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Sand–tyre chips mixtures in undrained and drained cyclic triaxial tests

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This paper deals with the stress-controlled cyclic triaxial response of saturated sand rubber chips (RC) mixtures at different gravimetric contents. Three types of sand were utilised, namely, uniform medium and coarse sands, as well as a combination of the two at two relative densities. A series of consolidated undrained and drained single-stage cyclic triaxial tests was performed using standard equipment to evaluate the development of the accumulated excess pore-water pressures, permanent axial and volumetric strains. Samples were anisotropically consolidated prior to cyclic loading. The results showed that for all sands adding 10% RC increases the liquefaction potential. Compared to pure sand, increasing the chips content beyond 20–30% generally yields an improvement as long as the sand is not too coarse. Expressions have been derived for the evolution of accumulated pore-water pressure and the relation between excess pore-water pressure from the undrained test and cumulative volumetric strain from drained tests. Although the findings were derived for a single set of loads, they reveal the basic characteristics of the cyclic composite material behaviour.

Notation

C_c	coefficient of curvature
C_u	uniformity coefficient
c'	cohesion intercept
d_{50}	mean grain size
e	void ratio
$e_{\max}$	maximum void ratio
$e_{\min}$	minimum void ratio
G_s	specific gravity
I_D	relative density
N	number of cycles
N_f	number of cycles to initial liquefaction
r_u	pore-pressure ratio
$\bar{r}_u$	average pore-pressure ratio
t	time
Δu	excess pore-water pressure
$\varepsilon_{1,cy}$	cyclic axial strain
$\varepsilon_{v,cy}$	cyclic volumetric strain
ρ_d	dry density
$\rho_{d,\min}$	minimum dry density
$\rho_{d,\max}$	maximum dry density
σ'_{3c}	effective confining stress
ϕ'	internal angle of friction

1. Introduction

Ground motion caused by shaking due to earthquakes or other vibrations induces settlement of dry, unsaturated or saturated granular soils. Furthermore, in saturated sandy soils subjected to vibrations, excess pore-water pressure is generated that may cause liquefaction. The increase of excess pore-water pressure is always associated with a severe loss of strength and stiffness and may lead to collapse. Various ground improvement techniques

hitherto have been employed to strengthen sandy soils and mitigate their liquefaction potential. A newly developed concept for stabilising soils prone to liquefaction is reinforcement using shredded tyre material in the form of shreds/chips (see e.g. Towhata, 2008; Hong *et al.*, 2015). The literature on the application of this method contains a bank of data. One may mention among others the work by Humphrey and Sandford (1993), Edil and Bosscher (1994), Foose *et al.* (1996), Masad *et al.* (1996), Wu *et al.* (1997), Zornberg *et al.* (2004), Ghazavi (2004), Becker and Vrettos (2011), Hazarika *et al.* (2012), Senetakis *et al.* (2012), Noorzad and Raveshi (2017), Madhusudhan *et al.* (2019) and Banzibaganye *et al.* (2019). A gravimetric content in the range of 20–40% by dry weight of sand–tyre chips mixture was generally reported as an optimum to reinforce sandy soils with respect to strength enhancement. Increase of resistance to liquefaction was addressed in a few studies focusing on the assessment of excess pore-water pressure during cyclic loading (cf. Okamoto *et al.*, 2007; Promputthangkoon and Hyde, 2007; Kaneko *et al.*, 2013; Mashiri, 2014; Hong *et al.*, 2015; Li *et al.*, 2016; Bahadori and Farzalizadeh, 2018; Shariatmadari *et al.*, 2018; Zhou and Wang, 2019). The soil used in these studies was mainly a poorly graded sand. Indraratna *et al.* (2018) and Qi *et al.* (2018) investigated cyclic loading of granular material composed of pulverized coal-wash and steel furnace slag mixed with rubber crumbs.

The results from these studies related to sands show a certain contradiction. The findings by Mashiri (2014) concluded that increasing the content of tyre chips up to 30% in the mixtures reduces liquefaction potential, although an increase was noticed for an inclusion of 40%. Mashiri asserts that in a mixture with this content, the sand matrix void ratio becomes greater than the maximum void ratio of the sand, and this

leads to an increased residual pore-water pressure. On the contrary, Hong *et al.* (2015) and Shariatmadari *et al.* (2018) observed the opposite behaviour. Their findings indicate that increasing tyre chips and ground rubber in sand decreases both shear strength and the resistance to liquefaction. The cause could be sample conditions and loading characteristics, or the small particle size of the ground rubber used in the tests.

Starting from the well-known empirical relationship by Seed and Booker (1977), several other equations were developed to express the average pore-water pressure ratio of sand as a function of the normalised cycle number, among others by Egglezos and Bouckovalas (1998). A recent study on the evolution of excess pore-water pressure and volumetric strain during undrained and drained strain-controlled cyclic triaxial tests of sand, respectively, is documented in Chen *et al.* (2019). An equation relating pore-water pressure ratio and volumetric strain was proposed. The rebound modulus of the pore-pressure ratio was also obtained.

The contradictory findings of previous studies with respect to liquefaction behaviour of sand-rubber chips (RC) mixture with various chips contents can be attributed to the type of sand, the relative density of the mixtures and the loading pattern applied. To better understand the impact of tyre chips on the liquefaction resistance, a series of stress-controlled cyclic triaxial tests were performed on three types of sand – that is, uniform medium sand (S2), coarse sand (S3) and a combination of the two. Two relative densities corresponding to loose and medium compaction were used. The loading stresses imposed on all specimens were similar. The evolution of excess pore-water pressure under undrained conditions is compared to the accumulated cyclic volumetric strain under drained conditions. A functional relationship between these quantities is established dependent on the chips content and the density.

2. Material and testing procedure

2.1 Material

Three types of clean sand S2, S3 and S4 were used in this study, with particle-size distribution as shown in Figure 1. They are classified as uniform medium sand, uniform coarse sand, and poorly graded sand, respectively. Properties are summarised in Table 1 and include specific gravity G_s , mean grain size d_{50} , uniformity coefficient C_u , coefficient of curvature C_c , minimum and maximum values of dry density and void ratio, $\rho_{d,min}$, $\rho_{d,max}$ and e_{min} and e_{max} , respectively. Furthermore, standard triaxial compression tests were conducted at specific values of initial void ratio e_0 . The shear strength parameters at peak – that is, cohesion intercept c' and internal angle of friction ϕ' – are given in Table 2.

Shredded tyre materials were collected from a local tyre recycling company, and were free of steel, wires and fibres. After sieving to a desired size ranging between 4 and 12 mm the

Figure 1. Particle-size distribution for the different sands and RC. S2: uniform medium sand; S3: uniform coarse sand; S4: poorly graded sand mixture

Table 1. Properties of the sands used

Description	S2	S3	S4
G_s : g/cm ³	2.65	2.65	2.65
d_{50} : mm	0.32	0.77	0.5
C_u	1.7	1.5	2.7
C_c	1	0.9	1
$\rho_{d,max}$: g/cm ³	1.591	1.663	1.798
$\rho_{d,min}$: g/cm ³	1.388	1.466	1.466
e_{min}	0.665	0.594	0.807
e_{max}	0.909	0.807	0.474

Table 2. Shear strength parameters for the sands from triaxial tests

Description	S2	S3	S4
e_0	0.748	0.666	0.588
c' : kPa	9.5	8.7	10.3
ϕ' : degrees	37.5	35.4	37.0

Figure 2. RC mixed with sand S2

grain-size distribution curve depicted in Figure 1 was produced. A photograph of the RC mixed with sand is shown in Figure 2. This material is indicated as RC in this study. The density and specific gravity were 0.64 g/cm^3 and 1.05, respectively. These values are close to those reported by previous studies – for example, Masad *et al.* (1996) and Wu *et al.* (1997). At this density, the standard triaxial test yielded at peak a cohesion $c' = 23.3 \text{ kPa}$ and an internal angle of friction $\phi' = 13.2^\circ$. Shear strength parameters were determined using a large direct shear box ($30 \times 30 \text{ cm}$) where $c' = 7.9 \text{ kPa}$ and $\phi' = 25.2^\circ$.

2.2 Equipment

A dynamic cyclic triaxial testing system was used to conduct the cyclic triaxial tests. This system with the commercial name ELDYN was manufactured and supplied by GDS. It comprises a velocity-controlled load frame capable of applying a frequency up to 10 Hz and an axial load of up to 10 kN through a servo-controlled actuator. A static load at a constant rate or cyclic load can be applied to the specimen through axial movement. The axial load applied to the specimen is measured by an internal submersible load cell. The triaxial cell can accommodate cell pressures up to 2 MPa. The drainage ports are on the top cap and the base pedestal. Pore pressure in the sample is measured at the top. The confining stress level is applied by a pneumatic cell pressure controller. For saturation, a 2 MPa back-pressure/volume controller is used. This device also measures the change in volume of the sample during saturation, consolidation, static triaxial compression and subsequent cyclic loading stages. The induced axial deformation is measured by a high-accuracy external linear displacement transducer (strain gauge). The entire system is controlled by a GDS dynamic controller, which provides a four-channel dynamic data logger with 16 bits from which various transducers are connected.

2.3 Sample preparation, saturation and testing programme

Samples 100 mm in diameter and 150 mm high were prepared following standard techniques, as described by Banzibaganye *et al.* (2019). Samples were prepared for sands S2, S3 and S4 at RC contents of 0, 10, 20 and 30% by dry mass under wet conditions to prevent segregation of particles. Higher values of RC content were not considered as they yield higher compressibility of the composite material, which restricts application in practice. The total target mass of each specimen was divided into five equal portions, each transferred in a sample preparation mould and compacted until a thickness of 30 mm was achieved. Two values of relative density – that is, 30 and 50% – were selected for this study. Figure 3 shows a sample of S2 mixed with 30% RC before the start of the test. The properties of the tested samples are summarised in Table 3, where ρ_d is the dry density, e is the void ratio of the composite material and I_D is the relative density. The table also contains the test type – that is, drained (D) or undrained (U), as well as results

Figure 3. Sample of sand S2 at 30% RC content before start of the test

in terms of cycles to initial liquefaction N_f , which are discussed in the next sections.

To obtain the desired degree of saturation for the saturated samples prior to testing, the procedure described in ASTM D 5311-92 (ASTM, 2004) using the back-pressure technique was followed. The obtained B values were above 0.95 at back-pressure levels of around 800 kPa for pure sand and around 200 kPa for the sand–RC mixtures. All specimens were consolidated under an effective confining stress of 100 kPa. Stress-controlled testing was conducted according to ASTM D 5311-92 (ASTM, 2004).

A pre-cyclic static deviator stress of 45 kPa was imposed on all specimens followed by a cyclic deviator stress of amplitude 50 kPa at a frequency of 1 Hz. The latter value lies within the frequency range recommended for cyclic triaxial testing by ASTM D 5311-92 (ASTM, 2004). The anisotropic pre-cyclic loading aimed at better simulating field conditions underneath loaded structures or railway tracks.

Two types of stress-controlled cyclic triaxial test were conducted: (a) consolidated undrained single-stage dynamic cyclic triaxial tests to evaluate the development of excess pore-water pressure and (b) consolidated drained single-stage dynamic cyclic triaxial tests with measurement of volume change evolution with cycle number.

2.4 Failure criteria

Failure in saturated sand due to cyclic loading is defined in terms of the pore-water pressure Δu or displacements. For the

Table 3. Sample properties, test conditions and cycles to liquefaction

RC: %	ρ_d : g/cm ³	e	I_D	Soil type	N_f	Test type	Test no.
0	1.442	0.838	0.3	S2	24	U	CY01
	1.442	0.838	0.3		No	D	CY02
	1.484	0.785	0.5		43	U	CY03
	1.484	0.785	0.5		No	D	CY04
10	1.351	0.843	0.3		14	U	CY05
	1.351	0.843	0.3		20	D	CY06
	1.393	0.787	0.5		30	U	CY07
	1.393	0.787	0.5		30	D	CY08
20	1.246	0.870	0.3		22	U	CY09
	1.246	0.870	0.3		35	D	CY10
	1.287	0.809	0.5		24	U	CY11
	1.287	0.809	0.5		34	D	CY12
30	1.132	0.917	0.3		32	U	CY13
	1.132	0.917	0.3		40	D	CY14
	1.171	0.853	0.5		40	U	CY15
	1.171	0.853	0.5		No	D	CY16
0	1.520	0.744	0.3	S3	No	U	CY17
	1.520	0.744	0.3		No	D	CY18
	1.558	0.701	0.5		No	U	CY19
	1.558	0.701	0.5		No	D	CY20
10	1.328	0.875	0.3		66	U	CY21
	1.328	0.875	0.3		No	D	CY22
	1.365	0.823	0.5		178	U	CY23
	1.365	0.823	0.5		No	D	CY24
20	1.188	0.961	0.3		202	U	CY25
	1.188	0.961	0.3		No	D	CY26
	1.228	0.897	0.5		321	U	CY27
	1.228	0.897	0.5		No	D	CY28
30	1.070	1.029	0.3		272	U	CY29
	1.070	1.029	0.3		No	D	CY30
	1.108	0.958	0.5		No	U	CY31
	1.108	0.958	0.5		No	D	CY32
0	1.553	0.707	0.3	S4	7	U	CY33
	1.553	0.707	0.3		25	D	CY34
	1.615	0.641	0.5		26	U	CY35
	1.615	0.641	0.5		No	D	CY36
10	1.388	0.795	0.3		4	U	CY37
	1.388	0.795	0.3		7	D	CY38
	1.436	0.734	0.5		5	U	CY39
	1.436	0.734	0.5		10	D	CY40
20	1.274	0.829	0.3		20	U	CY41
	1.274	0.829	0.3		34	D	CY42
	1.307	0.782	0.5		23	U	CY43
	1.307	0.782	0.5		30	D	CY44
30	1.155	0.879	0.3		39	U	CY45
	1.155	0.879	0.3		No	D	CY46
	1.189	0.824	0.5		44	U	CY47
	1.189	0.824	0.5		No	D	CY48

former, the pore-water pressure ratio r_u is introduced by relating Δu to the minor effective consolidation stress σ'_{3c}

$$1. \quad r_u = \frac{\Delta u}{\sigma'_{3c}}$$

and liquefaction is occurring when r_u attains a value of 1 (initial liquefaction). For the second criterion (not discussed herein) a threshold value of the double amplitude axial strain is used – for example, 5% or more (cf. El Mohtar, 2009).

As mentioned above, the cyclic tests reported herein were conducted under anisotropic pre-cyclic stress conditions. The effects of the pre-cyclic static shear stress have been investigated, among others, by Vaid and Chern (1983), Hyodo *et al.* (2002) and Vaid *et al.* (2001). The soil resistance depends on relative density and confining stress level, and becomes significantly weaker when loading conditions lead to a stress reversal. In the set-up selected herein, the combined shear stress was always positive – that is, no reversal conditions prevailed. In that case, a threshold value for the accumulated strain – for

example, 5% – may be used to indicate the onset of failure (Hyodo *et al.*, 2002).

Due to the compressibility of RC material, the plastic axial strain developed in sand–RC mixtures may be higher than that exhibited by pure sand before the occurrence of initial liquefaction. The increase of material ductility induced by the increase of RC content plays an essential role in cyclic strength mobilisation, even after the occurrence of initial liquefaction.

3. Results and discussion

The results in this study are presented in terms of cycle-dependent excess pore-water pressure and axial strain for the undrained tests and cycle-dependent volumetric strain from the drained tests. Figures 4–12 display selected results. It should be pointed out that in some of the drained tests on the composite material, liquefaction occurred after a sufficiently large number of cycles despite the open drainage line (cf. Table 3).

Figure 4. Evolution with time of the cyclic axial strain in undrained tests for S2 at (a) 0% and (b) 10% RC and for $I_D = 0.3$

Figure 5. Comparison of the cyclic axial strain in undrained tests for S2 and sand mixture S4 at 30% RC content; $I_D = 0.3$

For ease of discussing the effect of RC, the computed average data are used. They are determined by averaging minimum and maximum values within each cycle. Consequently, the value of the average pore pressure $\bar{r}_u$ remains slightly below 1 at initial liquefaction.

3.1 Results for cyclic axial deformation in undrained tests

Figure 4 clearly presents time histories of axial strain recorded during undrained cyclic loading for various mixtures of S2 and RC (material S2RC). It can be seen that the sample containing 10% RC in test CY5 exhibited higher deformation compared to that of pure sand in test CY1. It was observed that further increase of RC content increases the resistance to axial deformation of the saturated composite material. The results for the other mixtures formed from sand S3 and S4 (material S3RC and S4RC), not shown here, follow the same trend. The cyclic settlement of sand–RC mixtures is also affected by the relative density. The test data from various mixtures revealed that the settlement from the mixtures under $I_D = 0.5$ is lower than that of the mixtures under $I_D = 0.3$, as expected.

For the sand mixture S4 prepared without any RC the occurrence of initial liquefaction was immediately followed by an abrupt failure. The addition of RC renders the composite material more ductile, in particular at RC content larger than 20%.

Comparison of the cyclic deformation for S2 and sand mixture S4 showed that pure sand mixture S4 (test CY33) and S4 at 10% chips content (test CY37), not shown here, exhibited earlier an abrupt failure with higher deformation compared to the case of S2 (test CY1). As mentioned above, adding a small amount of RC in these sands (up to ~10%) increases the permanent axial deformation.

A marginal increase was observed for sand mixture S4 at 10% chips (CY37), while it was extreme for S2 (CY5). A drastic reduction of axial strain was observed at 20% and higher chips content for sand mixture S4. It can be observed from Figure 5 that the deformation of these sands at 30% RC follows the same lines. This behaviour shows that at higher RC content, sands S2 and S4 may exhibit almost similar cyclic response. The same was observed for sand–tyre chips mixtures at $I_D = 0.5$.

The observed behaviour at 10% RC may be attributed to the decreased density and insufficient ductility. At higher chips contents the strains were lower despite the lower density that was compensated by an increased ductility, interlocking of chips and sand particles, reinforcement due to increase of shear strength as well as damping effects. Similar behaviour is reported by Bahadori and Farzalizadeh (2018).

3.2 Results for excess pore-water pressure in undrained tests

For the validation of the testing procedure, the results for the excess pore-water pressure are compared with the predictive equations for fine sand developed by Egglezos and Bouckovalas (1998) from a large number of cyclic triaxial tests. The evolution of pore-water pressure with number of cycles is expressed by means of a power-law function:

$$2. \quad \Delta u(N) = \Delta u(1)N^c$$

where $\Delta u(1)$ denotes the value after the first loading cycle. That equation has been derived for fine sand with uniformity coefficient C_u of approximately 1.5, and the exponent c as determined by curve fitting is equal to 0.48 for cohesionless soil. The comparison is made herein for S2, which is coarser but exhibits a similar uniformity coefficient. In Equation 2, the cyclic stress amplitude, the confining stress and the void ratio are incorporated in $\Delta u(1)$, making this value the decisive parameter for the quality of the prediction. Figure 6 compares the test data and the prediction showing a good agreement at both relative densities. The other sands tested (S3 and S4) were either much coarser or less uniform, and deviated from the validity range of Equation 2.

Figure 7 displays typical time histories of the development of excess pore-water pressure for pure sand S2 and sand S3. Figure 8 shows the corresponding curves for S2 at 20 and 30% RC content.

For samples prepared at similar I_D , test CY1 (pure S2) yielded liquefaction, while in test CY17 (pure S3) liquefaction did not occur. Test CY33 (pure S4) specimen prepared at $I_D = 0.3$, not presented herein, exhibited abrupt failure already after a few cycles. Adding 10% RC to sand increases the accumulated excess pore-water pressure compared to pure

Figure 6. Comparison of undrained test data for sand S2 with predictive Equation 2 at two densities

Figure 7. Evolution with time of the excess pore-water pressure in undrained tests for (a) sand S2, (b) sand S3 at $I_D = 0.3$

sand. Further increase of RC decreases excess pore-water pressure. This can be seen in the data presented in Figure 8 for S2 RC mixtures.

Figure 8. Evolution with time of the excess pore-water pressure in undrained tests for S2 with (a) 20% and (b) 30% RC content at $I_D = 0.3$

Figure 9. Average pore-water pressure ratio $\bar{r}_u$ against number of cycles in undrained tests for S2 at various RC contents for (a) $I_D = 0.3$ and (b) $I_D = 0.5$

Twenty-five cycles were required to cause initial liquefaction for test CY1 (pure S2 at $I_D = 0.3$), while the mixture with 30% chips content (test CY13) necessitated 32 cycles. The results for material compacted at $I_D = 0.5$, not presented here, showed that pure sand was stronger against liquefaction than all other types of mixture. Initial liquefaction for pure sand (test CY3) occurred at 43 cycles, while the sample with 30% chips content (test CY15) liquefied after 40 cycles (cf. Table 3).

A comparison for S2 and S4 at various chips contents, not shown here, showed that pure sand mixture S4 (test CY33) and S4 at 10% chips content (test CY37) exhibited rapid development of excess pore-water pressure compared to S2 (tests CY1 and CY5). The rate of excess pore-water pressure generation decreases remarkably from a gravimetric content of 20% and higher for sand mixture S4, and was moderate for sand S2. Despite the rate of excess pore-water pressure development in these two sands, excess pore-water pressure becomes closely similar at 30% RC.

A direct comparison among tests for S2 is visualised in Figure 9, which displays the computed average pore-pressure ratio $\bar{r}_u$ against the number of cycles. Bear in mind that, as mentioned above, $\bar{r}_u$ is always smaller than r_u , and its highest value remains slightly below 1 at initial liquefaction. A similar trend, not presented herein, was observed in the test series for the sands S3 and S4 at $I_D = 0.3$ and 0.5.

As shown in Table 3, all samples prepared with coarse sand S3 required a higher number of cycles for initial liquefaction compared to the samples of S2 and S4. Note that tests on S3 sand with numbers CY17 ($I_D = 0.3$) as well as CY19 and CY31 ($I_D = 0.5$) did not liquefy, even after 500 cycles. Nevertheless, the excess pore-water pressure was still increasing.

It can be suggested that increasing chips content in sands S2 and S4 beyond those used in this study may produce a higher cyclic resistance compared to pure sand. The increase of resistance to liquefaction of poorly graded sand for chips content

of 20% and above is also reported in Kaneko *et al.* (2013), Mashiri *et al.* (2015), Li *et al.* (2016); Bahadori and Manafi (2016) and Bahadori and Farzalizadeh (2018). Zhou and Wang (2019) also reported an increase, but the content of tyre chips was in the range of 0–10% by dry mass. The opposite behaviour is indicated by Shariatmadari *et al.* (2018) and Promputthangkoon and Hyde (2007). In the latter study, tyre chips content was in the range of 0–15% by dry mass, and this range is too narrow to allow general conclusions. The same applies to the study by Zhou and Wang (2019).

The increased resistance to liquefaction of sand-RC mixtures may be attributed to the rigidity of RC, which is lower than that of sand, permitting some volume compression under the induced excess pore-water pressure (Towhata, 2008). Consequently, the volume compression of RC yields a condition similar to drainage or dewatering, which delays the development of excess pore-water pressure.

Figure 10 plots the average pore-pressure ratio $\bar{r}_u$ against the normalised cycle number (N/N_f). It can be seen that curves from different tests are close to each other. This means that a unique $\bar{r}_u$ against N/N_f relationship may be established, roughly independent of the relative density and the RC content but still strongly dependent on the soil type, at least for the sands S2, S3 and S4 investigated herein. The relationship $\bar{r}_u$ against N/N_f may be approximated by the following equation:

$$3. \quad \bar{r}_u = \frac{aN/N_f}{b + cN/N_f}$$

with the following parameter sets: $a = 2.3$, $b = 0.44$, $c = 2$ for sand S2 and S4; $a = 28$, $b = 1$, $c = 20$ for sand S3. This equation is valid for RC contents up to 30%.

3.3 Results for volume change in drained tests

In the drained cyclic tests, the volume change data were recorded. During these tests, a complete cycle of stress loading causes grain slipping followed by volume compaction and water discharge through the drainage line. In an undrained test on the same material under similar loading, excess pore-water pressure is developed. Chen *et al.* (2019) presented a strain-based model to assess excess water pressure by the volumetric strain changes in fine sand. This link was already suggested by Martin *et al.* (1975) and Byrne (1991) in formulating a constitutive model for the liquefaction of sands.

The change in volumetric strain in the drained test is dependent on the RC content and the cyclic stress amplitude. The change in volume during the drained cyclic tests was measured at the end of each cycle. The change in volumetric strain $\varepsilon_{v,cy}$ against number of cycles N for various RC contents are displayed in Figure 11.

Figure 10. Average pore-pressure ratio against normalised cycle number in undrained tests for mixtures of RC with (a) sand S2, (b) sand S3 and (c) sand S4

Generally, increasing the amount of RC in sand up to 20% yielded an increase in volume change for all types of mixtures. A decrease was noticed for chips content larger than 20%,

Figure 11. Volumetric strain against number of cycles in drained tests for various chips contents: (a) for S2 at $I_D = 0.3$; (b) for S2 at $I_D = 0.5$

which can be attributed to the increased overall compressibility due to particle re-arrangement during the test.

Note that for specimens that did not liquefy, the volume change increases slightly or becomes almost stable at around 120 cycles. Chen *et al.* (2019) reported a stabilisation of volume change after 250 cycles for the drained cyclic tests on saturated fine sand.

3.4 Link between drained and undrained test results

A relationship between excess pore-water pressure in undrained tests and volume change in drained tests can now be established. This relationship is portrayed by the curves $\bar{r}_u$ against $\varepsilon_{v,cy}$ in Figure 12 and can be approximated through curve fitting by the following equation:

$$4. \quad \bar{r}_u = \frac{\varepsilon_{v,cy}}{0.07 + \varepsilon_{v,cy}^{0.9}}$$

Figure 12. Excess pore-water pressure ratio in undrained tests against volumetric strain in drained tests for (a) S2 and (b) S4 at 10/20/30% RC content

Not included in the regression analysis are tests with no liquefaction or abrupt failure just after a few cycles as well as results from coarse sand (S3), since in most of the tests this type of sand exhibited only minor excess pore-water pressure.

4. Conclusions

The effects of RC on excess pore-water pressure ratio and axial strain in undrained tests as well as on the volumetric in drained tests were investigated. The testing procedure has been validated in terms of excess water-pressure evolution with an empirical predictive equation from the literature.

Generally, the addition of RC to sand at a gravimetric content of up to around 10% increases its susceptibility to liquefaction. This applies to all sands investigated.

- (a) Compared to pure sand, increasing the chips content beyond 20% yielded an improvement for sand mixture S4,

- whereas for S2 this improvement was observed at 30% chips content. For the coarse sand S3, the addition of chips does not improve its performance with respect to liquefaction. This general trend applies to both relative densities selected in the tests – that is, loose condition at $I_D = 0.3$ and medium dense condition at $I_D = 0.5$.
- (b) For all sand types, the inclusion of 10% RC causes higher deformation compared to pure sand. Further increase of RC increases resistance to axial deformation, rendering the composite material more ductile.
 - (c) Increase of RC content up to 20% yields an increase in volumetric strain. Higher values beyond this threshold did not have any significant effect.
 - (d) Two empirical relationships have been derived from the tests: one for the increase of the excess pore-water-pressure ratio with the normalised cycle number, and a second one linking excess pore-water pressure from an undrained test and cumulative volumetric strain from drained tests. They may be used for sands of similar characteristics and chips content up to 30%.
 - (e) The results are limited to an effective confining stress of 100 kPa, a single value of pre-cyclic static deviatoric stress and a single cyclic stress amplitude. Different stress paths may be investigated using this study as a guide.

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