e_c is the global post-consolidation void ratio of the silty sand; FC is the fines content in terms of a percentage of the total weight of the mixtures.

Fig. 14 shows that the cyclic resistance ratio, CRR curves of sand-silt mixtures corresponding to N_L =15 are parallel to each other. The cyclic resistance of sand decreases linearly with the increase in sand skeleton void ratio as the silt content increases. Sand samples exhibits low range of sand skeleton void ratios (e_{cor}<1) while the sand-silt mixtures with 50% fines content shows high range of skeleton void ratios (e_{cor} >2). The trend depicted in Fig. 14 is in good agreement with observations made by other investigators (Finn et al. 1994; Polito and Martin 2001; Tao et al. 2004).

According to sand skeleton void ratio concept, the presence of non-plastic silt particles in pore voids do not play any role in interlocking of particles in sand matrix. However the increase in pore voids with increase in silt fines leading to rapid buildup of pore pressure. This is the reason why the silt soils are more susceptible to liquefaction than sands.

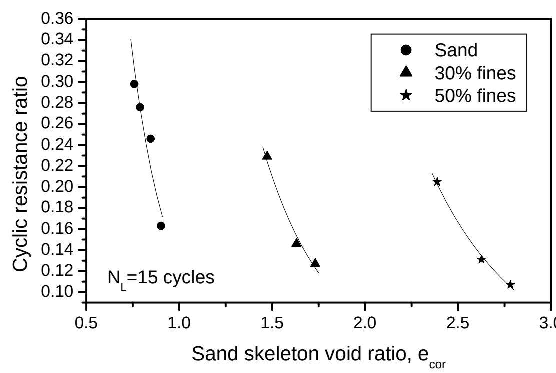


Fig. 14 Variation in cyclic resistance with sand skeleton void ratio for sand-silt mixtures at 15 cycles of loading

CONCLUSIONS

Based on the results of experimental cyclic triaxial tests performed on sand-silt mixtures by varying material and test parameters, the following major conclusions are arrived at.

- The pore pressure build-up in sand and sand-silt mixtures is rapid in the first and last few load cycles and gradually increases during the intermediate load cycles. Samples have experienced very small axial strains before the occurrence of initial liquefaction. Within a few cycles after the initial liquefaction, large strains have developed in the soil samples. Sudden increase of deformation is due to loss of strength of the soil at initial liquefaction.

- The pore pressure build-up is rapid for the samples subjected to high consolidation pressures. Deformation at initial liquefaction is low for the samples consolidated at low pressures and high for highly consolidated samples.
- The liquefaction resistance decreases as the consolidation pressure increases in all the sand-silt specimens.
- The pore pressure build-up is rapid in case of sand-silt mixtures whereas its build up is slow in case of clean sand. The addition of silt in sand causes an increase of axial deformation.
- For a given relative density or void ratio, the cyclic resistance of sand decreases with the increase in non-plastic silt content up to 50%. However, the reduction in liquefaction resistance is very less in-between sand-silt mixtures with 30% and 50% fines due to buildup of slight variation in rate of pore pressure.
- The cyclic resistance of sand decreases linearly with the increase in sand skeleton void ratio as the silt content increases. It is due to the fact that the presence of non-plastic silt particles in pore voids does not play any role in interlocking of particles in sand matrix.

REFERENCES

Amini, F. and G. Z. Qi [2000]. "Liquefaction Testing of Stratified Silty Soils", *J. of Geotech. and Geoenvironmental Engrg., ASCE,* Vol. 126, No. 3, pp. 208-216.

Chien, L.K., Y.N. Oh and C.H. Chang [2002]. "Effects of Fines Content on Liquefaction Strength and Dynamic Settlement of Reclaimed Soil", *Canadian Geotechnical Journal*, Vol. 39, pp. 254-265.

Carraro J.A.H., P. Bandini and R. Salgado [2003]. "Liquefaction Resistance of Clean and Nonplastic Silty Sands Based on Cone Penetration Resistance", *Jl. of Geotech. and Geoenv. Engrg*., Vol. 129, No. 11, pp. 965-976.

Finn, W.L., R.H. Ledbetter and G. Wu [1994]. "Liquefaction in Silty Soils-Design and Analysis-Ground Failures under Seismic Conditions", *Geotech. Sp. Publ.* No. 44, pp. 51-76.

Hyodo, M., A.F.L. Hyde & N. Aramaki [1998]. "Liquefaction of Crushable Soils", *Geotechnique,* Vol. 48, No. 3, pp. 1-17.

Koseki J., K. Ishihara and M. Fujii [1986]. "Triaxial Liquefaction Tests on Sand with Fines", *21st Japanese Annual Meeting on Geotechnical Engineering*, pp. 595-596.

Kuerbis, R., D. Negussey and Y.P. Vaid [1988]. "Effect of Gradation and Fines Content on The Undrained Response of Sand", *Hydraulic fill structures, Geotechnical Special publication 21,* D.J.A. Van Zyl and S.G. Vick (eds.), New York, pp. 330-345.

Ladd, R.S. [1978]. "Preparing Test Specimens using Undercompaction", *Geotech. Test. J.*, Vol. 1, No.1, pp. 16-23.

Lade, P. and J.A. Yamamuro [1997]. "Effects of Non-Plastic Fines on Static Liquefaction of Sands", *Canadian Geotechnical Journal,* Vol. 34, No. 6, pp. 918-928.

Naeini, S.A. and M.H. Baziar [2004]. "Effect of Fines Content on Steady-State Strength of Mixed and Layered Samples of a Sand", *Soil Dyn. And Erthq. Engrg,* Vol. 24, pp. 181-187.

Pitman, T.D., P.K. Robertson and D.C. Sego [1994]. "Influence of Fines on the Collapse of Loose Sands", *Canadian Geotechnical Journal,* Vol. 31, pp. 728-739.

Polito, C.P. and J.R. Martin II [2001]. "Effects of Non-Plastic Fines on the Liquefaction Resistance of Sands", *Jl. Geotech. and Geoenv. Engrg*, Vol. 127, No. 5, pp. 408-415.

Seed H.B., K. Tokimatsu, L.F. Harder and R.M. Chung [1985]. "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations", *Jl. of Geotech. Engrg.,* ASCE, Vol. 111, No. 5, pp. 1425-1445.

Seed, R.B. and L.F. Harder [1990]. "SPT based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength". In: *Proceedings of the Seed Memorial Symposium, Vancouver,* B.C., May 1990, J.M.Duncan (ed.), BiTech Publishers, Richmond, B.C., p. 351-376.

Sitharam, T.G., L. Govinda Raju and B.R. Srinivasa Murthy [2004]. "Cyclic and Monotonic Undrained Shear Response of Silty Sand from Bhuj Region in India", *Journal of Earthquake technology,* Vol. 41, No. 2- 4, pp. 249-260.

Troncoso J.H. and R. Verdugo [1985]. "Silt content and Dynamic Behaviour of Tailing Sands", *11th Proc. of Int. Conference on SM & FE,* pp. 1311-1314.

Thevanayagam, S. and G.R. Martin [2002]. "Liquefaction in Silty Soils-Screening and Remediation Issues", *Soil Dynamics and Earthquake Engineering,* Vol. 22, pp. 1035-1042.

Tao, M., J.L. Figueroa and A.S. Saada [2004]. "Influence of Nonplastic Fines Content on the Liquefaction Resistance of Soils in terms of Unit Energy", *Proc. of the Cyclic behaviour of Soils and Liquefaction Phenomena,* Bochum, Germany, A.A. Balkema, Rotterdam, T.N., pp. 223-231.

Ueng T.S., C.W. Sun and C.W. Chen [2004]. "Definition of Fines and Liquefaction Resistance of Maoluo River Soil", *Soil Dynamics and Earthquake Engineering,* Vol. 24, pp. 745-750.

Wijewickreme, D., M.V. Sanin and G.R. Greenaway [2005]. "Cyclic Shear Response of Fine-Grained Mine Tailings", *Canadian Geotechnical Jl.,* Vol. 42, No. 2, pp. 1408-1421.

Xenaki, V.C. and G.A. Athanasopoulos [2003]. "Liquefaction Resistance of Sand – Silt Mixtures-An experimental Investigation of the Effect of Fines", *Soil Dynamics and Earthquake Engineering*, Vol. 23, pp. 183-194.

Yamamuro, J.A. and P.V. Lade [1998]. "Steady-State Concepts and Static Liquefaction of Silty Sands", *Journal of Geotech. and Geoenv. Engrg*, Vol. 124, No. 9, pp. 868-877.

Yamamuro, J.A. and K.M. Covert [2001]. "Monotonic and Cyclic Liquefaction of Very Loose Sands with High Silt Content", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 4, pp. 314-324.