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Undrained monotonic and cyclic simple shear behaviour of carbonate sand

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The paper presents a study of the undrained behaviour of an uncemented carbonate sand (Quiou sand) under simple shear loading conditions. The experimental study was conducted through cyclic and monotonic undrained/constant volume simple shear tests carried out on reconstituted specimens prepared by using the sedimentation in water (WS) method. Tests were carried out on specimens reconstituted at two void ratios (i.e. loose and dense) and different effective consolidation stresses. Furthermore, to account for the effect of non-zero mean shear stress level, cyclic simple shear tests were performed under both symmetrical and non-symmetrical cyclic loading. Two types of failure modes have been observed in cyclic tests, that is 'cyclic liquefaction' or 'cyclic mobility', depending on whether or not they were conducted under shear stress reversal conditions. A unified framework seems to exist whereby undrained monotonic and cyclic response can be comparatively analysed. In particular, normalising the cyclic liquefaction resistances obtained from symmetrical tests by phase transformation strengths determined in corresponding monotonic tests, provides a cyclic liquefaction resistance curve which was found to be unique, irrespective of initial void ratio and vertical effective stress. Undrained cyclic shear strength of the tested sand appears to be affected by the presence of a non-zero mean shear stress, following a pattern of behaviour, which is similar for both loose and dense specimens. Furthermore, the normalised stress–strain curves of the cyclic tests show back-bone curves that are practically coincident with the equivalent monotonic curves.

KEYWORDS: calcareous soils; earthquakes; failure; laboratory tests

La présente communication donne des informations sur le comportement non drainé d'un sable carbonaté non cimenté (Quiou) soumis à de charges de cisaillement simple. L'étude expérimentale a été menée par le biasis de tests de cisaillement simples cycliques et monotones non drainés/à volume constant, effectués sur des spécimens reconstitués préparés en utilisant la méthode de sédimentation dans l'eau. Des tests ont été effectués sur des echantillons reconstitués avec deux indices de vide (à savoir lâche et dense), et différentes contraintes de consolidation. En outre, dans le but de tenir compte de l'effet d'un niveau de contrainte de cisaillement moyenne non nul, des tests de cisaillement simples cycliques ont été effectués sous charge cyclique symétrique et asymétrique. On a relevé deux types de modes de défaillance au cours des tests cycliques, à savoir « liquéfaction cyclique » et « mobilité cyclique », selon que ces tests étaient effectués, ou non, en condition d'inversion des contraintes de cisaillement. Il semble que l'on soit en présence d'un cadre unifié dans lequel il est possible d'effectuer une analyse comparée des réponses monotones et cycliques non drainées. En particulier, la normalisation des résistances de liquéfaction cyclique, obtenues dans le cadre de tests symétriques, par des resistances de transformation de phase déterminées dans des tests monotones correspondants, permet d'obtenir une courbe de résistance à la liquéfaction cyclique qui s'est avérée unique, indépendamment de l'indice de vide initial et des contraintes efficaces verticales. La résistance cyclique au cisaillement non drainée des sables testés semble être affectée par la présence d'une contrainte de cisaillement moyenne non nulle, suivant un tendance de comportement similaire pour des spécimens lâches et denses. En outre, les courbes de contrainte - déformation normalisées des tests cycliques indiquent des courbes de base coïncidant pratiquement avec les courbes monotones équivalentes.

INTRODUCTION

In recent years the work on understanding and modelling the behaviour of uncemented calcareous soils has been tackled on a number of fronts. Much of this has focused on compression and monotonic behaviour (Semple, 1988; Golightly, 1989; Coop, 1990; Carter *et al.*, 2000; Coop & Airey, 2003; Coop, 2005).

Several investigations, however, have also been devoted to the undrained behaviour under cyclic loading with special emphasis on the following features: examination of suitable liquefaction criteria (Hyodo *et al.*, 1998; Fahey, 2001); development of theoretical and empirical models for predicting the rate of pore pressure generation due to cyclic loading and strain response (Kaggwa *et al.*, 1988; Mao *et al.*, 2000); effect of different combinations of mean and cyclic shear stress level (Kaggwa & Poulos, 1990; Mao & Fahey, 2003); application of 'fatigue loading' based theories in order to account for the effect of non-zero mean stress (Joer *et al.*, 1995; Mao *et al.*, 1999); repeated loading subjected to a large number of cycles (Airey & Fahey, 1991).

Various researchers (Hyodo *et al.*, 1998; Mao and Fahey, 2003) showed that a good correspondence exists between undrained behaviour under monotonic and cyclic loading conditions. In particular, such equivalence has been characterised in terms of both effective stress-paths and 'current' (actual) shear stress-ratio (τ/σ'_v) plotted against shear strain (γ) plots (Mao & Fahey, 2003). From these studies it appears that the back-bone curves of cyclic tests correspond well to the monotonic τ/σ'_v–γ curves, if the shearing rate adopted in the two types of tests is similar.

It was also demonstrated by Hyodo *et al.* (1998) that, plotting the results of cyclic triaxial tests as liquefaction resistance normalised with respect to a monotonic reference

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shear strength (i.e. phase transformation state) against number of cycles, is an appropriate means of unifying the response observed, irrespective of initial effective confining pressure, density index and type of material.

Although significant ongoing advances in understanding basic responses of such materials have been made, work in this area is still in progress Thus, it appears important to collect more data so as to determine the best modelling approaches for the various materials and for different applications, and to determine the important similarities and differences in their responses.

The current paper aims at providing a contribution to the fundamental understanding of undrained strength characteristics of uncemented carbonate sands through monotonic and cyclic simple shear tests. A skeletal uncemented carbonate sand, Quiou sand (QS), has been tested in a Norwegian Geotechnical Institute (NGI) type simple shear apparatus under undrained/constant volume conditions. It is well recognised that simple shear loading more accurately reproduces field loading conditions expected during earthquakes and thus it is preferred over cyclic triaxial testing.

In cyclic tests, reconstituted specimens were tested over different values of vertical effective consolidation stress and void ratio. In order to investigate the influence of a non-zero mean shear stress on undrained cyclic shear strength, often referred to as initial (i.e. static) shear stress, cyclic tests were performed at different loading conditions (i.e. symmetrical plotted against non-symmetrical).

SIMPLE SHEAR TESTING

Simple shear (SS) tests were conducted using a modified version of the Geonor/NGI (Bjerrum & Ladva, 1966) apparatus. The NGI version of the SS apparatus (Porcino *et al.* 2006) is one of the most commonly used: it is characterised by a cylindrical sample (80 mm in diameter and 20 mm in height) laterally confined by a rubber membrane reinforced by steel wires. The lateral restraint provided by the reinforced membrane assures a condition of zero horizontal extension on the vertical boundary.

Undrained tests were carried out under 'constant volume' conditions. In this way changes in applied vertical stress ($\Delta\sigma_v$), which are required to keep the sample height constant, are assumed to be equal to the excess pore water pressure (Δu) that would develop if the test were truly undrained with pore pressure measurements (Finn, 1985; Dyvik *et al.*, 1987). The main advantages of constant volume testing using an NGI-type configuration are

(*a*) specimen does not need to be fully saturated
(*b*) no problems associated with pore water pressure compliance during tests (i.e. membrane penetration effects)
(*c*) specimen preparation and test procedure are easier to perform.

To enable an automatic adjustment of the vertical stresses in undrained tests under both monotonic and cyclic loading conditions, the standard NGI type SS apparatus has been modified through an automated electro-mechanical control system (Caridi, 2006; Porcino *et al.*, 2006).

The tested material was QS (Golightly, 1989; Bellotti *et al.*, 1991; Porcino *et al.*, 2005a) dug out from a borrow pit close to the village of Plouasne in Brittany (France). It consists of a greyish white sub-angular to sub-rounded uncemented calcitic carbonate coral sand (Golightly, 1989). The material was initially oven-dried and passed through a 2 mm sieve in order to remove gravel-sized particles. The grading curve and the mineralogical properties of the soil before testing are presented in Fig. 1 and Table 1, respectively. The specific gravity of QS is $G_s = 2{\cdot}702$, and the maximum (e_{max}) and minimum (e_{min}) void ratios determined using the Kolbuszewski (1948) method were 1·169 and 0·763, respectively.

Figure 2 shows typical one-dimensional compression curves of the tested sand obtained from dry reconstituted specimens in a specially designed oedometer apparatus, in terms of specific volume ($v = 1 + e$), and mean normal effective stress (p'). K_0 values necessary to assess p' for these curves were drawn from anisotropically K_0 consolidated tests performed in the triaxial apparatus on the same sand. For the sake of comparison in the same figure, the isotropic normal compression line provided by Coop (2003) is also reported.

Reconstituted specimens were prepared at two values of density index ($I_r \cong 40\%$ and $I_r \cong 75\%$ on average) using sedimentation in water (WS) method, which was shown in previous investigations closely to replicate the in situ fabric of natural sand deposits of marine origin (Vaid *et al.*, 1999; Porcino *et al.*, 2004; Ghionna and Porcino, 2006).

In preparing samples, the sand was spooned gently into the water, layer by layer, until the height was just above the

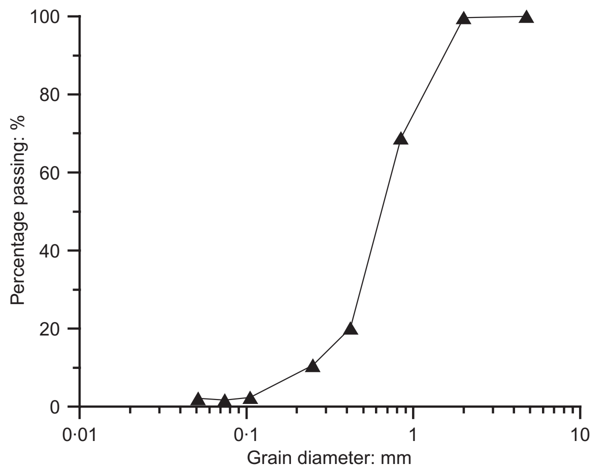

Fig. 1. Grading curve and mineralogical properties of Quiou sand

Table 1. Mineralogical properties of Quiou sand

Shell fragments	61·2%
Rock fragments and other	6·5%
Quartz	18·1%
Aggregates	14·2%

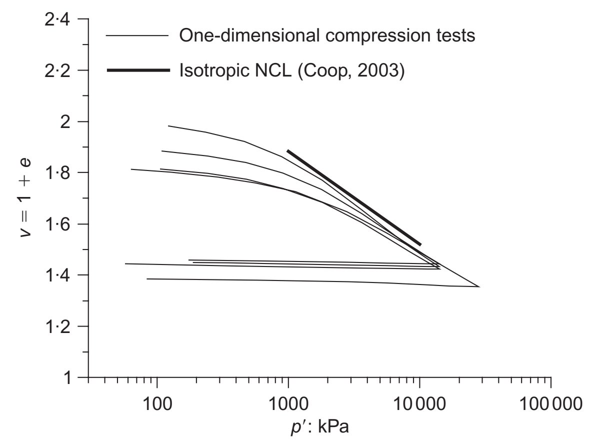

Fig. 2. Isotropic NCL and one-dimensional compression data for Quiou sand

top of the mould. In this way, density index values $I_r \equiv 35$–40% were initially achieved, regardless of the drop height. Higher densities were obtained by 'tapping' on the lateral surface of the mould while maintaining a small seating load on the sample cap and keeping drainage lines open (Caridi, 2006).

The undrained monotonic SS tests were carried out at different initial vertical effective consolidation stresses (σ'_{v0}) (Table 2). In order to investigate the effect of varying initial static shear stress (τ_{st}), cyclic SS tests were performed at three different values of the initial static shear stress ratio (α). It is defined as the ratio of the initial static shear stress (τ_{st}) on the horizontal plane to the initial vertical effective stress (σ'_{v0}). In particular, $\alpha = 0$ corresponds to symmetrical tests while $\alpha \neq 0$ corresponds to non-symmetrical tests. In the latter type of test, cyclic shear stresses (τ_{cyc}) were applied such that two different sets of test conditions were generated, namely

(*a*) no shear stress-reversal ('one-way' tests): $\tau_{st} \geqslant \tau_{cyc}$
(*b*) shear stress-reversal ('two-way' tests): $\tau_{st} < \tau_{cyc}$.

Symmetrical cyclic tests were carried out under two initial vertical effective stresses equal to 100 kPa and 200 kPa for both loose and dense specimens. Non-symmetrical tests were performed by initially applying a static shear stress up to the required target value before starting the cyclic phase.

A displacement rate of 0·1 mm/min was applied for monotonic shearing while a frequency of 0·02 Hz was maintained in cyclic shearing, The occurrence of a 3·75% maximum shear strain (γ_{max}) was assumed to define failure conditions for all tests in order to ensure a better control of undrained/constant volume test conditions. For this reason all tests were automatically stopped at the attainment of such condition.

In future laboratory research, it would be desirable to run tests at higher shear strain levels. A complete list of the tests performed in the present study is given in Table 2.

UNDRAINED MONOTONIC SIMPLE SHEAR BEHAVIOUR

The interpretation of results from undrained simple shear tests (monotonic and cyclic) was based on the approach proposed by Roscoe (1970) that the horizontal plane is the plane of maximum shear stress. This assumption can be considered reasonable in undrained tests whatever the initial void ratio, in agreement with other authors (i.e. Vaid & Sivathayalan, 1999).

Figure 3 shows the undrained monotonic SS behaviour

Table 2. Summary list of tests on tested sand

Test No.	Test type		e_0	I_r: %	σ'_{v0}: kN/m^2	$\alpha = \tau_{st}/\sigma'_{v0}$	CSR $= \tau_{cyct}/\sigma'_{v0}$	N_{cyc}
M1-L	Monotonic		1·007	40	50	—	—	—
M1-D	Monotonic		0·867	74	50	—	—	—
M2-L	Monotonic		0·986	45	100	—	—	—
M2-D	Monotonic		0·846	80	100	—	—	—
M3-L	Monotonic		0·993	43	150	—	—	—
M3-D	Monotonic		0·829	84	150	—	—	—
M4-L	Monotonic		0·984	46	200	—	—	—
M4-D	Monotonic		0·877	72	200	—	—	—
C1-L	Symm.		0·984	46	100	—	0·10	91
C2-L	Symm.		1·019	37	100	—	0·15	13
C3-L	Symm.		0·993	43	100	—	0·20	3
C4-L	Symm.		1·007	40	100	—	0·25	1
C1-D	Symm.		0·888	70	100	—	0·15	108
C2-D	Symm.		0·858	77	100	—	0·20	22
C3-D	Symm.		0·884	70	100	—	0·25	11
C4-D	Symm.		0·891	70	100	—	0·30	8
C5-L	Symm.		1·005	40	200	—	0·10	182
C6-L	Symm.		1·014	38	200	—	0·15	21
C7-L	Symm.		0·996	43	200	—	0·20	3
C8-L	Symm.		0·984	46	200	—	0·25	2
C5-D	Symm.		0·817	87	200	—	0·15	23
C6-D	Symm.		0·837	82	200	—	0·20	12
C7-D	Symm.		0·848	79	200	—	0·25	3
C8-D	Symm.		0·845	80	200	—	0·30	2
C9-L	Non-symm.	One-way	1·031	34	100	0·15	0·10	141
C10-L	Non-symm.	One-way	1·020	37	100	0·15	0·15	7
C11-L	Non-symm.	Two-way	1·028	35	100	0·15	0·20	2
C12-L	Non-symm.	Two-way	0·98	47	100	0·15	0·25	1
C93-D	Non-symm.	One-way	0·852	78	100	0·15	0·10	Failure did not occur
C10-D	Non-symm.	One-way	0·887	70	100	0·15	0·15	89
C11-D	Non-symm.	Two-way	0·861	76	100	0·15	0·20	23
C12-D	Non-symm.	Two-way	0·880	71	100	0·15	0·25	7
C13-L	Non-symm.	One-way	1·050	30	100	0·25	0·10	15
C14-L	Non-symm.	One-way	1·031	34	100	0·25	0·15	2
C15-L	Non-symm.	One-way	1·042	31	100	0·25	0·20	1
C16-L	Non-symm.	One-way	0·980	47	100	0·25	0·25	1
C13-D	Non-symm.	One-way	0·880	71	100	0·25	0·10	130
C14-D	Non-symm.	One-way	0·827	84	100	0·25	0·15	68
C15-D	Non-symm.	One-way	0·824	85	100	0·25	0·20	5
C16-D	Non-symm.	One-way	0·873	73	100	0·25	0·25	1

observed on loose and dense reconstituted specimens of QS. In particular the shear stress (τ) plotted against shear strain (γ) plots and the corresponding stress-paths in the τ plotted against σ'_v plane are depicted. In both cases, it is evident that specimens reached the phase transformation state (PT) (Ishihara *et al.*, 1975) at a very early stage of the tests and then went on to develop a strain-hardening type response. Even though the tests were conducted under different initial vertical stresses and void ratios, the stress-paths show a similar pattern of behaviour.

The best fit through the final points of the tests corresponds to a mobilised internal friction angle at the ultimate/steady state (ϕ'_{US}) equal to 41·4°, which is in quite good agreement with the value of critical state internal friction angle ($\phi'_{CS} = 40°$), determined from drained triaxial compression tests (Porcino *et al.*, 2005a). All the PT points fall into two straight lines (PT lines) that are practically coincident for both loose and dense specimens.

The strain-hardening behaviour of QS in monotonic loading gives rise to very high values of undrained shear strength. Owing to the large strain required to mobilise this strength, the strength at the 'yield' point (i.e. phase transformation) (τ_{PT}) is often a more realistic strength to adopt for design (Carter *et al.*, 2000).

UNDRAINED CYCLIC SIMPLE SHEAR BEHAVIOUR

Typical results from undrained cyclic tests under both symmetrical ($\tau_{st} = 0$) and non-symmetrical ($\tau_{st} \neq 0$) cyclic loading conditions are shown in Figs 4 to 6.

There are depicted

(*a*) the effective stress-paths followed during the tests
(*b*) the plots of applied shear stress ($\tau_{st} + \tau_{cyc}$) plotted against shear strain (γ)
(*c*) the trend with number of cycles (N_{cyc}) of cyclically induced shear strain (γ).

A comparative analysis of these figures clearly shows that, whatever the initial void ratio (e_0) and the initial vertical effective stress (σ'_{v0}) of the sand, the onset of failure oc-

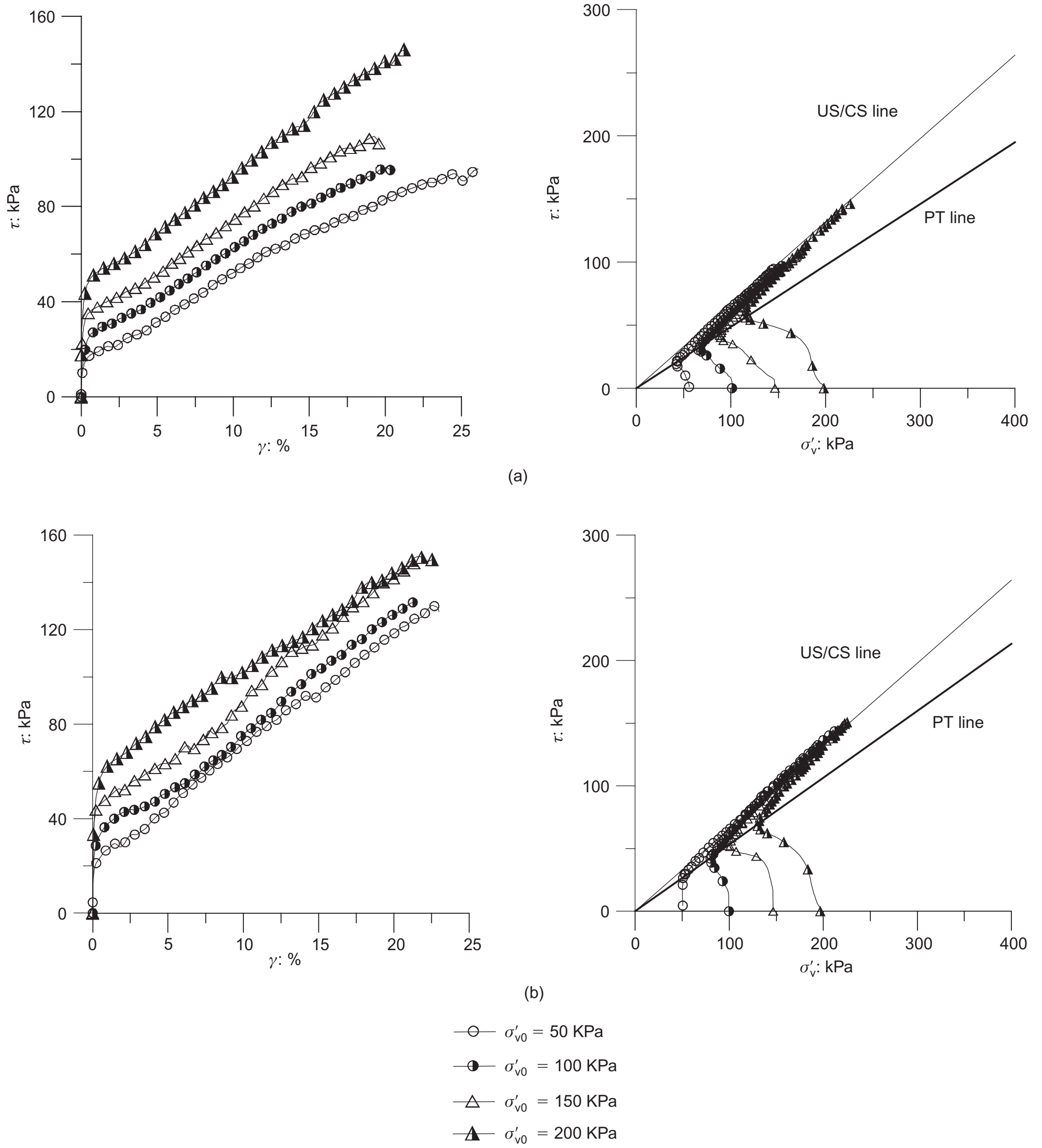

Fig. 3. Undrained monotonic response of Quiou sand: (a) loose specimens; (b) dense specimens

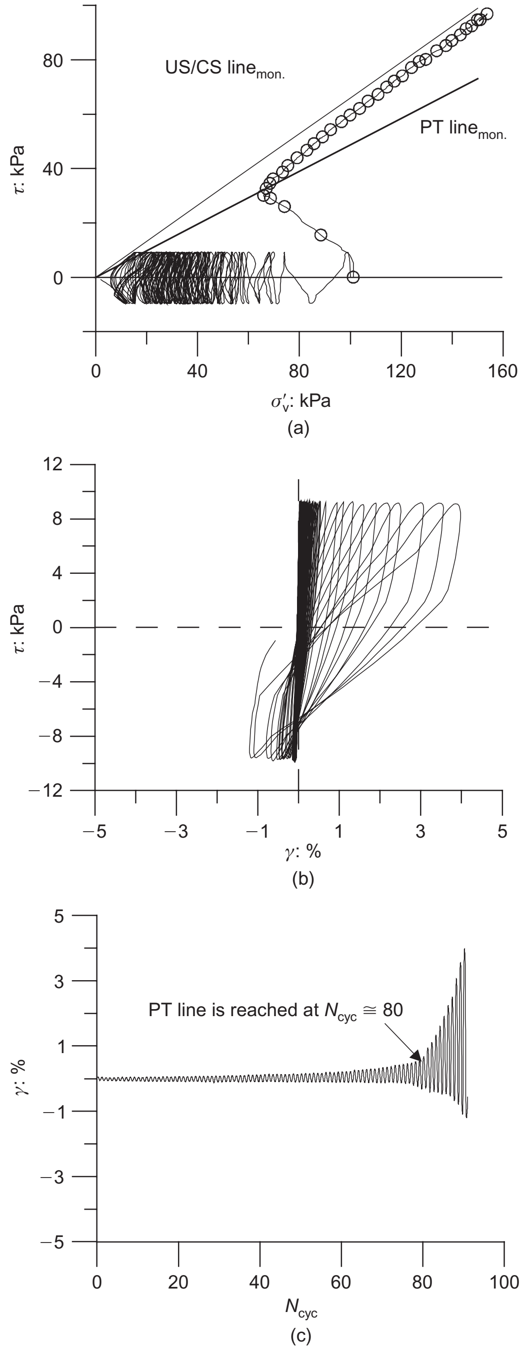

Fig. 4. Symmetrical cyclic SS test results (QS_Loose; $\sigma'_{v0} = 100$ kPa; $\alpha = 0$) (Test C1-L).

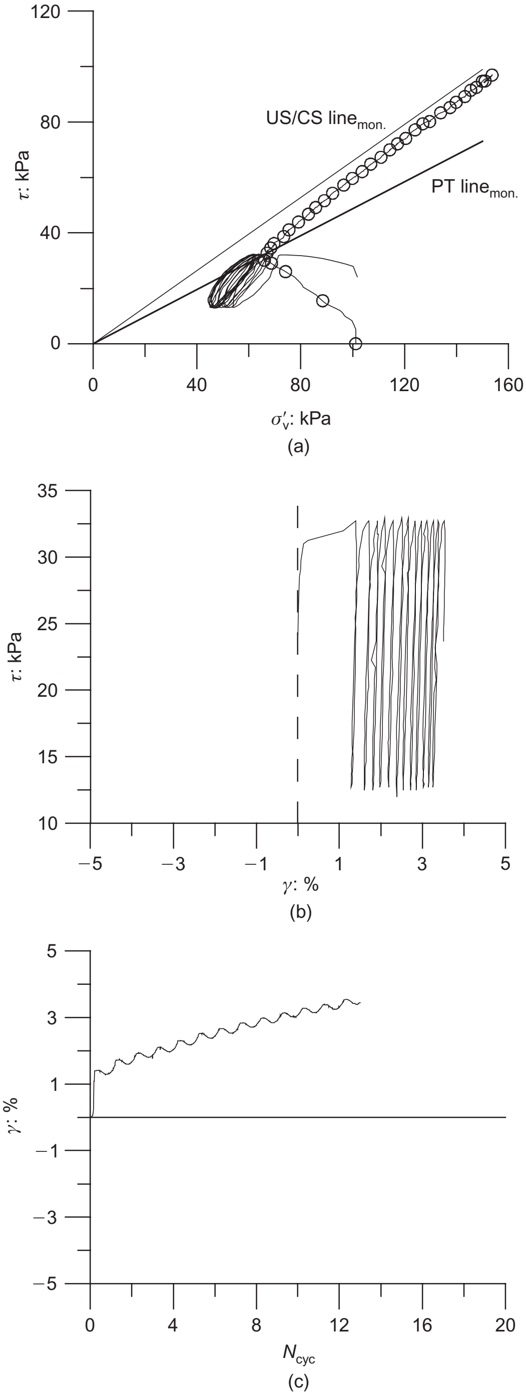

Fig. 5. Non-symmetrical cyclic SS test results (QS_Loose; $\sigma'_{v0} = 100$ kPa; $\alpha = 0{\cdot}25$) (Test C13-L)

curred by three distinct mechanisms that are described in the following.

In symmetrical tests a 'cyclic liquefaction' (Robertson and Wride, 1997) failure mode occurred (Fig. 4). Failure was caused by reduction in the cyclic stiffness, leading to excessive cyclic or 'swing' displacements (Fig. 4(b) and Fig. 4(c)). Shear strains accumulated slowly in the beginning; then, the rate of shear strain increment accelerated (Fig 4(c)) as it is also apparent in the S-shaped τ–γ curves in Fig. 4(b). At the onset of liquefaction ($\Delta u/\sigma'_{v0} = 0{\cdot}95$), induced cyclic shear strains reached values approximately equal to 3·75% in single amplitude.

A quite different behaviour is observed in non-symmetrical tests depending on whether the applied cyclic shear stress (τ_{cyc}) relative to initial static shear stress (τ_{st}), yields a shear stress reversal condition (Fig. 6) or not (Fig. 5). Upon non-reversal conditions ($\tau_{cyc} < \tau_{st}$) (Fig. 5), a 'cyclic mobility' failure mode (Robertson and Wride, 1997) occurred followed by stabilisation phenomena. Failure resulted from accumulation of excessive permanent or 'drift' strain (Fig. 5(b)), the cyclic stiffness remaining relatively unaffected.

Figure 5(a) shows clearly that closed loops exhibit an evident tendency to remain stable far from the critical state line. In tests with 'shear stress reversal' ($\tau_{cyc} > \tau_{st}$) (Fig. 6), an intermediate behaviour is observed in the sense that the cyclic softening and dilatant re-stiffening behaviour shown in Fig. 4 is followed by a progressive accumulation of shear strains in the direction of the driving shear force, similar to Fig. 5. According to Seed *et al.* (2003), this type of complex 'ratcheting' behaviour usually controls 'small to moderate' earthquake-induced deformations and displacements.

It is interesting to note (Figs 4 to 6) that when the

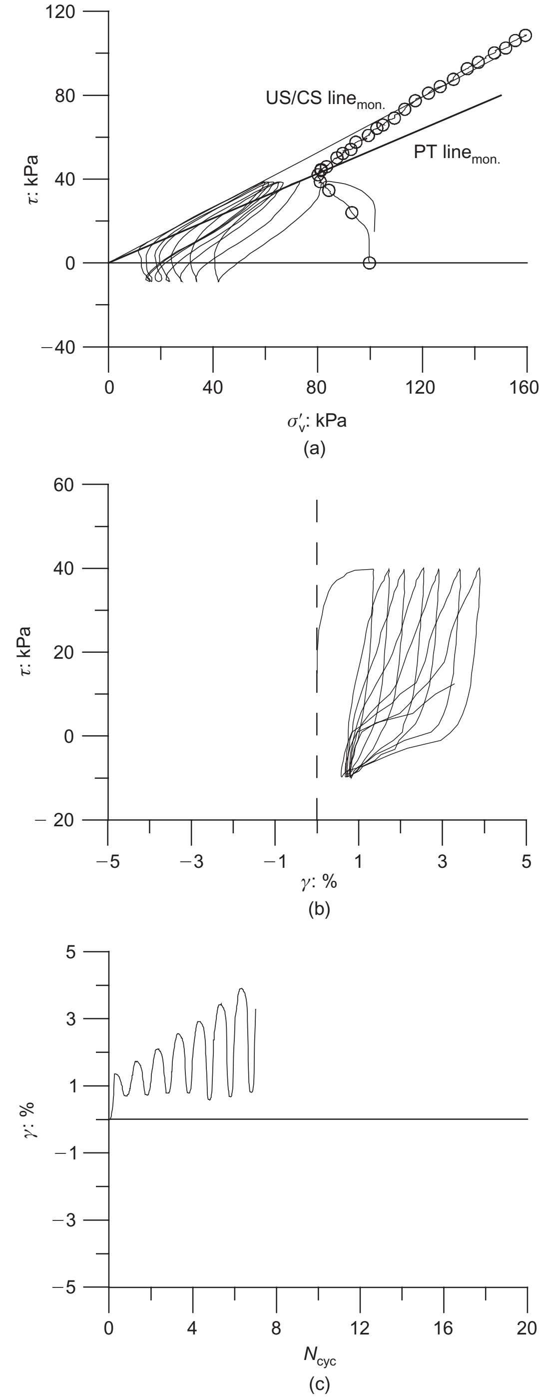

Fig. 6. Non-symmetrical cyclic SS test results (QS_Dense; $\sigma'_{v0} = 100$ kPa; $\alpha = 0{\cdot}15$) (Test C12-D)

effective stress-paths cross the phase transformation line obtained from monotonic tests, the specimens become 'unstable', that is they develop large shear strains and, correspondingly, an abrupt increase of pore water pressures.

CORRESPONDENCE BETWEEN CYCLIC AND MONOTONIC TESTS

The correspondence between cyclic and monotonic SS tests can be illustrated by examining the cyclic stress-paths shown in Figs 4(a), 5(a) and 6(a), in relation to the monotonic stress-paths and strength envelopes observed in the companion monotonic tests (Fig. 3).

A fairly good correspondence between the ultimate state lines from cyclic and monotonic undrained tests are observed in these figures. It is more evident in tests where a condition of pore water pressure ratio ($R_u = \Delta u / \sigma'_{v0}$) equal to 0·95 was reached at the final stage of the test.

Conversely, it appears that cyclic phase transformation (PT_{cyc}) lines, which separate 'contractive' from 'dilative' response in individual cycles, is different from, and located below, the PT lines obtained in monotonic tests.

In Fig. 7 PT_{cyc} lines have been drawn by linking 'elbows' of the stress-path loops of three tests characterised by different density index values and initial static shear stresses (in particular symmetrical and non-symmetrical cyclic loading). They appear to be poorly dependent on the initial static shear stress level and density index of the tested sand. In addition, in symmetrical tests a clear dependence on the shearing direction (positive or negative) does not appear.

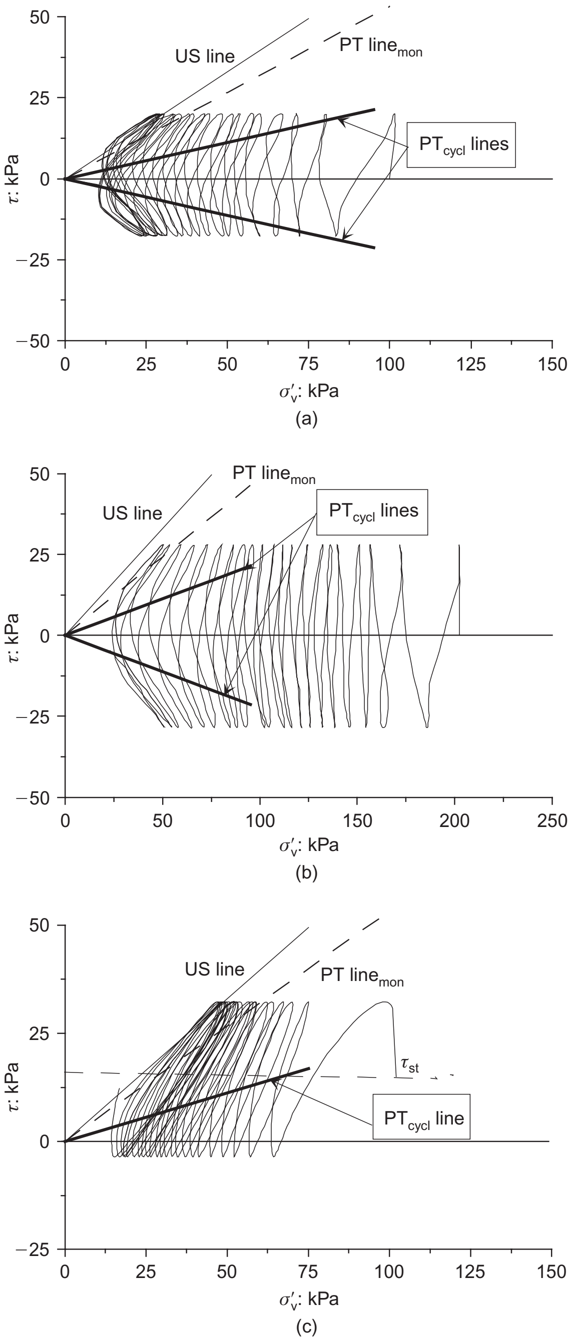

Fig. 7. Cyclic phase transformation lines for three undrained cyclic SS tests: (a) QS_Dense; $\sigma'_{v0} = 100$ kPa; $\alpha = 0$) (Test C-2D); (b) QS_Loose; $\sigma'_{v0} = 200$ kPa; $\alpha = 0$) (Test C6-L); (c) QS_Dense; $\sigma'_{v0} = 100$ kPa; $\alpha = 0{\cdot}15$) (Test C11-D)

Such experimental evidence is consistent with the findings gathered from previous investigations on different calcareous soils (Hyodo *et al.*, 1998; Mao & Fahey, 2003).

The tests presented in Fig. 7 refer to the cases where the applied shear stress ($\tau_{st} + \tau_{cyc}$) is less than the shear stress at the monotonic phase transformation state (τ_{PTmon}). The results of another set of tests where $\tau_{st} + \tau_{cyc} \geqslant \tau_{PTmon.}$ are reported in Fig. 8. As already shown in Figs 4 to 6, the role of the PT line on the failure characteristics of tested sand appears clearly. In particular, as the effective stress-path reaches the PT line determined in the companion monotonic tests, it seems to remain more or less stable close to the monotonic PT line.

Figures 9(a) and 9(b) show the stress–strain curves of the tests shown respectively in Figs 4 and 5, after normalisation of the shear stress by the actual value of σ'_v at all stages of the cyclic test. It can be seen that the back-bone curve, passing through the apices of each cycle, is practically coincident or just above the corresponding monotonic curve, superimposed in the same figures. According to Mao and Fahey (2003), rate effects could explain the small differences which are observed between the two types of curves.

UNDRAINED CYCLIC SHEAR STRENGTH

For all cyclic tests considered in this research, as previously mentioned, the occurrence of a 3·75% maximum shear strain was deemed to define failure conditions.

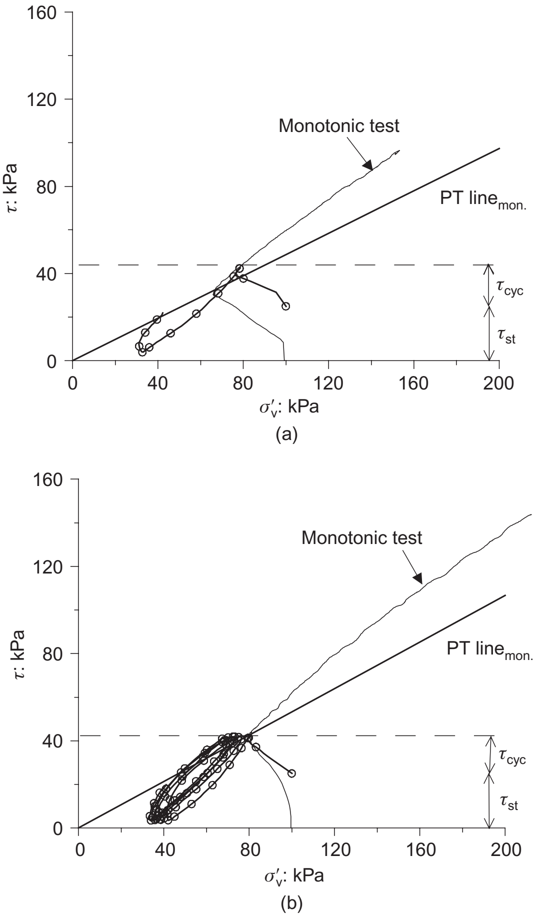

Fig. 8. Comparison between monotonic and cyclic effective stress-paths for non-symmetrical tests with τ_{st} + τ_{cyc} $\geqslant$ τ_{PTmon}: (a) (QS_Loose; $\sigma'_{v0} = 100$ kPa; $\alpha = 0{\cdot}25$) (Test C15-L); (b) (QS_Dense; $\sigma'_{v0} = 100$ kPa; $\alpha = 0{\cdot}25$) (Test C15-D)

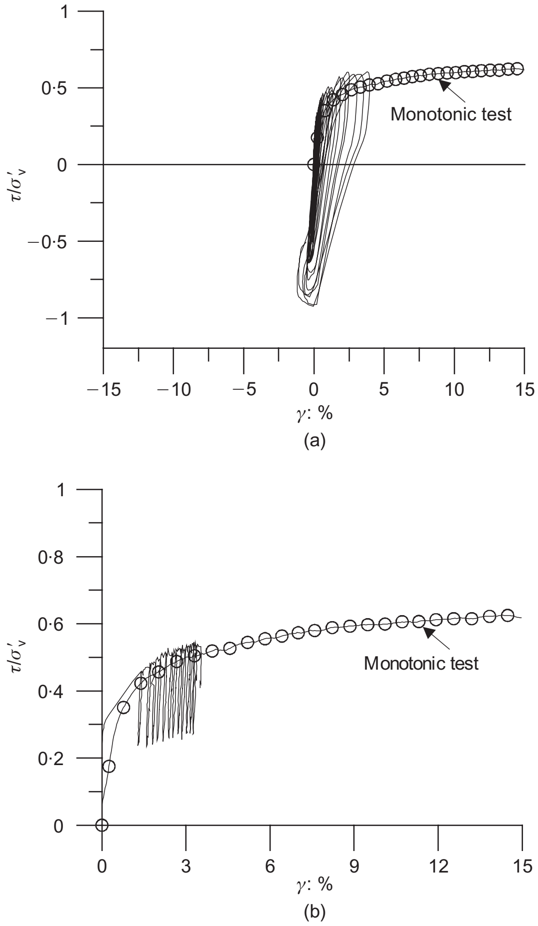

Fig. 9. Comparison between normalised stress–strain curves in monotonic and cyclic tests: (a) symmetrical tests (QS_Loose; $\sigma'_{v0} = 100$ kPa; $\alpha = 0$) (Test C1-L); (b) non-symmetrical tests (QS_Loose; $\sigma'_{v0} = 100$ kPa; $\alpha = 0{\cdot}25$) (Test C13-L).

Symmetrical cyclic tests

Cyclic liquefaction resistance curves obtained from symmetrical tests on both loose and dense specimens of QS are presented in Fig. 10(a). The curves are reported in terms of cyclic stress ratio (CSR $= \tau_{cyc}/\sigma'_{V0}$) against the number of cycles (N_{cyc}) required to reach a maximum shear strain of 3·75%. Results in Fig. 10(a) refer to specimens consolidated at two vertical effective stresses (σ'_{v0}) equal to 100 and 200 kPa.

For dense specimens there appears to be a dependence on the initial vertical effective stress with the cyclic strength tending to increase as the vertical stress decreases; conversely, for loose specimens, data points for σ'_{v0} equal to 100 kPa and 200 kPa are very close to one another.

Figure 10(b) demonstrates that when the cyclic strength ratio (τ_{cyc}/σ'_{v0}) is normalised by the monotonic phase transformation strength ratio (τ_{PT}/σ'_{v0}) in all tests (Fig. 10(b)), data points are less scattered. Accordingly, it is possible to seek a unique relationship regardless of effective initial vertical stress and density index of the tested sand. The results obtained confirm the validity of the approach proposed by other researchers in cyclic triaxial tests (Hyodo *et al.*, 1998) and simple shear tests (Fahey, 2001). For soils that tend to exhibit in monotonic shear a strain-hardening type behaviour, such approach would entail cyclic undrained shear strength of sands to be predicted from the phase transformation (PT) strength in the corresponding monotonic tests.

According to Fahey (2001), the best fit for cyclic failure

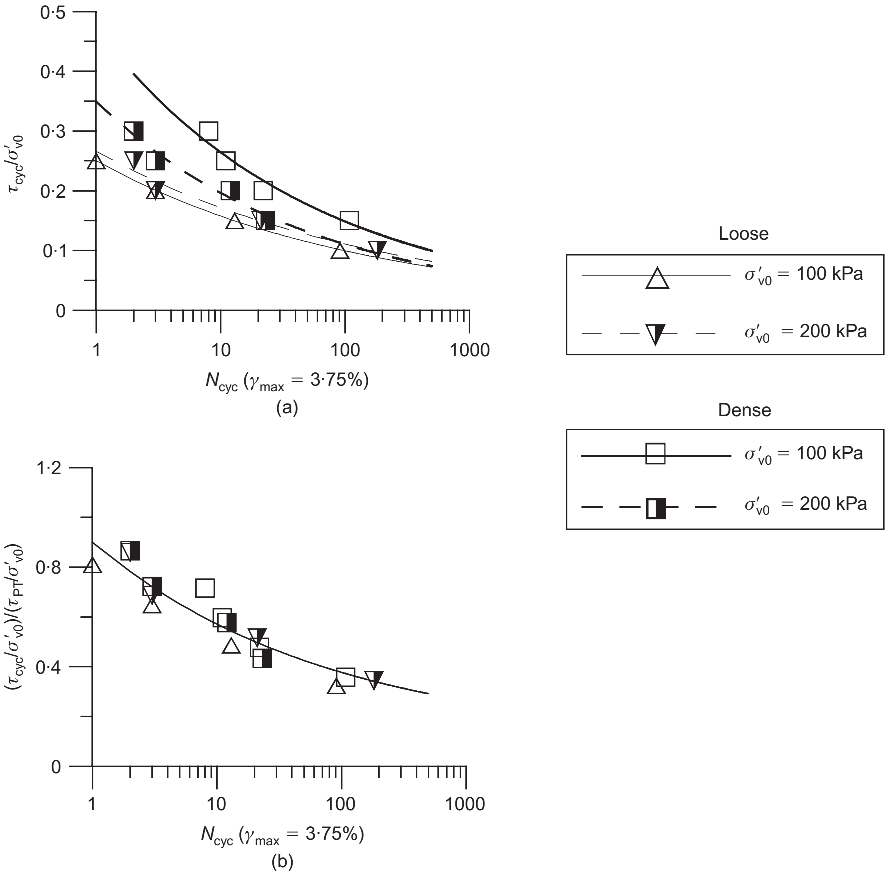

Fig. 10. Cyclic liquefaction resistance of Quiou sand from symmetrical cyclic SS tests: (a) cyclic liquefaction curves for different tests conditions; (b) cyclic stress ratio normalised to phase transformation strength

curve for a wide variety of calcareous soils can be expressed by a single equation:

$$\frac{(\tau_{cyc}/\sigma'_{v0})}{(\tau_{PT}/\sigma'_{v0})} = (b - c)(N_{cyc})^{-r} + c \tag{1}$$

where b, c and r are empirical parameters depending on type of material.

In particular 'b' represents the intercept of the curve on the vertical axis drawn for $N_{cyc} = 1$ and 'c' the cyclic threshold at very large number of cycles. Fig. 10(b) illustrates that the proposed normalised liquefaction resistance curve performs very well in fitting data points gathered from the present study, providing for QS the following parameters: $b = 0{\cdot}90$; $c = 0{\cdot}10$; $r = 0{\cdot}23$.

NON-SYMMETRICAL CYCLIC TESTS

Figure 11 shows the undrained cyclic strength curves obtained from symmetrical ($\alpha = 0$) and non-symmetrical tests ($\alpha = \tau_{st}/\sigma'_{v0} > 0$) on both loose and dense sand specimens.

Results obtained from tests carried out with different initial static shear stress values (τ_{st}) indicate that, as the level of τ_{st} increases, the tested sand is more vulnerable to failure.

The influence of an initial static shear stress on cyclic strength can be accounted for by introducing a static shear stress ratio correction factor (K_α which represents the relative variation in cyclic strength owing to the presence of non-zero driving shear stresses. This factor is defined as (Seed *et al.*, 2003)

$$K_\alpha = \frac{\mathrm{CSR}_{\text{failure}, \alpha>0}}{\mathrm{CSR}_{\text{failure}, \alpha=0}} \tag{2}$$

CSR being the equivalent uniform cyclic shear stress ratio required to 'trigger' failure in a prefixed number of cycles. Fig. 12 presents data of cyclic simple shear tests reported by Idriss & Boulanger (2003) where correction factor K_α is plotted against α for different density states. It should be noted that Fig. 12 is appropriate only for silica sands at an initial vertical stress lower than or equal to 200 kPa ($\sigma'_{v0} \leqslant$ 200 kPa). In the same figure experimental results gathered from the present research are also superimposed.

Figure 12 shows that measured K_α values are never greater than one in the tested sand indicating that the presence of an initial static shear stress generally causes a loss of strength that can be quite severe as α approaches the value of 0·2. Furthermore, it is apparent that for loose specimens the curves relative to carbonate sands follow the same trend as that observed for silica sands, whereas significant differences are observed for dense specimens. Such differences in behaviour are consistent with those observed by Kaggwa (1988) in undrained cyclic triaxial tests under anisotropic consolidation condition. The observed pattern of behaviour is believed to be attributable to the higher compressibility and the greater tendency for particle crushing of the carbonate materials compared to silica sands.

CONCLUSIONS

The undrained behaviour of an uncemented carbonate sand (Quiou) was investigated through monotonic and cyclic simple shear tests. In cyclic tests reconstituted specimens were prepared at two initial density index values (loose and

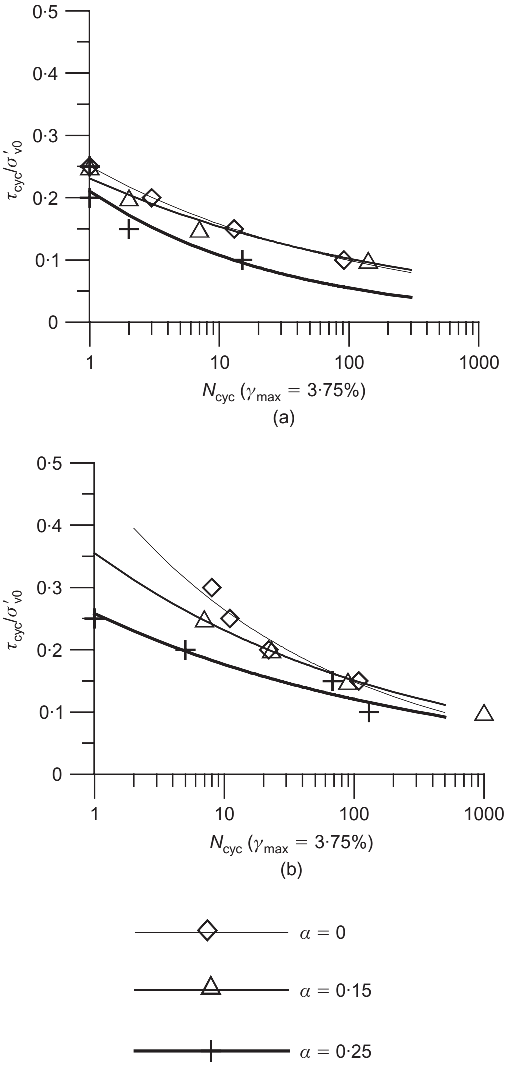

Fig. 11. Influence of the initial static shear stress on undrained cyclic shear strength curves of Quiou sand for: (a) loose specimens $\sigma'_{v0} = 100$ kPa and (b) dense specimens $\sigma'_{v0} = 100$ kPa

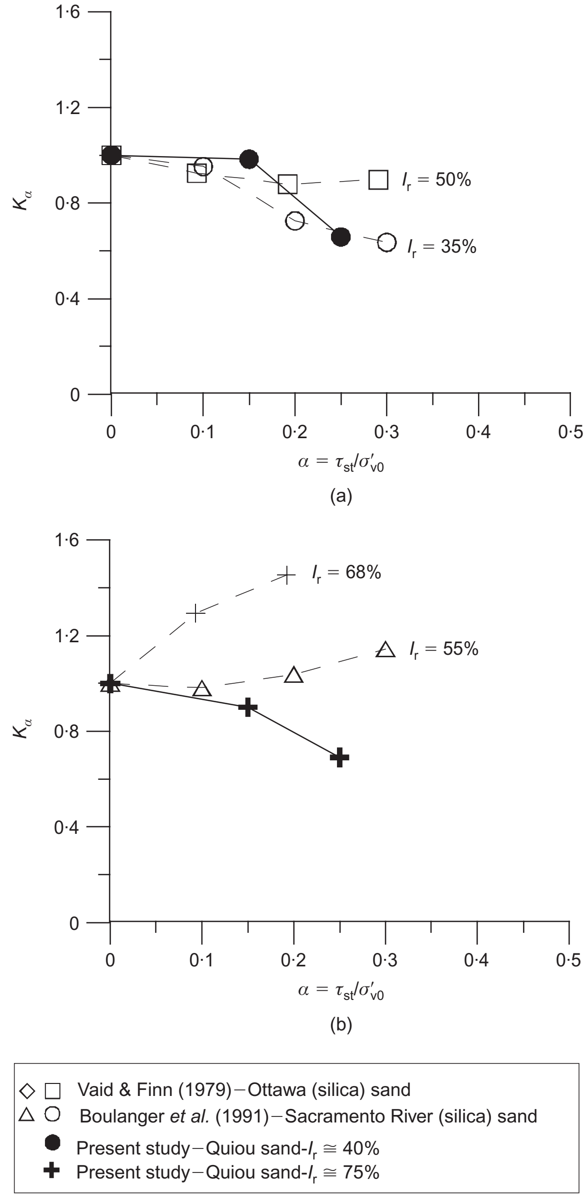

Fig. 12. Correction factor for initial static shear stress (K_α) plotted against initial static stress ratio (α): comparison with data from silica sands (Idriss & Boulanger, 2003)

dense) and consolidated under two different vertical effective stresses. In order to take into account the effect of a non-zero mean shear stress, cyclic simple shear tests were run under both symmetrical and non-symmetrical loading conditions.

The main conclusions which can be drawn from the investigation may be summarised as follows.

(*a*) In monotonic tests a strain-hardening type response is observed, whatever the initial void ratio and effective stress state of the tested sand. The ultimate strength envelopes are practically coincident with the critical state envelope determined in drained triaxial tests (Porcino *et al.* 2005b). The phase transformation line, linking the elbows of the effective stress paths, is practically unique regardless of initial void ratio and consolidation stress.

(*b*) In cyclic tests, two different failure modes occur: under symmetrical loading conditions, failure occurs by reduction in the cyclic stiffness leading to excessive cyclic 'swing' strains. Highly non-symmetrical loading conditions lead to failure by accumulation of permanent 'drift' strains, the cyclic stiffness remaining relatively unaffected.

(*c*) Based on symmetrical cyclic tests, normalisation of the cyclic strength with respect to the phase transformation strength from the corresponding monotonic tests provides an effective means of unifying the undrained response, irrespective of density index and initial vertical stress conditions.

(*d*) The comparison between monotonic and cyclic tests was conducted in terms of normalised undrained stress–strain behaviour. It was evidenced that the back-bone curves, passing through the apices relative to the individual cycles of the normalised stress–strain curves, match the monotonic curves in a satisfactory way. It was also found that a clear phase transformation (PT_{cyc}) line exists in any cyclic test but it was shown as being separate from the PT line obtained in the corresponding monotonic test. In addition, it was recognised that a unique undrained ultimate failure envelope ($\phi'_{CS/US} \cong 40°$) exists for both cyclic and monotonic tests.

(*e*) Cyclic undrained simple shear strength is strongly affected by the presence of a mean shear stress. In particular, whatever the initial void ratio, QS specimens are more susceptible to 'triggering' of failure conditions when tested under an initial static shear stress. Such behaviour is similar to that observed on loose silica sands, all other conditions being the same.

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