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Small-Strain Shear Modulus of Calcareous Sand under Anisotropic Consolidation

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ABSTRACT: In this study, the small strain shear modulus of a calcareous sand was investigated by conducting bender element tests on both horizontal and vertical planes. The effects of sample preparation method, stress path and stress history on the developing of void ratio, the parameters in the modified Hardin Equation and the stiffness anisotropy were examined. The test results show that the moist tamping samples have the least void ratio variation among the five samples. The void ratio recovery in $\sigma'_h = 100$ kPa tests is higher than that in the $\sigma'_v = 100$ kPa tests. The samples prepared in dry state have lower stiffness than those prepared in moisture state, which is not influenced by the anisotropic stress state. The stiffness anisotropy induced by the sample preparation method is significant under anisotropic consolidation. In $\sigma'_h = 100$ kPa tests, the stiffness ratios at the end of the unloading stage are lower than the initial values at the loading stage, which is not found in the $\sigma'_v = 100$ kPa tests, meaning that the stress history and stress path could affect the stiffness anisotropy and cover the impact of fabric anisotropy.

KEYWORDS: small strain shear modulus, anisotropic consolidation, bender element, stiffness anisotropy, sample preparation method

INTRODUCTION

Small-strain shear modulus G_0 is an important mechanical parameter in geotechnical engineering, reflecting the deformation resistance of soil subjected to static and dynamic small amplitude shear loadings. The factors affecting the small-strain stiffness behavior of sandy soils can be summarized: physical properties, e.g., particle shape, size, fine content and gradation (Wichtmann and Triantafyllidis, 2009; Carraro et al., 2009; Yang and Gu, 2013; Choo et al., 2015; Wichtmann et al., 2015; Enomoto, 2016; Goudarzy et al., 2016; Payan et al., 2016; Liu et al., 2020; Shi et al., 2020) and testing environment, e.g., saturation and temperature (Alramahi et al., 2010; Khosravi et al., 2018; Rajabi and Sharifipour, 2018; Wang et al., 2019). The factors affecting soil fabric and stress state such as sample preparation method, cementation, isotropic and anisotropic consolidation and over-consolidation, are also important for G_0 (Jardine, 1992; Fernandez and Santamarina, 2001; Hoque and Tatsuoka, 2004; Sawangsuriya et al., 2006; Lee et al., 2009; Clayton et al., 2010; Lee et al., 2011; Ng and Xu, 2012; Gu et al., 2013; Fioravante et al., 2013; Montoya and DeJong., 2015; Senetakis and Li, 2017; Simatupang et al 2018; Payan and Chenari, 2019; Mendoza-Ulloa et al., 2020; Shi et al., 2021a; Liu et al., 2021; He et al., 2021). Some of the factors, for example, the overconsolidation ratio OCR , can be quantified and introduced into the Hardin equation (Hardin and Black, 1966) to predict G_0 of sand, which is expressed as:

$$G_0 = Af(e)\left(\frac{p'}{p_a}\right)^n OCR^m \quad (1)$$

where A and n are the experimentally determined constants; $f(e)$ is the void ratio function; OCR is defined as the ratio between the maximum effective stress in the loading stage and the effective stress in the unloading stage.

Calcareous sands are typical biogenic building materials and always own very high CaCO_3

content (Wang et al., 2011; Van Impe et al., 2015; Giang et al., 2017; Xiao et al., 2017; Xiao et al., 2019a; Liu et al., 2020). The mechanical properties of calcareous sand are always different with those of silica sands due to its special physical characteristics (Cassidy et al., 2002; Coop et al., 2004; Qadimi and Coop, 2007; Xiao et al., 2019b; Xiao et al., 2020; Xu et al., 2020; Tong et al., 2020; Tong et al., 2021; Xiao et al., 2021; Wu et al., 2021). Calcareous sands always exhibit higher irregularity in particle shape, which could promote the stiffness as the result of the stronger grain interlocking compared with that of silica sand at a similar state (Giang et al., 2017; Lv et al., 2019; Rui et al., 2020; Wei et al., 2020; Shi et al., 2021a). The small-strain shear modulus of calcareous sand was captured with the developing of offshore geotechnical engineering in marine areas where the infrastructure such as oil platform and artificial island are built upon the materials (Wang et al., 2011; Van Impe et al., 2015). Although the investigations on G_0 of calcareous sands have been performed during last decades, most of the attentions were paid on the isotropic stress state. Study disclosing the small-strain stiffness of calcareous sand under anisotropic consolidation is quite limited. It is generally recognized that the mechanical behavior of sands could be different under isotropic and anisotropic stress states, without excluding the small-strain stiffness (Hoque and Tatsuoka, 2004; Fioravante et al., 2013; Gu et al., 2013; Senetakis and Li, 2017; Payan and Chenari, 2019; He et al., 2021). For example, Fioravante (2000) performed multidirectional bender element tests on Kenya calcareous sands under both isotropic and anisotropic consolidation and showed that the small-strain shear modulus on horizontal plane is dominantly controlled by the horizontal effective stress and the stiffness on vertical planes can be conditioned with the variation of the ratio between horizontal and vertical effective stresses. In addition, stress history is an important parameter affecting the small-strain shear modulus of calcareous sands (Fioravante et al., 2013). It was proved that the interlocking among the calcareous grains could

result in the uncompleted recover of soil fabric at the unloading stage, producing higher stiffness correspondingly (Shi et al., 2020). He et al. (2021) performed a series of multidirectional bender element tests on both calcareous and silica sands under anisotropic consolidation. They proposed that calcareous sands show significant differences in stiffness and stiffness anisotropy under anisotropic consolidation compared with silica sands. Until now, study revealing the unloaded stiffness behavior of calcareous sands under anisotropic stress state, especially disclosing the developing of stiffness anisotropy, is still limited. Especially considering the significant spatial dependent behavior of stiffness, the developing of stiffness anisotropy under anisotropic unloading stress path could be multifarious due to the various characteristics of soils having different origins.

The anisotropic small-strain stiffness, indicating the spatial differentiation of G_0 in soil matrix, can be determined by both laboratory and in situ testing techniques, for example bender element test and cross hole test respectively (Fiovarante et al., 1998; Youn et al., 2008; Ku and Mayne, 2013). In this study, a series of stress path controlled triaxial tests and multidirectional bender element tests were performed to evaluate the small-strain stiffness of a calcareous sand from Persian Gulf. The shear wave velocities can be measured on horizontal and vertical planes and expressed as V_{HH} , V_{HV} and V_{VH} , corresponding to the small-strain shear modulus G_{HH} , G_{HV} and G_{VH} where the first and second subscripts denote the directions of shear wave propagating and particle motion respectively. The differences between G_{HH} and G_{HV} are always used for indicating the magnitude of stiffness anisotropy since both of them are measured on horizontal plane. Shi et al. (2021a) showed that the stiffness anisotropy of calcareous sands is highly depended on fabric anisotropy under isotropic stress state. However, the developing of stiffness anisotropy considering the effect of fabric anisotropy under anisotropic stress state is still

unexplored.

TESTING MATERIALS, APPARATUS AND PROGRAM

Testing Materials

The calcareous sand, named S1 sand, is from an artificial island in Persian Gulf (Shi et al., 2021a). It is mainly composed of marine deposits like shell and coral. Based on the ASTM standard (ASTM International, 2014), the CaCO_3 content of S1 sand was measured by using hydrochloric acid and can reach over 95%. The physical properties of S1 sand are: specific gravity $G_s=2.81$; $C_u=3.89$; $D_{50}=0.602$ mm; $e_{\max}=1.19$; $e_{\min}=0.73$. The particle size gradation and grain morphology of S1 sands are shown in Figure 1. As described in Shi et al. (2021a), the grain shape of S1 sand is angular and subangular, which was examined and quantified by X-ray CT scan tests and showed a sphericity value of 0.454. The sphericity herein is defined as the ratio of the surface area of a sphere having the same volume as a particle to the actual area of the particle (Shi et al., 2021a).

Testing Apparatus

All the tests were performed in a stress path control triaxial cell with the installation of three sets of bender elements. The bender elements were fabricated with the piezoelectric ceramics manufactured by Fuji Ceramics Corporation, Japan, having the size of 5mm in width, 10mm in length and 0.5mm in thickness. The piezoelectric ceramics were coated with a layer of epoxy resin for isolation and protection. The wire diagram of the transmitting and receiving bender elements is series type. For measuring the shear wave velocity on horizontal planes, the frictional bender element technique was used (Fioravante, 2000). In this technique, bender element was glued on a small metal plate which was attached on the inner side of membrane through a hole. The metal plate has a similar curvature with the soil sample and a thickness of around 0.5 mm. The travel

time of the shear waves was determined with the distance between two metal plates. Apart from the bender element, the shear wave velocity measurement system also consists of a digital oscilloscope and a signal amplifier. In this study, a set of hall effect sensors were used for assisting the measurement of volume change. The diagram of the tested samples can be found in Figure 2.

Testing Program

In this study, all the samples were prepared by the five methods detailed in Shi et al. (2021b) which are air and water pluviation (AP and WP), dry and moist tamping (DT and MT) and dry funnel deposition (DFD). Special attention should be given to the AP and DFD methods since they could be mixed up due to their different definitions in literatures. In the AP method, a specially designed pluviator, which is consisted of a sand holding funnel, five layer sieves and a sand collector, was used. The sample relative density is related to the mass flow rate of sands which is controlled by the funnel opening size. The DFD method was carried out by depositing sands by a long neck funnel with keeping the funnel opening near zero distance to sample surface. The densification was achieved by tapping the side of split mold. The initial relative densities of the samples were kept at $60\pm3\%$. After saturation, samples were loaded and unloaded following the stress path plotted in Figure 3. The bender element tests were performed at the end of each loading stage which was kept at round 30 min for arriving at a stable fabric state. Bender elements were excited at a frequency of 15kHz and an amplitude of 20V. The peak-peak method was adopted for interpreting signals. The typical developing patterns of the transmitted and received signals are shown in Figure 4. It can be seen that with the variation of the effective stress, the travel time of the shear waves decreases firstly and increases again. In addition, the amplitude and the waveforms of the received signals seems not be affected by stress state. At the end of each test, the sands were collected and dried in an oven for checking the particle breakage. The test program

is summarized in Table 1.

TEST RESULTS

Developing of Void Ratio

Firstly, it is necessary to mention that no significant particle breakage was found in all the tests, which is consistent with the conclusion on the particle breakage of the same sand under isotropic stress conditions in Shi et al. (2021a). They proposed that the insignificant breakage is attributed to the insufficient stress and well gradation of the calcareous sand.

Figure 5 (a) and Figure 5 (b) show the decrease and increase of the void ratio at the loading and unloading stages of the two anisotropic consolidation tests respectively. All the void ratios are normalized with the void ratio at the effective stress of 100 kPa for the comparison of volume change. For both of the tests, it is seen that the recovering of the void ratio at the unloading stage is much lower than the volume change at the loading stage, meaning that the interlocking among grains impedes the fabric recovery significantly. It is also found that the void ratio released at the unloading stage in the tests of $\sigma'_h=100\text{kPa}$ is higher than that in the test of $\sigma'_v=100\text{kPa}$. Looking into the effect of sample preparation method, it is seen that the MT samples show the least volume variation among the five samples, especially in the tests of $\sigma'_h=100\text{kPa}$. It is also noticed that the samples prepared in dry states, for example the AP, DT and DFD samples, show larger volume changes than the samples prepared in moisture states like the MT and WP samples. Shi et al. (2021a) concluded that sample made in dry state has a weaker grain interconnection, resulting in a lower soil stiffness, which can also be verified by the different soil deformation behavior given in this study. It is also known that the mean effective stress p' in the test of $\sigma'_v=100\text{kPa}$ is much higher than p' in the tests of $\sigma'_h=100\text{kPa}$. However, the volume changes at the highest stress in the two tests show little differences, except the MT samples. It means that

with the increase of the effective stress, the volume change becomes smaller and smaller. However, there is premise that particle breakage does not happen and original grains hold the sample structure. Once particles are broken at a certain stress level, the variation of void ratio could become complex and show a different pattern. The MT sample shows larger volume change in the $\sigma'_v=100\text{kPa}$ test than in the $\sigma'_h=100\text{kPa}$ test, meaning the over-consolidated structure has the potential for further volume change at a higher stress level.

Anisotropic Small-Strain Shear Modulus

Figures 6 and 7 show the developing of G_0 of S1 sands under anisotropic consolidation. All G_0 is normalized with the void ratio function proposed by Jamiolkowski et al. (1995) to exclude the effect of void ratio, as expressed:

$$f(e)=e^{-b} \quad (2)$$

where b is an experimentally determined constant. Shi et al. (2020) proposed that b is 1.195 for the calcareous sand. Figure 6 shows the developing of G_{HH} , G_{HV} and G_{VH} with the effective vertical stress σ'_v . It is seen that G_{HV} and G_{VH} are increased with the increasing of σ'_v . However, for G_{HH} , the increase of the effective stress seems having negligible effect on the developing of stiffness, meaning that the stiffness of the calcareous sand shows high spatial dependency on the direction of the effective stress. This characteristic of G_{HH} keeps the same with the conclusion given by Fioravante et al. (2013) on Kenya carbonate sand. In addition, the cross-anisotropic stiffness behavior found in the isotropic consolidation tests (Shi et al., 2021a), represented by the equality between G_{HV} and G_{VH} , is still valid in the anisotropic consolidation tests although the stiffness anisotropy indicated by the ratios between G_{HH} and G_{HV} could be lower than 1 due to the constant G_{HH} . In Figure 6 (e), it is interesting to note that G_{HV} is higher than G_{VH} for the moist tamping samples. Shi et al. (2021a) indicated that the inequality between

G_{HV} and G_{VH} of the moist tamping samples is resulted by the vertical tamping effort during the sample reconstruction. It seems that this difference between G_{HV} and G_{VH} can not be eliminated by the anisotropic consolidation stress path.

To further examine the stiffness variation with σ'_h , the three modulus G_{HH} , G_{HV} and G_{VH} versus the normalized effective horizontal stress are plotted in Figure 7. It is seen that G_{HH} shows a higher increasing rate than G_{HV} and G_{VH} with the increase of the effective horizontal stress, meaning that the stiffness increment on the horizontal plane is more contributed by horizontal stress. By combining the test results of $\sigma'_v=100\text{kPa}$, it is concluded that G_{HV} and G_{VH} exhibit almost identical increase trends with the increase of horizontal and vertical stresses respectively, meaning that the stiffness on vertical planes shows negligible dependency on stress anisotropy for the calcareous sand.

Figure 6 and Figure 7 also show the stiffness of S1 sands at the unloading stage. As indicated by Fioravante et al. (2013), He et al. (2019), Shi et al. (2020) and He et al. (2021), G_0 of calcareous sands at unloading stage is apparently higher than that at the loading stage, which can be attributed to the strong interlocking among grains resulted by the irregular particle shape. In this study, the effects of sample preparation method and stress path on this stiffness behavior of calcareous sand were examined. In Figure 6, it is found that G_{HV} and G_{VH} follow the general trend that stiffness is higher at the unloading stage excluded the MT samples. It can be easily understood that the unaffected G_{HH} is due to the insensitivity of the horizontal stiffness to the variation of the vertical stress, as He et al. (2021) proposed that the small strain stiffness is more affected by the stress in the direction of wave propagation. For the MT samples, G_{HH} , G_{HV} and G_{VH} show almost constant stiffness values at the both loading and unloading stages. This special stiffness behavior verifies the deduction in Shi et al (2021a) that the moist tamping samples of S1

sands at around 60% relative densities exhibit an over-consolidated state due to the tamping effort during the sample reconstruction, which is not changed by the anisotropic consolidation conducted in this study.

The Hardin equation was modified by Fioravante et al. (2013) for predicting the shear wave velocity of Kenya carbonate sand under anisotropic consolidation, which can be expressed for calculating G_0 as:

$$G_0 = Af(e)\left(\frac{\sigma'_h}{p_a}\right)^{n_h}\left(\frac{\sigma'_v}{p_a}\right)^{n_v}OCR^m \quad (3)$$

where n_h and n_v are the constants determined in anisotropic consolidation tests. The constant m is determined by the regression on the relationship between the ratio of the normalized G_0 at unloading and loading stages and OCR , as detailed in Shi et al (2020). In this study, the constants, A , n_h and n_v are determined with the data of the loading stage in Figure 6 and Figure 7. The respective determination of m for the two tests can be found in Figure 8 and Figure 9. Shi et al (2020) reported that the m values of G_{HH} , G_{HV} and G_{VH} are almost identical for S1 sands under isotropic consolidation. From Figure 8, it seems that the effects of sample preparation and stress path on m can not be neglected. To make a direct comparison on the values of theses parameters, A , n_h , n_v and m of all the tests are summarized in Table 2. It is seen that the samples prepared by the five methods show differences in A , behaving that the DFD samples and the MT samples have the lowest and the highest A respectively. The MT samples also show the lowest n_h and n_v among five samples, indicating that they have the strongest fabric and are the least sensitive to the variation of the effective stress. For the parameter m , the consistency among G_{HH} , G_{HV} and G_{VH} based on the isotropic consolidation tests in Shi et al. (2020) is not valid anymore. For G_{HH} in $\sigma'_h=100\text{kPa}$ tests and G_{HH} , G_{HV} and G_{VH} of the MT samples, m is around 0,

meaning that for these tests, the effect of loading history on G_0 is negligible.

He et al. (2021) found that n_h and n_v are not consistent for G_{vH} and the differences are larger for calcareous sands with more angular grains. However, in this study, this phenomenon can not be unified for the samples prepared by different methods. He et al. (2021) also show the different conclusions in literature about the values of n_h and n_v and they proposed that differences could be due to the mineralogy and particle shape of sands. Looking into the test results of this study, the effect of sample preparation method could not be neglectful.

Stress-Induced Stiffness Anisotropy

The ratio between G_{HH} and G_{HV} versus the effective vertical and horizontal stress is plotted in Figure 10 (a) and (b) to indicate the developing of the stiffness anisotropy of the calcareous sand in the loading and unloading processes. It is seen that the effect of sample preparation method on stiffness anisotropy in isotropic consolidation tests (Shi et al. 2021a) is still valid for the anisotropic consolidation tests. The AP and WP samples show the highest stiffness anisotropy. The lowest stiffness anisotropy can be found in MT and DFD samples, which means the order of fabric anisotropy induced by the sample preparation method is not changed by the anisotropic stress state. The stress induced anisotropy is significant for all the tests, behaving as the decreasing and the increasing of the stiffness ratios with the increase of the effective stress for $\sigma'_v=100\text{kPa}$ and $\sigma'_h=100\text{kPa}$ tests respectively. For the stiffness anisotropy at the loading and unloading processes, differences are found between the two anisotropic consolidation tests. It is seen that the stiffness anisotropy at the loading and unloading stages is almost identical for the $\sigma'_v=100\text{kPa}$ tests. However, for the $\sigma'_h=100\text{kPa}$ tests, the ratios between G_{HH} and G_{HV} at the end of the unloading stage are apparently lower than the initial stiffness ratios at the loading stage. This observation can be explained that for the $\sigma'_h=100\text{kPa}$ tests, the stress induced

anisotropy is more significant than that in the $\sigma'_v=100\text{kPa}$ tests and the vertical loading results in higher stiffness on vertical planes. The mechanism is similar with that of the higher vertical stiffness found in MT samples. Li and Zeng (2014) reported that with the increase of the loading in the one-dimensional consolidation tests, the long axis of sand grains tends to be horizontal aligned and the fabric anisotropy can be enhanced. Following their findings, it is deduced that the decrease of the stiffness anisotropy in this study is not induced by the change of the soil fabric. For the $\sigma'_v=100\text{kPa}$ tests, the equality of stiffness anisotropy at the loading and unloading stages indicates that stiffness is not affected with directionality. Also, the particle orientation and the fabric redistribution are deemed to be hardly affected by the variation of the horizontal loadings as well. It can be concluded that for the calcareous sands used in this study, the stiffness anisotropy in the anisotropic consolidation test is more contributed by the stress history and stress path, which could cover the impact of fabric anisotropy. He et al. (2021) showed different stiffness anisotropy behavior of calcareous and silica sands subjected to anisotropic loading and unloading. They found that silica sand is hardly affected by the anisotropic loading history. However, for calcareous sand, the stiffness anisotropy can not recover to the initial state and is changed significantly at the end of the unloading stage. They deduced that particle shape is the primary factor inducing the different stiffness anisotropy behavior. Combined with the findings in this study, it is proposed that the couple effects between particle shape and stress path could lead the developing of stiffness anisotropy of the calcareous sands, which is worth to be investigated in further study.

CONCLUSIONS

This study investigates the anisotropic stiffness behavior of a calcareous sand from Persian Gulf. Bender element tests were performed on the sand samples made by five sample preparation methods as air and water pluviation, dry and moist tamping and dry funnel deposition in two

anisotropic consolidation tests. The effects of sample preparation method and stress path on the developing of void ratio, the constants in Hardin equation and the stiffness anisotropy of the calcareous sands were examined. The main conclusions can be summarized as:

(1) The developing of void ratio is affected by the sample preparation method and stress path. Except the tests on MT samples, the $\sigma'_v=100\text{kPa}$ tests show similar void ratio variation with the $\sigma'_h=100\text{kPa}$ tests at the highest stress state although it has a higher mean effective stress. The moist tamping samples subjected to loadings show the least volume variation among the samples prepared by the five methods. The void ratio released at the unloading stage in the tests of $\sigma'_v=100\text{kPa}$ is higher than that in the test of $\sigma'_h=100\text{kPa}$.

(2) The small strain stiffness of the calcareous sand at the unloading stage is higher than that at the loading stage, which is proposed to be induced by the irregular particle shape. The increases of G_{HH} are mainly contributed by the increase of horizontal effective stress. The over-consolidated state of moist tamping samples is not changed by the anisotropic consolidation stress state. The parameter m in Hardin equation in anisotropic consolidation tests does not keep constant for G_{HH} , G_{HV} and G_{VH} . The MT samples show the lowest values of A , n_h and n_v among the samples prepared by the five methods. The comparison between the values of n_h and n_v is also affected by sample preparation method.

(3) The stiffness anisotropy induced by the sample preparation methods is significant for the anisotropic consolidation tests. At the end of unloading stage in the $\sigma'_h=100\text{kPa}$ tests, the stiffness anisotropy ratios are lower than the initial values at loading stage, which is not found in the $\sigma'_v=100\text{kPa}$ tests. For the calcareous sands used in this study, the stiffness anisotropy in the anisotropic consolidation tests is affected by the stress history and stress path more than the fabric redistribution. It is proposed that the couple effects between particle shape and stress path could

lead the developing of stiffness anisotropy of the calcareous sands.

DATA AVAILABILITY STATEMENT

Some or all data, models, or code generated or used during the study are available from the corresponding author by request.

DECLARATION OF INTERESTS

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Table 1. Test program using different sample preparation methods

Tests	Sample preparation method	Initial relative density	Initial void ratio
$\sigma'_h = 100$ kPa	Air pluviation	57%	0.908
	Water pluviation	61%	0.892
	Dry tamping	57%	0.905
	Dry funnel deposition	62%	0.885
	Moist tamping	62%	0.895
$\sigma'_h = 100$ kPa	Air pluviation	62%	0.883
	Water pluviation	60%	0.904
	Dry tamping	60%	0.894
	Dry funnel deposition	62%	0.882
	Moist tamping	58%	0.912

Table 2. Summary of parameters in Equation (3).

Tests	Parameters	G_0	Air pluviation	Water pluviation	Dry tamping	Dry funnel deposition	Moist tamping
$\sigma'_h =$ 100 kPa	A	HH	120.38	121.99	97.49	88.78	127.14
		HV	64.74	74.05	76.13	70.98	112.25
		VH	63.50	75.95	72.43	71.12	95.52
	n_v	HH	-0.032	0.081	0.102	0.049	0.004
		HV	0.340	0.339	0.322	0.317	0.302
		VH	0.350	0.346	0.365	0.332	0.220
	m	HH	-0.031	-0.068	0.026	-0.068	-0.026
		HV	0.168	0.179	0.174	0.179	0.025
		VH	0.162	0.156	0.183	0.156	0.015
$\sigma'_v =$ 100 kPa	A	HH	115.22	123.55	97.96	87.39	125.58
		HV	67.70	78.88	77.66	76.90	113.25
		VH	71.35	73.26	76.12	74.74	101.03
	n_h	HH	0.569	0.562	0.582	0.603	0.391
		HV	0.355	0.328	0.369	0.330	0.213
		VH	0.365	0.329	0.354	0.326	0.203
	m	HH	0.197	0.176	0.171	0.211	0.001
		HV	0.188	0.166	0.177	0.209	0.001
		VH	0.187	0.179	0.161	0.192	-0.025

Figure 1. Particle size gradation and morphology

Figure 2. Diagram of the tested sample with bender elements and hall effect sensors

Figure 3. Stress path of the anisotropic consolidation tests: (a) $\sigma'_h = 100$ kPa; (b) $\sigma'_v = 100$ kPa.

Figure 4. Waveforms of shear wave under different effective stresses

Figure 5. Void ratio variation versus the effective stress: (a) $\sigma'_h = 100$ kPa; (b) $\sigma'_v = 100$ kPa.

Figure 6. Normalized shear modulus versus the effective stress: (a) air pluviation; (b) water pluviation; (c) dry tamping; (d) dry funnel deposition; (e) moist tamping

Figure 7. Normalized shear modulus versus the effective stress: (a) air pluviation; (b) water pluviation; (c) dry tamping; (d) dry funnel deposition; (e) moist tamping.

Figure 8. Ratio of the normalized shear modulus versus OCR for tests with $\sigma'_h = 100$ kPa: (a) air pluviation; (b) water pluviation; (c) dry tamping; (d) dry funnel deposition; (e) moist tamping

Figure 9. Ratio of the normalized shear modulus versus OCR for tests with $\sigma'_v = 100$ kPa: (a) air pluviation; (b) water pluviation; (c) dry tamping; (d) dry funnel deposition; (e) moist tamping.

Figure 10. Variation of stiffness anisotropy versus the effective stress: (a) $\sigma'_h = 100$ kPa; (b) $\sigma'_v = 100$ kPa.