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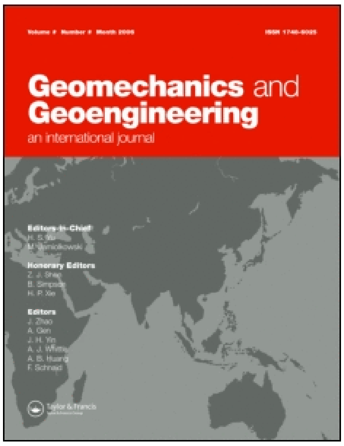

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Influence of cyclic pre-shearing on undrained behaviour of carbonate sand in simple shear tests

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The paper will offer insights regarding the effects of cyclic prestrain history on the undrained behaviour of an uncemented carbonate sand (Quiou sand) through a modified NGI simple shear (SS) apparatus. Tests were carried out on specimens reconstituted at two void ratios (loose and dense) by using the water sedimentation method. The influence of cyclic undrained pre-shearing was investigated by applying both small and large limiting shear strains; after reconsolidation, the specimens were again subjected to the same cyclic phase as before, until the sample liquefied. The pattern of post-cyclic behaviour of the tested sand showed the relevant role assumed by the phase transformation line obtained from monotonic tests on virgin specimens. For both loose and dense specimens a threshold value of cyclically induced pre-shear strain was found at which the positive effect of previous cyclic loading history on liquefaction resistance disappears or starts to reverse. A normalising criterion, capable of providing a unified description of the pore-pressure build-up curves for different pre-shearing histories, was verified. Finally, a direct comparison between the tested sand and a silica sand with similar grain size features was also presented in order to ascertain the influence of particle crushing on the post-cyclic liquefaction behaviour.

Keywords: carbonate sand; pre-shearing; cyclic liquefaction; simple shear

1. Introduction

Undrained mechanical behaviour of sand is heavily influenced by its previous stress-strain history. The effect of any seismic stress-strain history (cyclic 'pre-shearing'), which can increase or decrease the liquefaction resistance, is a matter of concern to a soil engineer, in connection with problems related to previously subjected earthquakes capable of inducing small or relevant cyclic shear strains.

Lessons learned from several earthquakes, such as the 1983 Nihonkai Chubu earthquake (Yasuda and Tohno 1988), have shown that even in a previously liquefied silica sand, re-liquefaction can occur due to an aftershock which is a great deal smaller than the main shock. Most previous studies on this topic have focused on the re-liquefaction behaviour of silica sands under triaxial or simple shear apparatus (Finn *et al.* 1970, Tatsuoka and Yoshihara 1974, Matsuoka *et al.* 1985, Ishihara and Okada 1982) while little attention has yet been paid to carbonate soils (i.e., Kaggwa 1988, Carter *et al.* 2000). Accordingly, further studies are required to comprehensively characterise the behaviour of carbonate sands, and to fully recognise the possible spectrum of responses.

As a result of previous studies it was argued that a subsequent liquefaction of a previously liquefied silica sand depends on several factors, such as initial state variables (void ratio and consolidation stresses), reconstitution method, direction of pre-shear (triaxial compression or extension), etc. A microstructural interpretation of re-liquefaction mechanism of granular soils under cyclic loading has been provided recently by Oda *et al.* (2001).

Conversely, when the cyclic shear strains caused by earthquakes remain below a prefixed value ('small pre-shearing') a strengthening effect caused by pre-shearing was observed (Finn *et al.* 1970, Seed *et al.* 1977, Ishihara and Okada 1978, Shamoto *et al.* 1978). The concept of 'phase transformation state' can be used in sands characterised by a dilative-type response to match a boundary limit between small and large pre-shearing (Ishihara and Okada 1978, 1982, Suzuki and Toki 1984).

The goal of this research is to improve the basic understanding of the mechanics of post-cyclic undrained behaviour of carbonate sands under simple shear (SS) loading mode, which simulates more properly the stress conditions of in in-situ soil during seismic events. The effects of prestrain history, 'triggering' (or not) a liquefaction condition, on the subsequent undrained cyclic stress-strain-strength response are investigated and analysed.

2. Test material and apparatus

Undrained monotonic and cyclic tests were undertaken by using a simple shear (SS) apparatus (Geonor NGI type) (Bjerrum and

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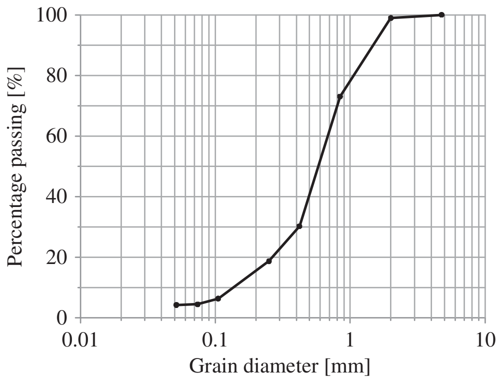

Figure 1. Grain size distribution curve of tested material.

Landva 1966). Cylindrical specimens are about 80 mm in diameter and 20 mm in height and are laterally confined by using a wire-reinforced membrane which ensures approximatively K_0 conditions during shearing and consolidation.

Undrained tests were carried out by automatically adjusting the vertical load in order to maintain the height of the specimen unchanged during shear (constant volume conditions). The change in total vertical stress ($\Delta\sigma_v$) during shear equals the excess pore water pressure (Δu) generated in an equivalent truly undrained test (Dyvik *et al.* 1987).

Several upgrades of the original version of the apparatus were performed by the authors through the introduction of automated electro-mechanical control systems capable of performing monotonic and cyclic simple shear tests under both stress and strain controlled mode (Caridi 2006; Porcino *et al.* 2006).

The tested material was Quiou sand (QS) (Golightly 1989, Bellotti *et al.* 1991, Porcino *et al.* 2005, 2008) which consists of a greyish-white sub-angular to sub-rounded uncemented calcitic carbonate coral sand (Golightly 1989). The specific gravity of QS is G_s = 2.702, and the maximum and minimum index void ratios determined using the Kolbuszewski (1948) method were e_{max} = 1.169 and e_{min} = 0.763. The material was initially oven-dried and passed through a 2 mm sieve in order to remove gravel-sized particles. The gradation curve of the virgin sand (before testing) is shown in Figure 1. All reconstituted samples were prepared by using the water sedimentation (WS) method at two mean values of density index, I_r, namely 40% (loose specimens, L) and 75% (dense specimens, D).

The water sedimentation procedure, first described by Sivathayalan (1994) for simple shear tests and subsequently perfected by Caridi (2006), was adopted. In particular dry sand was placed into the sample cavity filled with de-aired water. The height of drop was maintained at about 1 to 3 mm above the water surface to minimise any segregation process. Additional 'tapping' on the lateral surface of the mould allowed denser specimens to be prepared. At this stage a small seating load was applied to the specimen while keeping drainage lines open.

In order to compare the response of two sands having similar grain-size characteristics but different mineralogical features, some cyclic SS tests were also performed on Ticino (TS) sand (D_{50} = 0.56 mm), a well-studied uncemented natural silica sand whose characteristics are reported in Bellotti *et al.* (1991). Reconstituted specimens of TS sand were prepared at the same density state of QS sand by using the previously described procedure and subjected to K_0-consolidation under a vertical effective stress (σ'_{v0}) equal to 100 kPa. A complete list of the tests performed in the present study is reported in Table 1.

Table 1. Summary list of tests

Test N°	Sand	Test type (first loading)	$\gamma_{lim,\ preshearing}$ [%]	Test type (re-loading)	$I_{r\ (after\ consolidation)}$ [%]	$e_{c\ (after\ consolidation)}$	σ'_{vo}[kPa]	$CSR = \frac{\tau}{\sigma'_{vo}}$
1	QS_L	Cyclic	1	Cyclic	41	1.00	100	0.10
2	QS_L	Cyclic	1	Cyclic	43	0.99	100	0.15
3	QS_L	Cyclic	1	Cyclic	47	0.98	100	0.20
4	QS_L	Cyclic	1	Cyclic	38	1.01	100	0.25
5	QS_D	Cyclic	1	Cyclic	73	0.87	100	0.17
6	QS_D	Cyclic	1	Cyclic	74	0.87	100	0.20
7	QS_D	Cyclic	1	Cyclic	72	0.88	100	0.25
8	QS_D	Cyclic	1	Cyclic	76	0.86	100	0.30
9	QS_L	Cyclic	3.75	Cyclic	37	1.02	100	0.10
10	QS_L	Cyclic	3.75	Cyclic	42	1.00	100	0.15
11	QS_L	Cyclic	3.75	Cyclic	43	0.99	100	0.20
12	QS_L	Cyclic	3.75	Cyclic	36	1.02	100	0.25
13	QS_D	Cyclic	3.75	Cyclic	72	0.88	100	0.15
14	QS_D	Cyclic	3.75	Cyclic	67	0.90	100	0.20
15	QS_D	Cyclic	3.75	Cyclic	77	0.86	100	0.25
16	QS_D	Cyclic	3.75	Cyclic	68	0.89	100	0.30
17	QS_L	Cyclic	2	Cyclic	46	0.98	100	0.20
18	QS_D	Cyclic	2	Cyclic	74	0.87	100	0.30
19	QS_L	Monotonic	-	-	40	1.01	50	-
20	QS_L	Monotonic	-	-	45	0.99	100	-
21	QS_L	Monotonic	-	-	43	0.99	150	-
22	QS_D	Monotonic	-	-	74	0.87	50	-

(*Continued*)

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Table 1. (*Continued*)

Test N°	Sand	Test type (first loading)	$\gamma_{\text{lim, preshearing}}$ [%]	Test type (re-loading)	$I_{r\ (\text{after consolidation})}$ [%]	$e_{c\ (\text{after consolidation})}$	σ'_{vo}[kPa]	$CSR = \frac{\tau}{\sigma'_{vo}}$
23	QS_D	Monotonic	-	-	80	0.84	100	-
24	QS_D	Monotonic	-	-	84	0.83	150	-
25	QS_L	Cyclic	1	Monotonic	39	1.01	100	0.20
26	QS_D	Cyclic	1	Monotonic	72	0.88	100	0.20
27	QS_L	Cyclic	3.75	Monotonic	41	1.00	100	0.20
28	QS_D	Cyclic	3.75	Monotonic	75	0.86	100	0.20
29	TS_L	Cyclic	3.75	Cyclic	43	0.99	100	0.10
30	TS_L	Cyclic	3.75	Cyclic	38	1.01	100	0.15
31	TS_L	Cyclic	3.75	Cyclic	38	1.01	100	0.20
32	TS_L	Cyclic	3.75	Cyclic	45	0.99	100	0.25
33	TS_D	Cyclic	3.75	Cyclic	80	0.84	100	0.15
34	TS_D	Cyclic	3.75	Cyclic	85	0.82	100	0.20
35	TS_D	Cyclic	3.75	Cyclic	79	0.85	100	0.25
36	TS_D	Cyclic	3.75	Cyclic	75	0.86	100	0.30
37	TS_L	Cyclic	1	Cyclic	45	0.99	100	0.20
38	TS_L	Cyclic	2	Cyclic	45	0.99	100	0.20
39	TS_D	Cyclic	1	Cyclic	71	0.88	100	0.30
40	TS_D	Cyclic	2	Cyclic	68	0.89	100	0.30

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3. Test procedure

Three types of cyclic tests were performed on Quiou sand; the tests are separated according to the histories imparted to the specimen, namely:

- initial tests on 'virgin' (or 'fresh') specimens (A-series tests);
- tests with 'small pre-shearing' (B-series tests)
- tests with large pre-shearing' (C-series tests)

The first type of test (i.e., freshly deposited conditions) is designed to replicate in-situ conditions of a soil which has not experienced seismic loading following the depositional process of the material. The results of these tests arise from the first phase of C-series tests.

On the other hand, B- and C-series tests provide insight into the behaviour of a soil with a seismic stress-strain history which may or may not have induced an initial liquefaction condition.

Two types of pre-shearing were mainly imparted to the specimen. In the first type, the tests were terminated when 1% limiting shear strain (in single amplitude) was reached ($\gamma_{SA} = 1.00\%$). This stress-strain history will be referred to as 'small pre-shearing' (B-series tests). In the second test type, a larger value of limiting shear strain, coincident with the generally adopted triggering liquefaction criterion ($\gamma_{SA} = 3.75\%$) (Ishihara 1993, NRC 1985), was selected. This stress-strain history will be referred to as 'large pre-shearing' (C-series tests). It is noteworthy that other liquefaction 'triggering' threshold criteria could be adopted (Wu *et al.* 2004) to study the response of soils subjected to repeated loading; nevertheless, in the present study it was decided to terminate tests in correspondence with a single amplitude shear strain equal to 3.75% in order to ensure a better control of undrained/constant volume test conditions. Additionally, a limited number of cyclic tests were carried out at an intermediate cyclic pre-strain level ($\gamma_{lim} = 2\%$ in S.A.). This was intended to examine more closely the effect caused by a previous cyclic loading history on the cyclic liquefaction resistance of tested sands.

All cyclic SS tests were carried out at an initial effective vertical stress (σ'_{v0}) equal to 100 kPa. When the prefixed limiting strain level was achieved, cyclic loading was terminated and the shear stress had returned to zero with measured residual shear strains. This condition is similar to that which is expected in realistic 'field' level ground situations.

Even though the residual strain, measured from where post-cyclic loading commences, has been adopted by other authors as a key factor in controlling the liquefaction resistance after a pre-shearing phase (Ishihara and Okada 1982), it is an intrinsically random quantity whose value cannot be exactly determined. Accordingly, in the present study, the prefixed limiting shear strain reached in the cyclic phase, was assumed to characterise the cyclic pre-shearing phase.

Sample was then subjected to reconsolidation, corresponding to the dissipation of in-situ excess pore water pressures induced during earthquake loading. At this stage the sample was brought to the same initial effective vertical stress ($\sigma'_{v0} = 100$ kPa) while measuring vertical strains with the accompanying changes in density.

After re-consolidation, for B- and C-series tests a second cyclic phase was carried out under identical cyclic stress ratio (CSR) of the initial test, until the selected failure criterion was achieved ($\gamma_{SA} = 3.75\%$) All cyclic tests were conducted in stress-controlled mode at a frequency of 0.02 Hz.

It should be noted that any specimen at re-loading had a change in density index values, depending on the maximum strain amplitude experienced at the end of cyclic loading. A pre-cyclic consolidation line (virgin) and a post-cyclic consolidation line (re-consolidation) were drawn from height change data obtained during the tests. Figure 2 shows example plots of void ratio (e) versus consolidation effective vertical stress (σ'_{v0}) for both loose and dense specimens. The post-cyclic compression line indicates a less compressible behaviour when compared to that exhibited in the first virgin consolidation; this pattern of behaviour is more evident in tests with small pre-shearing than with large pre-shearing (it

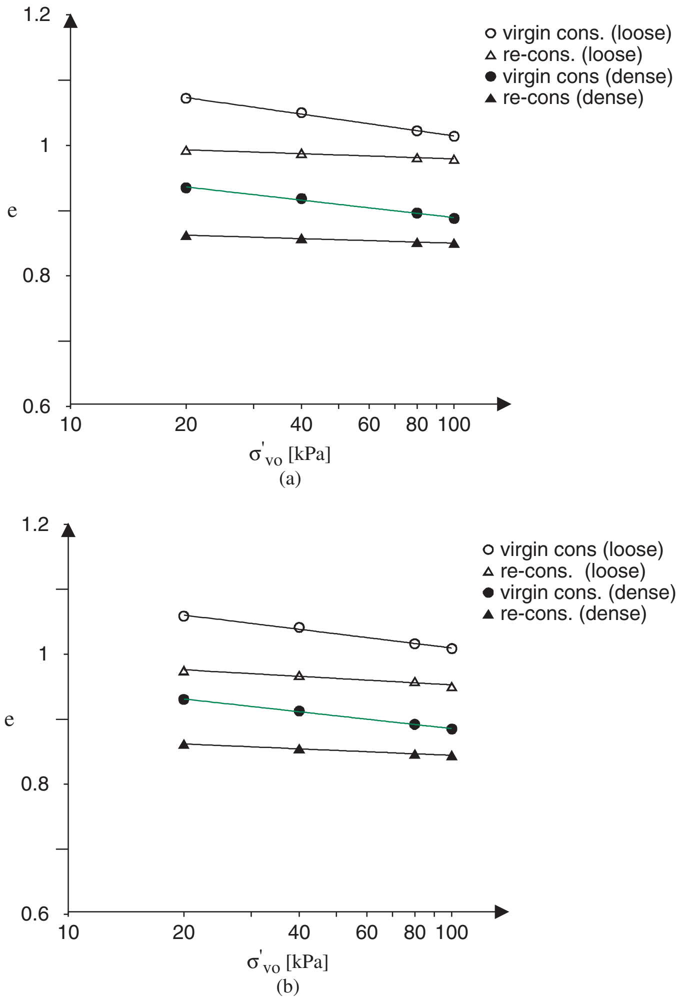

Figure 2. Compressibility features of Quiou sand specimens following (a) small-pre-shearing and (b) large-pre-shearing.

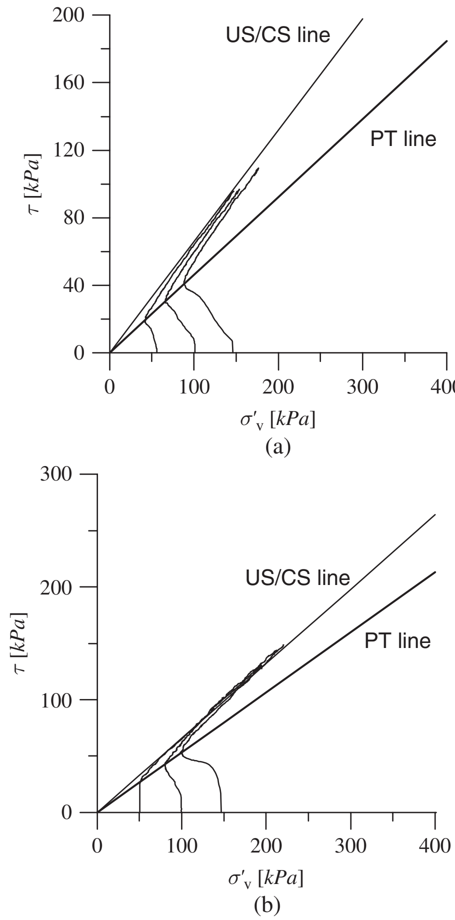

Figure 3. Undrained monotonic response of virgin QS sand: (a) loose and (b) dense.

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can be noted that the void ratio at the beginning of re-consolidation and at the end of first consolidation are not perfectly coincident because of minor disturbance effects resulting from the setting-up of equipment before the undrained shearing phase).

In addition to the cyclic test programme, a few monotonic SS tests were performed on both virgin and cyclically pre-sheared specimens of QS with the aim of examining the effect of a previous cyclic loading history on undrained monotonic strength of the tested sand. Tests were conducted in strain controlled mode at a displacement rate of 0.1 mm/min.

Figure 3 shows the monotonic undrained response of both loose and dense QS sand specimens with no prestrain history (virgin). Similar stress-path trends can be observed, regardless of initial void ratio and vertical effective stress: specimens show a strain-hardening type behaviour after a phase transformation state (Ishihara *et al.* 1975) was achieved. After this transition phase, the stress-paths reach the ultimate state (US) strength envelope corresponding to a mobilised internal friction angle at the ultimate/steady state ϕ'_{US} of 41.4°.

4. Influence of pre-shearing on undrained cyclic behaviour

With the aim of showing the effects of a previous cyclic loading history on tested sand, in Figures 4 and 5 the stress-paths and the corresponding stress-strain plots relative to two cyclic test types of 'small' (Figures 4a and 5a) and 'large' pre-shearing (Figure 4b and 5b) are depicted. Test results refer to loose specimens subjected to the same cyclic stress ratio ($CSR = \tau_{cyc}/\sigma'_{V0}$) equal to 0.15.

First, it can be verified that the definition of small and large pre-shearing adopted in the present study satisfies the criterion generally introduced in literature by other authors (Ishihara and Okada 1978, 1982, Suzuki and Toki 1984). According to this criterion the phase transformation line obtained from monotonic tests (PTL_{mon}) can be considered as a boundary line separating the two different domains.

In Figures 4b and 5b we can see the same plots of the previous figures relative to the second cyclic phase (after re-consolidation) until an initial liquefaction condition (γ_{SA} = 3.75%) is reached. The effect of large pre-shearing on tested sand (Figure 5b) is to cause a small decrease in cyclic resistance since the cycle number required to reach a 3.75% single amplitude shear strain is equal to 13 in the second phase

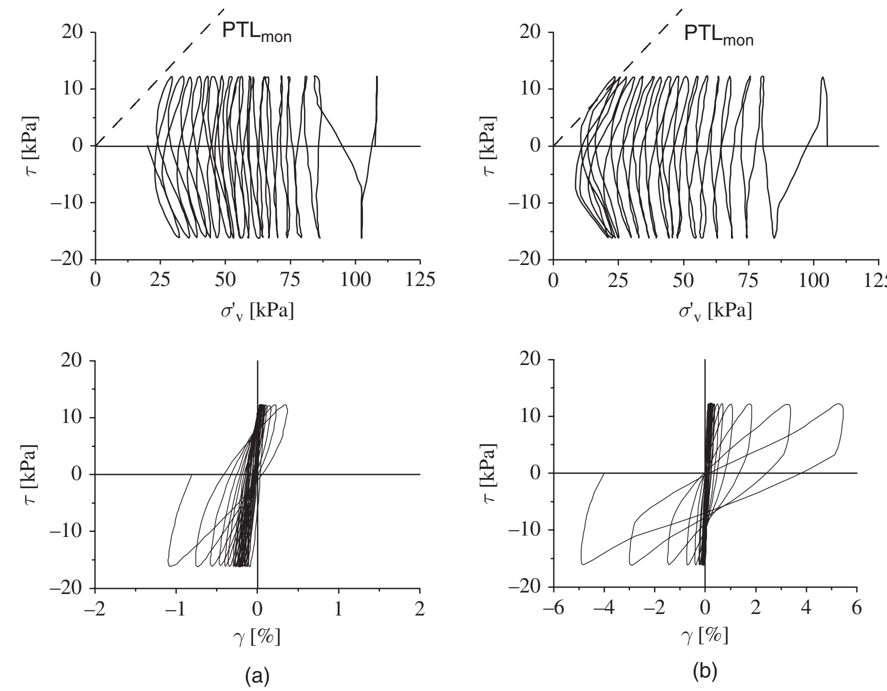

Figure 4. Application of (a) small and (b) large pre-shearing on loose specimens of QS sand.

as opposed to 16 in the first phase (virgin specimen). Conversely, after a small pre-shearing phase (Figure 5a), QS sand specimens can withstand many more cycles ($\cong$49) than they could in their virgin state before triggering initial liquefaction.

It is noteworthy that a ‘large pre-shearing’ phase seems to induce in the sample a more pronounced asymmetric trend of shear strains (Figure 5b), when compared with the freshly prepared specimens (Figure 4b).

The cyclic phase transformation (PT_{cyc}) lines, which separate a dilative response from a contractive one in individual cycles, are superimposed on the τ–σ'_v plots of Figure 5. The ratio corresponding to the PT_{cyc} line in the positive shearing direction is approximately 0.21, which locates well below the

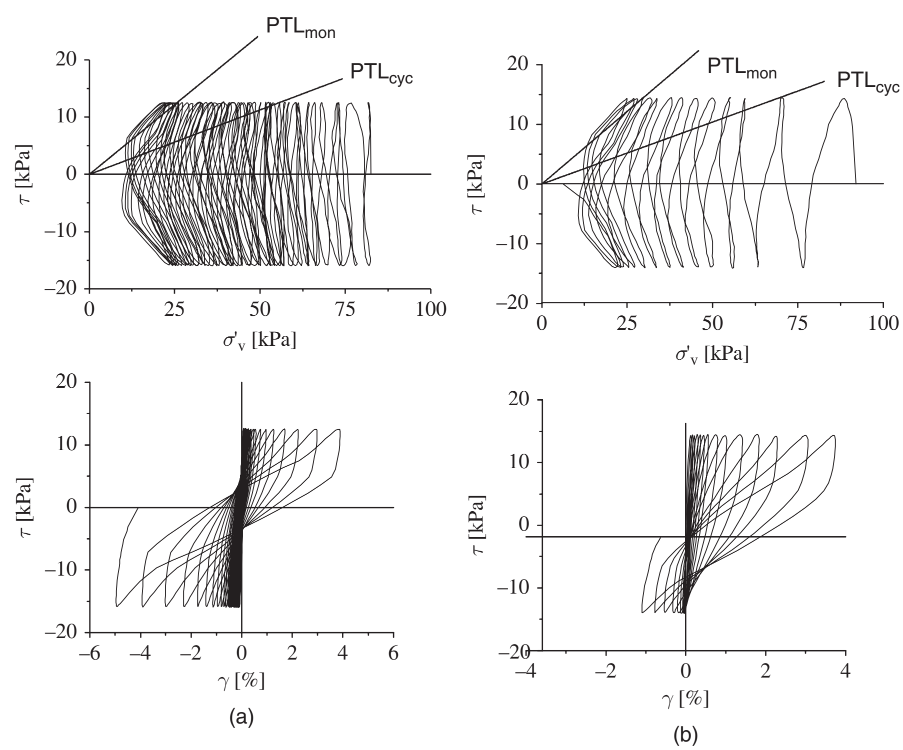

Figure 5. Effect of cyclic re-loading on loose specimens of QS sand previously subjected to (a) small and (b) large pre-shearing.

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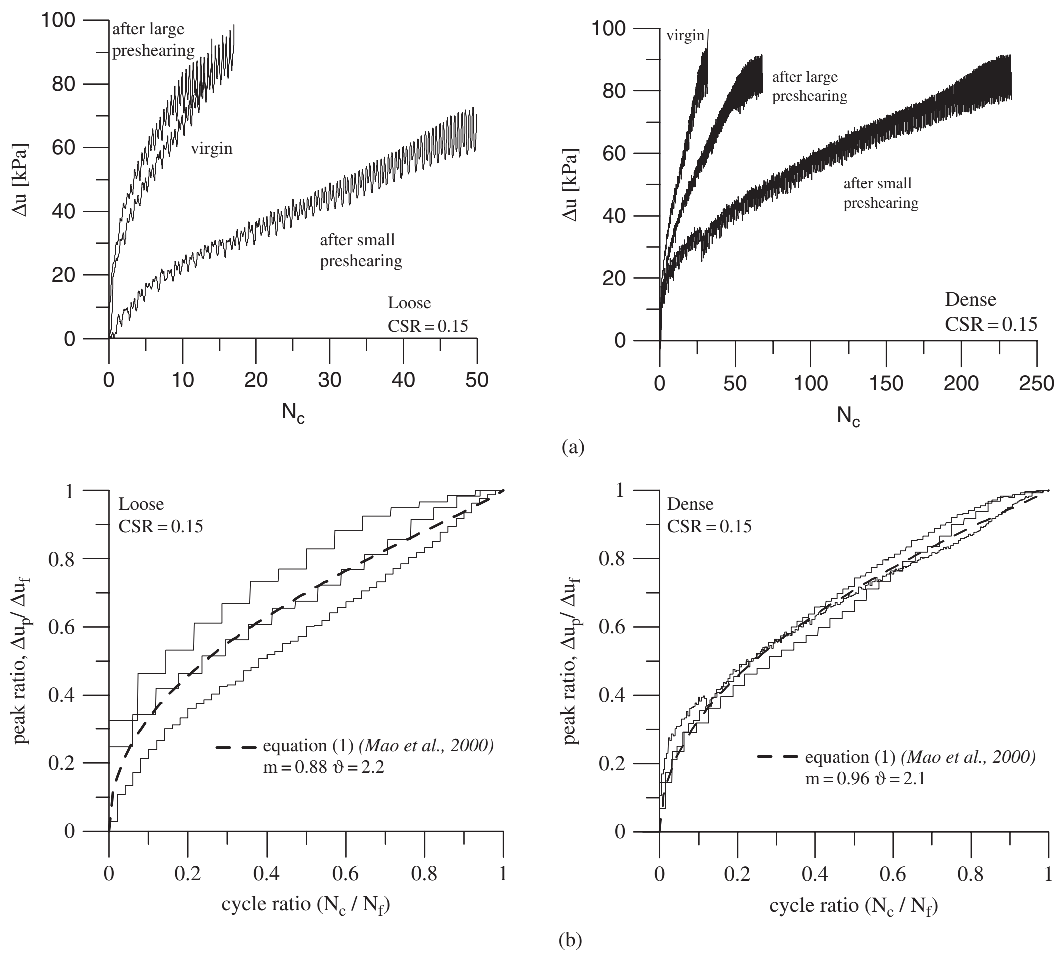

Figure 6. Pore pressure generation curves before (a) and after normalisation (b) of the cyclic simple shear tests on virgin and pre-sheared specimens of QS sand.

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PT strength envelope determined by monotonic tests, as shown in Figure 5.

In order to highlight the effect of pre-shearing on pore pressure generation behaviour of tested sand, Figure 6a shows the pore pressure build-up curves measured in six simple shear tests carried out on virgin and pre-sheared loose and dense specimens of Quiou sand. It can be observed that the rate of generation of excess pore pressure with cycle number increases with the magnitude of applied pre-shear strain.

With the purpose of better comparing pore pressure build-up curves from different tests, normalisation of current peak excess pore pressure by the maximum accumulated value at 'failure' (i.e., $\Delta u_p/\Delta u_f$) and normalisation of the cycle number by the number of cycles at 'failure' (i,e. N_c/N_f) can be properly adopted (Mao *et al.* 2000), as illustrated in Figure 6b. In the present study, as previously mentioned, Δu_f is defined as the value of Δu corresponding to a 3.75% S.A. shear strain. It can be seen that the normalisation process produces pore pressure generation curves for these tests that agree quite well with each other, regardless of initial density state and previous cyclic preloading history.

The amount of excess pore pressure generated in undrained cyclic loading is often used as the critical damage criterion in seismic-related foundation design. Therefore, many empirical correlations have been developed for characterizing the generation of pore water pressure due to cyclic loading (Lee and Albaisa 1974, Marcuson *et al.* 1990, Mao *et al.* 2000, Kammerer 2002, etc.) or for correlating the excess pore water pressure ratio in the first loading cycle with the number of cycles required to trigger liquefaction (Oda *et al.* 2001).

Mao *et al.* (2000) proposed the following expression to describe the normalised pore pressure build-up curves of calcareous soils in simple shear tests under generalised cyclic loading conditions:

$$\frac{\Delta u}{\Delta u_f} = \left[1 - \left(\frac{N_c}{N_f}\right)^m\right]^{1/\vartheta} \tag{1}$$

θ and Δu_f are related to the cyclic shear stress (τ_{cyc}) by the following expressions:

$$\theta = \theta_0 + p \cdot (\tau_m/\tau_{cyc}) \tag{2}$$

$$\Delta u_f = \sigma'_{vc} - \frac{(\tau_m + \tau_{cyc})}{M_f} \tag{3}$$

In eq. 1 to 3, m, p, and θ_0 are constants ($\theta = \theta_0$ for symmetric tests), σ'_{vc} is the consolidation vertical effective stress, and M_f is the slope of the critical state line.

Such an analytical model has been applied in the present study to symmetrical simple shear tests ($\tau_m = 0$) carried out on both virgin and pre-sheared specimens of Quiou sand. The model output curves are shown in Figure 6b together with the corresponding model parameters. As can be seen, the pattern of normalised curves in Figure 6b is nicely captured by the model.

5. Influence of pre-shearing on cylic liquefaction resistance

In Figures 7 and 8 the cyclic liquefaction resistance curves of virgin specimens (with no previous seismic history) are reported against those of pre-sheared specimens of QS sand. The curves are reported in terms of cyclic stress ratio (CSR = τ_{cyc}/σ'_{V0}) against the number of cycles (N_c) required to reach a shear strain of 3.75% in single amplitude. The results refer to both loose (Figure 7) and dense (Figure 8) specimens of QS sand.

It was discovered that specimens subjected to 'small pre-shearing' are far less susceptible to initiation of liquefaction than virgin specimens, irrespective of density index value. In particular, the increase in cyclic stress ratio required to cause failure after 10 cycles is 35% and 42% for loose and dense specimens respectively. Such results confirm the findings drawn by other authors for silica (Finn *et al.* 1970, Lee and Albaisa 1974, Seed *et al.* 1977, Shamoto *et al.* 1978) and carbonate sands (i.e., Kaggwa 1988).

Such a change in behaviour due to a small pre-strain cannot be explained solely in terms of the change in density index brought about by reconsolidation, which are relatively small. As shown in figure 9, upon cyclic loading the virgin sample, with approximately the same initial density state as the sample after 1% prestrain history, manifests a quite different pattern of behaviour in terms of cyclically induced pore pressure (Δu) and shear strains (γ).

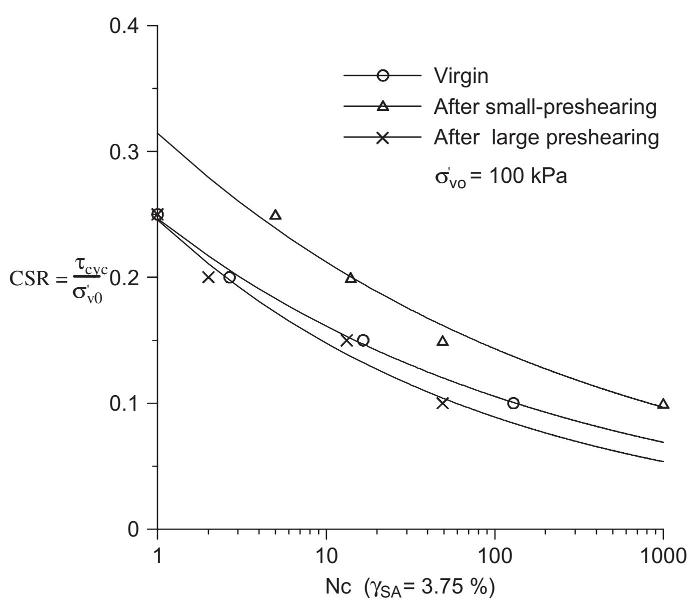

Figure 7. Cyclic liquefaction resistance curves of QS sand for both virgin and pre-sheared specimens (loose).

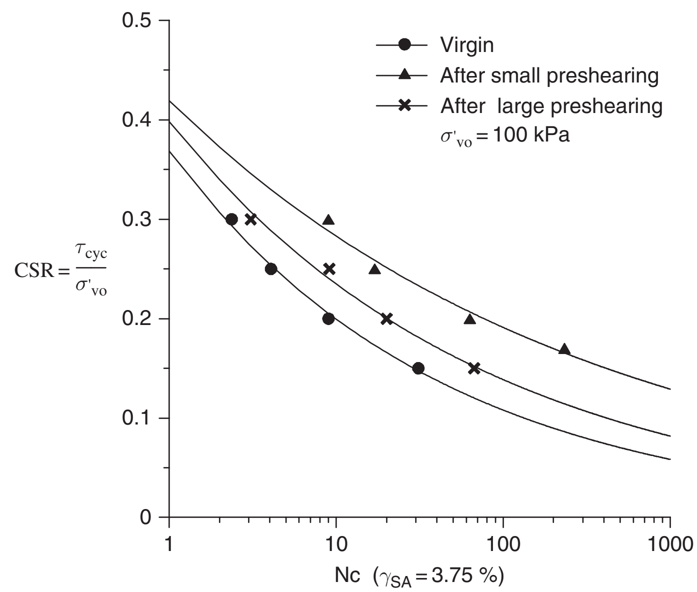

Figure 8. Cyclic liquefaction resistance curves of QS sand for both virgin and pre-sheared specimens (dense).

The undrained cyclic strength of samples subjected to a stress-strain history triggering initial liquefaction (large pre-shearing) is also shown in Figures 7 and 8. Figure 8 shows that for dense specimens the strengthening effects associated with small pre-shearing still persist in case of a large pre-shearing type strain- history. On the other hand, the liquefaction resistance of loose pre-sheared specimens at γ_{lim} = 3.75% (Figure 7) is similar to (or only slightly lower than) that of the virgin specimens.

Such behaviour shows some differences with respect to the findings gathered by other authors on silica sands (Finn *et al.* 1970, Ishihara and Okada 1978, Suzuki and Toki, 1984, Oda *et al.* 2001) for which the liquefaction resistance is in general significantly reduced when samples have been pre-sheared cyclically beyond a threshold value.

Since particle crushing is an important feature of the tested material, an investigation was made into the behaviour of a silica sand having similar grain size characteristics to QS sand but less susceptible to particle crushing.

To this purpose, in Figure 10 the comparative response of Ticino silica sand is reported. Test results refer to both loose and dense 'virgin' samples (with no previous seismic history) together with large pre-sheared specimens (limiting shear strain equal to 3.75% in S.A.).

The results of re-liquefaction tests highlight a more unstable response for loose silica sand specimens with respect to carbonate ones, even though they became denser due to dissipation of excess pore water pressure after the first liquefaction.

One might expect that the differences between the two sands in the subsequent cyclic phase are linked to grain crushing effects, however, further studies are needed to fully understand this phenomenon on a microstructural basis.

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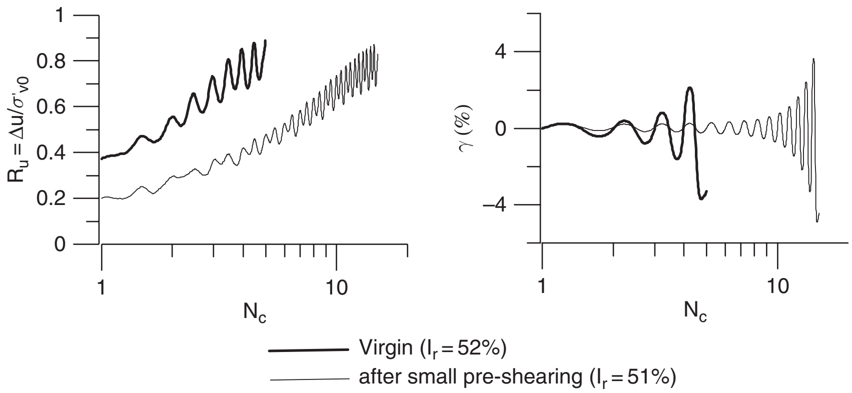

Figure 9. Comparison between the undrained cyclic response of a virgin and a pre-sheared specimen of QS sand prepared at the same density index.

The influence of cyclic pre-shearing on the liquefaction resistance of sand can be better viewed by the change in the number of cycles required to cause liquefaction in cyclic re-loading, ($N_{c,r}$), against the shear strain ($\gamma_{\lim}$) reached during the pre-shearing phase, as shown in Figure 11.

With the aim of showing a possible trend, additional tests were carried out on both Quiou and Ticino sands tested at an equivalent density state and at prefixed cyclic stress ratios (CSR). From Figure 11, it is obvious that as shear strain ($\gamma_{\lim}$) increases, so does the value of cyclic number ratio $R_n = \frac{N_{c,r}}{N_{c,virgin}}$, indicating a positive effect of the previous cyclic history up to a peak and, thereafter, it begins to decrease. For the considered initial density states and cyclic stress ratios, the values of $\gamma_{\lim}$ at which the peak appears were approximately 1% for QS sand and

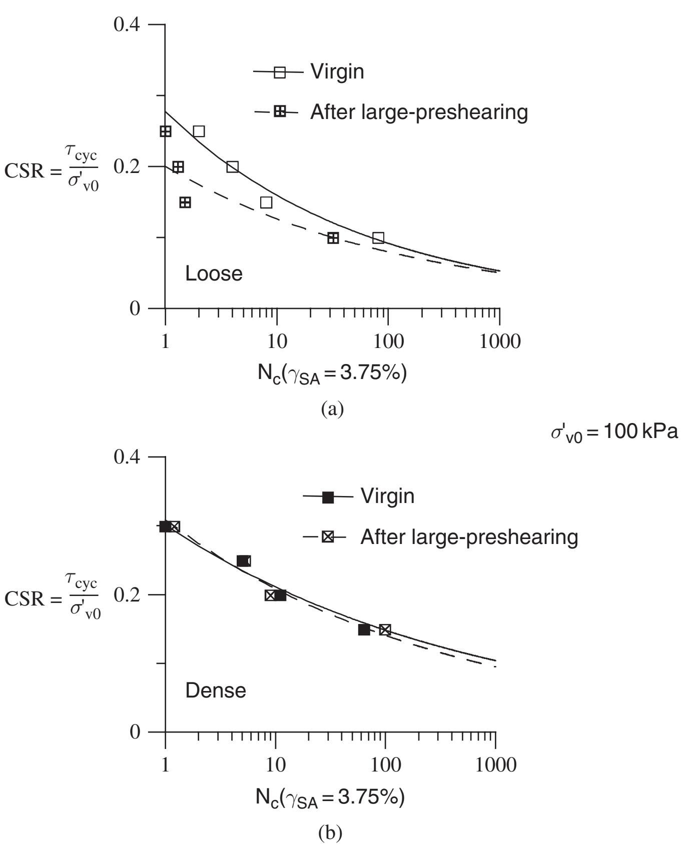

Figure 10. Cyclic liquefaction resistance curves of Ticino silica sand of both virgin and pre-sheared specimens (large pre-shearing): (a) loose and (b) dense.

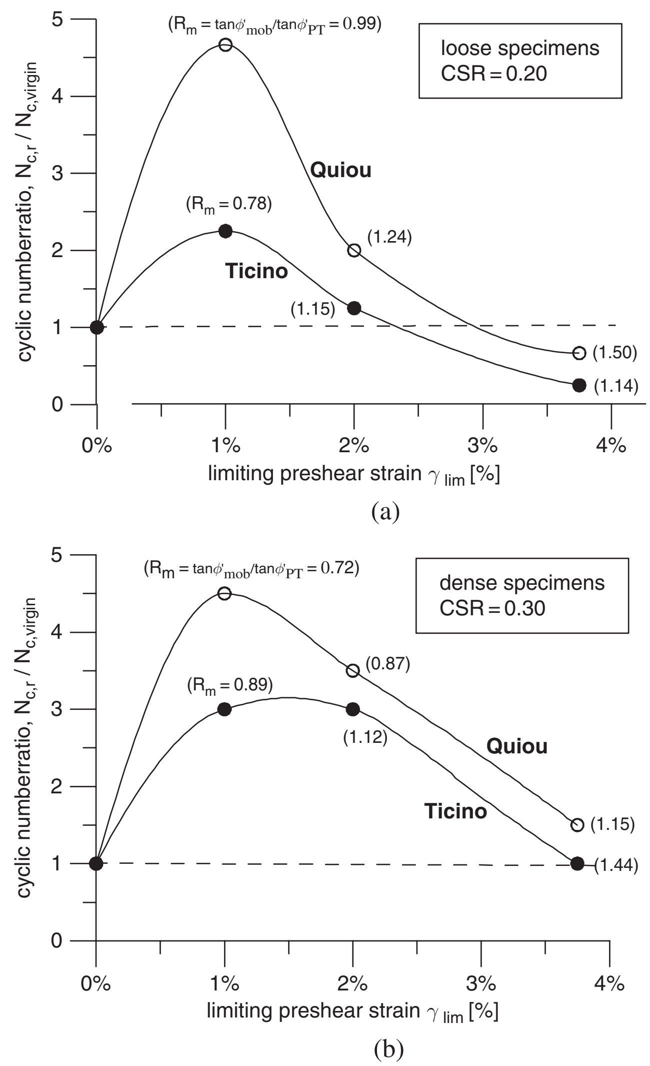

Figure 11. Effect of pre-shearing on the number of cycles required to cause liquefaction for QS and TS sands: (a) loose and (b) dense.

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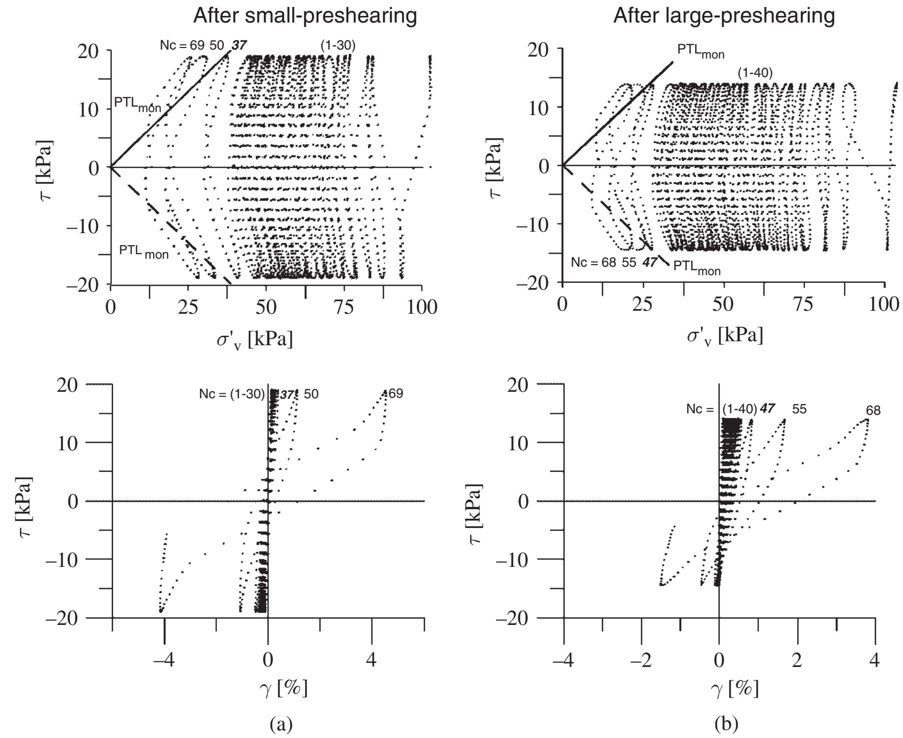

Figure 12. Correspondence between monotonic phase transformation line of virgin specimens of QS sand and cyclic response after pre-shearing (dense specimens).

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between 1% and 1.5% for TS sand. These values of limiting pre-shear strain correspond to the mobilised stress ratio at pre-shearing $R_m = \frac{\tan\phi'_{mob}}{\tan\phi'_{PT}} = 0.72$ to 0.99, where $\phi'_{\rm mob}$ is the mobilised internal friction angle and $\phi'_{\rm PT}$ is the internal friction angle obtained by monotonic tests at phase transformation state. In Figure 11 the values of R_m for the considered tests are reported in brackets. From the trend of the curves in Figure 11 it is clearly evident that, irrespective of the type of sand and initial density state, a threshold value of $\gamma_{\rm lim}$ exists at which the positive effect of pre-shearing tends to vanish or even to invert. For the considered cyclic stress ratios such a threshold value is located between 3% and 4.3% for QS or 2.5% and 3.75% for TS. Larger cyclic pre-strains tend to cause a decrease in liquefaction resistance with respect to virgin samples for both QS and TS sands.

6. Correspondence between undrained monotonic and cyclic behaviours

In the literature, it is widely recognised that the undrained cyclic behaviour of sands (with no previous seismic history) is governed by PT line obtained from monotonic tests (Hyodo *et al.* 1994, Vaid and Chern 1985, Alarcon-Guzman *et al.* 1988). In order to clarify the cyclic behaviour of prestrained specimens of tested sand, Figure 12 presents the stress-strain hysteretic loops together with the stress-paths relative to SS tests carried out on QS sand specimens subjected to a previous cyclic history of small (a) and large (b) pre-shearing . In the same figure the phase transformation line assessed by monotonic tests ($\mathrm{PTL_{mon}}$) on virgin samples was also superimposed.

From these results, it appears that cyclic behaviour of pre-strained specimens is strictly related to the position of virgin monotonic PT line, irrespective of type of loading history (small or large). As is shown, when the effective stress-path crosses the phase transformation failure envelope obtained from monotonic tests for the first time, the rate of shear strain development starts to increase and, correspondingly, a significant decrease of cyclic stiffness can be observed.

Figure 12 also suggests that the PT strength envelope is little affected by the previous seismic history of tested specimens. These findings are confirmed by the results reported in Figure 13, where the stress-paths relative to post-cyclic undrained monotonic response of loose QS specimens are depicted.

It can be noted that the application of a cyclic 'small pre-shearing' ($\gamma_{\rm lim}$ = 1%) causes a more evident 'strain-hardening' type behaviour in the subsequent monotonic test, until reaching the phase transformation line (PTL) identified for the virgin specimens. However, such a line is crossed in correspondence to two different shear stress values ($\tau_{\rm PT}$).

On the other hand, when a sample is subjected to cyclic 'large pre-shearing' (Figure 13) ($\gamma_{\rm lim}$ = 3.75%), the differences are less marked with respect to the virgin sample. In the ultimate stage of monotonic tests, at large shear strain levels, all the specimens (virgin and pre-sheared) reach the unique ultimate strength envelope (USL).

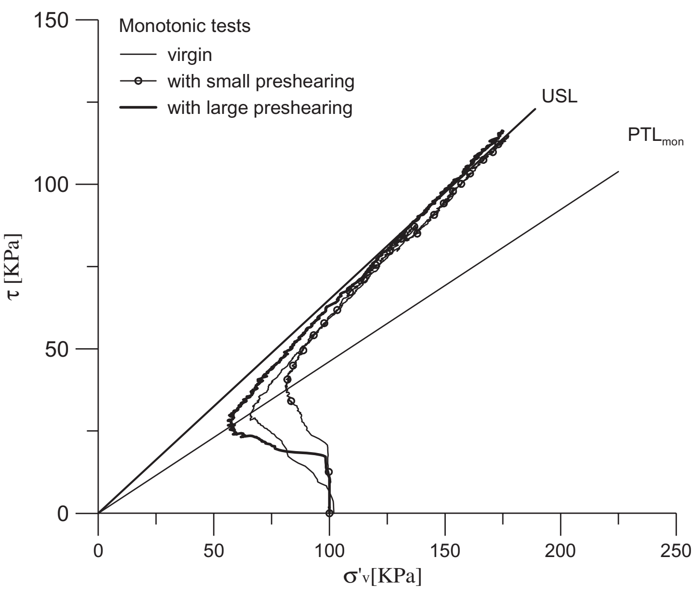

Figure 13. Effect of small and large preshearing on undrained monotonic response of loose sand specimens.

7. Conclusions

Undrained monotonic and cyclic simple shear tests were conducted through a modified NGI simple shear (SS) apparatus on Quiou (QS) sand to clarify the effect of seismic history of carbonate sands. The influence of prestrain history on both monotonic and cyclic response of tested sand was studied by terminating the cyclic shearing at two specified levels, namely γ_{lim} = 1% and 3.75%.

A more complete appraisal of the effect caused by a previous cyclic loading history on the cyclic liquefaction resistance of tested sands was reached by performing additional tests at an intermediate cyclic pre-strain level (γ_{lim} = 2% in S.A.).

Test results showed that undrained behaviour of tested sand is strongly affected by pre-strain history. In summary, the cyclic response on reloading following cyclic pre-strain history revealed that:

- The phase transformation line assessed by monotonic tests on virgin specimens (PT_{mon}) plays a relevant role in the cyclic undrained behaviour of tested sand at reloading: this means that liquefaction conditions at re-loading depend on the crossing of the stress-paths with the above PT_{mon} strength envelope. The above phase transformation line appears to depend very little on the previous applied loading history (virgin or pre-sheared specimens) and initial density index of tested sand.
- The cyclic small pre-shearing (γ_{lim} = 1% in S.A.) brings about a hardening effect in the specimens because they become more resistant to the pore-water pressure build up and tend to be less prone to the onset of liquefaction; this effect is apparent on both loose and dense specimens.
- For large pre-shearing (γ_{lim} = 3.75% in S.A.) the behaviour follows different patterns depending on whether the specimens are loose or dense. In particular, the hardening effect observed for small pre-shearing also occurs in the case of dense specimens; conversely, in the case of loose specimens, a small decrease in liquefaction resistance of pre-loaded vs. non-preloaded specimens was revealed.
- The normalising criterion of the pore-pressure build-up curves proposed by Mao *et al.* (2000) was verified to provide a unified description of pore-pressure generation, regardless of different pre-shearing history type.
- The effect of stress-history involving large pre-shearing is different for Quiou sand and a silica sand having similar grain size characteristics: Ticino sand (TS). In particular, once liquefied, loose specimens of TS sand show a lower resistance to re-liquefaction, while in dense specimens no significant effects were found. The reason for such differences between the two sands should be linked to the grain crushing features of carbonate soils.

In summary, the effect of a previous cyclic loading history is in general positive in the case of small pre-shearing. This effect tends to increase with γ_{lim} up to a peak which appears in correspondence with a limiting pre-shear stress ratio smaller than that mobilised at the phase transformation state in monotonic tests.

It was found that, for a threshold value of γ_{lim}, the positive effect of pre-shearing tends to vanish or even to invert. Larger cyclic pre-strains will cause a decrease in liquefaction resistance with respect to virgin samples, irrespective of the initial density state of the samples.

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