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Title:

On the undrained shear behavior of sand mixtures with non-plastic fines subjected to large shear displacement

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Abstract:

Fluidized landslides pose significant hazards due to the catastrophic failure and flow-like behavior with high mobility and long traveling distances. Non-plastic fines are abundant in fluidized landslides (the content of relatively fine-grained particles could reach 30% - 40% in some fluidized landslides), but the role of fine particles in the initiation and movement of these landslides is not well understood. To investigate the effects of non-plastic fines content (FC) on the undrained shear behavior of the mixture, undrained ring shear tests were performed on mixtures of silica sand ($D_{50} = 0.16$ mm) with varying proportions of non-plastic fines ($D_{50} = 0.035$ mm) by weight at different initial density conditions. Our findings show that liquefaction potential increases as fines content increases and initial density decreases. For each sample, the peak strength (τ_p), steady-state strength (τ_{ss}), and total dissipated shear energy to the steady state (W_{ss}) increased in general as the sample became denser. Given the same initial density, τ_p , τ_{ss} , and W_{ss} decrease with the increase of FC in general. We further compared these results by using intergranular void ratio (e_s) and equivalent intergranular void ratio (e_s^*). We found that plots of τ_p , τ_{ss} , and W_{ss} against e_s^* merged to a relatively narrow domain, indicating e_s^* serves as a comprehensive indicator for evaluating the FC's effects on undrained shear behavior. There exists an optimum e_s^* at which the brittleness index (I_B , which is normally utilized for analyzing the consequence of liquefaction associated with post-failure behavior) was minimal for the mixtures. The W_{ss} in the log scale continuously decreased with increasing e_s^* . The addition of fine particles could change the inter-particle force transfer. Fine particles partially come into the contacts between coarse grains, engaging in forces transfer structure (as active fine particles). As the FC increases, a greater proportion of fines actively engage in the forces transferring structure, resulting in increasing liquefaction potential and reducing the energy dissipation throughout the entire shearing process of the saturated mixtures, thereby facilitating the fluidization and mobility of landslides. This study provides the evaluation of the full process liquefaction behavior of mixtures rich in fines and further explains the fines'

role in the high mobility of fluidized landslides based on the energy dissipation view.

Keywords: Fluidized landslide; Fines content; Undrained ring shear test; Shear behavior; Equivalent intergranular void ratio; Energy dissipation

1. Introduction

Fluidized landslides represent a profound concern within the field of geohazard mitigation, given their great threat to human lives and local properties (Hung et al., 2001). Characterized by high mobility, extensive runout distances, and the inclination to mobilize and entrain additional material along their path, fluidized landslides rank among the most devastating appearances of mass movement. By now, a lot of investigations had been undertaken to elucidate the initiation and kinematic mechanisms of the fluidized landslides, and it has been revealed that liquefaction could be the main reason for their rapid motion (Castro and Poulos, 1977; Wang and Sassa, 2001).

For fluidized landslides, the coexistence of fine materials within the debris matrix is a general phenomenon. These fine materials may originate from diverse sources, either as inherent components of the initial landslide materials (Ishihara et al., 1990; Wang et al., 2015) or as a result of the fragmentation of the landsliding mass (Perinotto et al., 2015). It has also been reported that many secondary landslides had been triggered and fluidized on loosely deposited landslide materials (Hu et al., 2018), in which the presence of fine particles makes the landslide material into an inherently heterogeneous mixture, resulting in the changes in the initiation and kinematics of the landslides.

Since the 1964 Niigata earthquake, considerable efforts have been made to explore the liquefaction features of uniform soil materials and clean sands. Valuable insights into the liquefaction behavior of clean sands under both cyclic and static loading have been obtained (Seed and Idriss 1967; Castro 1969; Sladen

et al. 1985; Ishihara 1993; Take et al., 2015). Recently, there has been a growing focus on investigating the liquefaction behavior of soil mixtures comprising both sand and fines (Tafili et al., 2023). However, unlike the relatively well-defined outcomes observed in tests involving clean sand, the influence of fines content on the liquefaction phenomenon of sandy soil samples remains unclear, which could be exemplified by different findings reported in previous research efforts. Several studies have shown that the addition of non-plastic fines can enhance the liquefaction resistance of tested sands (Pitman et al., 1994; Kwa and Airey, 2017), while other investigations have demonstrated a decreasing tendency in resistance (Zlatović and Ishihara, 1995; Lade and Yamamuro, 1997; Yamamuro and Lade, 1999; Monkul et al., 2017). Moreover, several studies have proposed fines content threshold (FC_{th}) values, after that, the increase in fines content (FC) can either positively or negatively affect liquefaction resistance (Thevanayagam et al., 2002; Yang and Wei, 2012; Rahman and Sitharam, 2020). These diverse findings indicate that understanding of the undrained liquefaction phenomenon of mixed soil samples remains unclear, and further investigation will be needed.

In addition, many studies on mixtures have used triaxial apparatus with small samples and limited shear displacement, which may not fully capture the long-distance shearing behavior that appeared in these landslides with rapid long-runout movement (Wang et al., 2007). Nevertheless, it is often challenging for laboratory undrained triaxial tests to reach a true steady state. Some researchers state that a steady state is manifested at 30% axial strain in loose specimens, while they consider the state of peak stress following phase transformation (PT) can be described as the steady state in dense specimens (Wang and Yoshimine, 2023). Alternatively, some studies suggest extrapolating stress-strain data to find the critical state (Yilmaz et al., 2023). Consequently, the quest to identify the authentic full steady state strength in the undrained triaxial experiments in the laboratory, particularly in the context of very large displacement shearing characteristics of fluidized landslides, remains challenging. Furthermore, while the notion of transitional

fines content has gained acceptance in laboratory triaxial tests for mixture behavior (Zuo and Baudet, 2020; Barnett et al., 2021; Otsubo et al., 2022), its applicability in post-failure behavior subjected to large undrained shear displacement remains an unexplored frontier. Scant literature exists to date that delves into the impact of non-plastic fines content on the undrained shear behavior of mixtures when subjected to large displacements. This warrants further investigation to advance our understanding of this critical domain.

To better understand the influences of fines content (FC) on the undrained shear behavior of mixtures of sands with fines subjected to large-shear displacements, we conducted undrained ring shear tests on the mixtures of sands with varying proportions of non-plastic fines by weight under different initial densities. Through evaluating the undrained peak shear strength, steady state strength, brittleness index, and shear energy dissipated throughout the whole shear progress until steady state, we discussed the role of fines in undrained shear behavior, post-failure characteristics, and the mobility of fluidized landslides. This study can provide the experimental evaluation of the full process liquefaction behavior of fines-bearing geomaterials subjected to large shear displacement in the field of engineering geology, improving the accuracy of predicting the liquefaction characteristics of geological materials rich in fines. This study can further explain the fines' role in the high mobility of fluidized landslides based on the energy dissipation view, aiding the prediction of landslide behavior and the assessment of post-failure hazards.

2. Materials and Methods

2.1 Materials

In the study, silica sand No.7 (denoted as S7) and silica powder #100 (denoted as SP) were selected as representative coarse and fine materials because they are commercially available materials in Japan. The mixtures comprising S7 blended with varying proportions of SP were employed. Among them, SP, derived from silica sand through mechanical grinding, is characterized as non-plastic fines. We systematically

varied the fraction of SP in the mixtures, creating five compositions: 0%, 10%, 20%, 30%, and 40% by weight, since the content of relatively fine-grained particles could reach 30% - 40% in some natural fluidized landslides. The corresponding mixtures are hereafter referred to as S7, M10, M20, M30, and M40, respectively. Fig. 1a depicts the grain size distributions of samples, while Fig. 1b and 1c present scanning electron microscopy (SEM) images. The average particle sizes (D_{50}) for S7 and SP are measured as 0.16 mm and 0.035 mm, respectively.

Fig. 2 shows the measured permeability, maximum, and minimum void ratios ($e_{\max}$ and $e_{\min}$) following the Japanese Geotechnical Society standards (Japanese Geotechnical Society, 2011). As shown, the void ratio shows a distinct transitional behavior, marked by an estimated fines content threshold (FC_{th}): $e_{\max}$ and $e_{\min}$ reach their minimum at $FC = 30\%$ and 40% , respectively. Table 1 presents other basic properties of these samples.

2.2 Ring shear test

Apparatus

Ring shear apparatuses (DPRI-No.1 to DPRI-No.7) (Sassa et al., 2004) have been developed at the Disaster Prevention Research Institute (DPRI), Kyoto University, and these apparatuses enable the undrained shear tests on samples subjected to large shear displacement at diverse shear rates (with a maximum to 3.0 m/s) under different loading conditions. In this study, we employed a DPRI-No.6 machine, featuring a shear box with 350 mm in outer diameter, 250 mm in inner diameter, and 150 mm in height, and the mechanism can achieve shear velocities as high as 224 cm/s.

To help better understand the principle of the undrained ring shear experiment, the configuration of the ring-shear apparatus was presented in Fig. 3. In this setup, the sample is laterally confined between the pair of upper and lower restraining metal rings and is subjected to the normal loading by the annular loading

platen linked to the oil piston. In addition, the lower section of the machine's shear box is capable of bidirectional rotation, driven by the servo-motor system. Conversely, the upper part remains stationary, aided by two retaining torque arms employed for measuring shear resistance. Upon the occurrence of the tested sample's shear failure, the annular ring-shaped sample undergoes shearing due to the relative rotary motion (Fig. 3a). For undrained ring shear apparatus, the fundamental design centers on the undrained shear box that is impermeable (Fig. 3b). To ensure the undrained shearing, O-rings are placed on the upper loading platen. Simultaneously, the two restraining rings of the lower rotary part are equipped with bonded rubber edges. The rubber edges of the lower rotary pair are treated with a coating of Teflon spray and vacuum silicon grease, to reduce the possible friction. A constant touching force between the upper rings on the rubbers is precisely controlled and achieved by a gap control system. More detailed information on the design, construction, and operational procedures for the ring shear apparatuses is commonly available in Sassa et al. (2004).

Test Procedure

We conducted all the tests under a uniform initial stress state (initial normal stress = 200 kPa, and initial shear stress = 0 kPa). To attain samples with different initial densities, we employed both the moist-placement and dry-deposition methods according to procedures described by Ishihara (1993). For getting loose samples, we utilized the moist-placement method. Initially, we added the de-aired water into the dry sample to attain the moistened sample with a gravimetric water content of 5%, then the sample was poured into the shear box. The sample was carefully layered and tamped during packing to ensure uniformity. In the case of medium-dense or dense samples, we used the dry-deposition method. We freely poured the oven-dried specimen into the shear box in multiple layers. For getting various initial densities, we varied the tamping intensity during the packing process in different tests.

We saturated the samples by flowing CO₂ and then de-aired water. To ensure that specimens were fully saturated, we checked the B_D value, a parameter proposed by Sassa (1988) for assessing the sample's saturation degree in the undrained direct shear tests. It is noted that the B_D parameter is similar to B parameter, which was proposed by Skempton (1954) and normally used in the undrained triaxial compression tests. Because B_D parameter is almost equal to the parameter of B for the uniform samples (Okada et al., 2000), in this study, we used B_D parameter for the undrained ring shear tests. The B_D parameter is expressed as $B_D = \Delta u / \Delta \sigma$, the ratio between Δu (the increment of pore water pressure (PWP) under undrained conditions) and $\Delta \sigma$ (the increment of normal stress). If the B_D value ≥ 0.95 , the specimen can be considered approximately fully saturated. Here, all samples exhibited full saturation ($B_D \geq 0.95$) before running the undrained shear. After the assessment of the B_D value, the sample was subjected to consolidation under a given initial 200 kPa normal stress with drained conditions. Thereafter, the sample was sheared under undrained conditions by the controlling mode of raising shear stress, and the loading rate was 0.2 kPa/s. Throughout the tests, all the transducers record at a 10 Hz sampling rate. Here, the shearing continued to reach the steady state following Poulos (1981), i.e., at which both pore pressure and shear resistance do not show further variation. Table 2 presents the detailed results of undrained shear tests.

3. Undrained shear test results

3.1 Effects of Initial Density State

It is well known that initial density significantly influences undrained shear behavior (Wang and Sassa, 2002; Okada et al., 2005; Igwe et al., 2012; Cui et al., 2017). To have a better understanding of the undrained shear behavior of sand materials that suffered large displacement shearing under different initial densities, Fig. 4 presents the tested results of S7 with samples' initial void ratios of 0.99, 1.02, 1.07, 1.12, and 1.14 respectively (corresponding to decreasing density indices I_d of 0.28, 0.21, 0.11, 0.00, and -0.04, density

index $I_d = (e_{\max} - e) / (e_{\max} - e_{\min})$. As shown, Fig. 4a-b illustrates shear resistance, normal stress, and pore-water pressure against the shear displacement, and Fig. 4c presents the effective stress paths. The effective stress paths have a significant change with the increase in the void ratio. In the case of dense samples with void ratios being 0.99 and 1.02 (density indices of 0.28 and 0.21), the samples exhibited substantial strain hardening following the phase transformation (PT) location. In this phase, shear resistance increased because of a reduction in pore pressure, forming an "elbow" shape characterized by a turning point. After failure (peak strength), shear strengths decreased due to the progressive rise in PWP (Fig. 4a-c). This kind of behavior with phase transformation resulted from the potential of dilation followed by contraction of the sample during the undrained shear and then was termed as "D-C" behavior thereafter. It is also noticed that as the soil density decreased, the strain-hardening process weakened, leading to a reduction in peak strength. Among tested samples, S7 with a void ratio being 1.07 (density index of 0.11) exhibited the weakest strain-hardening process. For the loosest sample of S7, with void ratios being 1.12 and 1.14 (density indices of 0.00 and -0.04), the strain-hardening process was entirely replaced by contractive behavior, implying an absence of dilatancy throughout the shear process. This behavior is evident in Fig. 4b, where the pore-water pressure continually increased as shear displacement progressed. This kind of shear behavior resulted from the potential of pure contraction under undrained shearing and then was termed as "C" behavior thereafter. Moreover, both peak and steady-state strengths generally decreased as density decreased (Fig. 4a and c).

3.2 Effects of fines content

The effect of fines content on the undrained shear behavior is examined by conducting tests on samples of S7, M10, M20, M30, and M40 under different initial densities. The following sections compare the tested results.

Loose samples

In the loose state, we first selected two undrained ring shear tests on samples of S7 and M10 to examine whether the addition of fines can affect the undrained shear behavior of mixtures. The initial void ratios for S7 and M10 were 1.14 and 1.04, respectively. Results are illustrated in Fig. 5a-b, which include plots of normal stress, pore-water pressure, and shear resistance versus shear displacement (Fig. 5a). Fig. 5b illustrates the effective stress paths. As shown in Fig. 5a, the pore-water pressure in M10 was greater than that in S7 under the same shear displacement in general, although the difference was small. It is noteworthy that the effective stress paths behaved approximately the same, although M10 had a smaller steady-state shear strength.

In addition, the results of three tests on M20, M30, and M40 are compared in Fig. 5c and d to examine the impact of further increasing fines content on the undrained shear behavior under loose conditions. The void ratios for the samples of M20, M30, and M40 were 0.96, 0.88, and 0.81, respectively. Fig. 5c presents the normal stress, shear resistance, and pore-water pressure against shear displacement, while Fig. 5d depicts the effective stress paths. As presented in Fig. 5c and d, as the content of fines increased from 20% to 40%, the undrained shear strength at the steady state decreased. It is noteworthy that peak strength exhibited a slight increase when the contents of fines increased from 20% to 40%, although there were no significant differences between M30 and M40.

Dense samples

In the tests conducted on dense materials, as shown in Fig. 6, the influences of fines content on the undrained shear behavior at large shear displacement become more significant. In Fig. 6, the tested results on the samples S7, M10, M20, M30, and M40 are illustrated, in the forms of shear strength and pore water

pressure versus shear displacement (Fig. 6a and b), and effective stress paths (Fig. 6c). The void ratios are 0.95, 0.88, 0.81, 0.77, and 0.77, for S7, M10, M20, M30, and M40, respectively. An increase in FC results in notable changes in the effective stress paths. For the dense samples S7, M10, M20, and M30, substantial strain hardening is evident after the phase transformation (PT) point. In this phase, shear resistance experiences a significant increase attributed to decreasing pore pressure. After peak strength, shear strengths decreased with further progress of shearing because of the accumulation of PWP (as seen in Fig. 6a-c). As FC increases, the strain-hardening process weakens, resulting in peak strength reduction. This trend is most pronounced in M40 (with a void ratio of 0.77), in which, no hardening process could be identified, indicating the absence of dilatancy tendencies throughout the entire shear process. Furthermore, the shear strength at steady-state decreases with increasing FC in general (Fig. 6a and c). These results suggest that the addition of fines could elevate the liquefaction potential of mixtures (there is still a transition in liquefaction behavior from a “D-C” to a “C” pattern in the stress path), despite the increase in density index with increasing fines content.

4. Discussion

4.1 Density variables

From the above-mentioned figures, it is clear that both FC and initial density played key roles in the undrained shear behavior of the mixtures. An isolated consideration of density or FC effects in mixtures proves inadequate, failing to comprehensively capture the true influence of FC (Thevanayagam et al., 2002; Rahmani and Naeini, 2020; Porcino et al., 2021). Consequently, alternative density variables simultaneously considering the role of fine particles have been proposed (Thevanayagam, 1998). These variables, termed intergranular void ratio (e_s) and interfine void ratio (e_f), are defined as:

$$e_s = \frac{e + (FC/100)}{1 - (FC/100)} \quad (1)$$

$$e_f = \frac{e}{FC/100} \quad (2)$$

where e = void ratio, and FC = fines content. These parameters are introduced under the following assumption: In mixtures with low fines content, all fines are treated as voids, which has disregarded impact on the force structure; however, as fines content surpasses a critical value, the force structure becomes primarily governed by interactions between the fines.

Thevanayagam et al. (2002) extended their proposal to encompass the influence of both active and inactive fines given that mixtures include both inactive fine grains (which remain within voids produced by coarse particles) and active fine grains (which come into contact between the coarse particles. Two updated density variables: equivalent intergranular void ratio (e_s^*) and equivalent interfine void ratio (e_f^*), are expressed as:

$$e_s^* = \frac{e + (1-b)(\frac{FC}{100})}{1 - (1-b)(\frac{FC}{100})} \quad (3)$$

$$e_f^* = \frac{e}{\frac{FC}{100} + \frac{1-FC/100}{R_d^m}} \quad (4)$$

where b represents the percentage of fine particles that actively participate in force structure (Kanagalingam and Thevanayagam, 2005), R_d is the size disparity ratio (D_{10} / d_{50}), and m is a coefficient, with a range of $0 < m < 1$. It would be noted that in the triaxial test, the fines are most defined as silt (grain size $< 0.075\text{mm}$), and in our tests, the sample of SP, the silt content is about 84%. Here we referred to the definition of fines in the triaxial test, in the following discussions, we considered the silt content as calculated fines content in our SP samples in the later calculations, the silt contents are separate of 8.4%, 16.8%, 25.2%, and 33.6% in the SP samples named by M10, M20, M30, and M40, and we regarded the remain grains larger than 0.075 mm as coarser grains.

In the context of these variables, it is crucial to determine the threshold fines content (FC_{th}) and b (Ni et al., 2004; Barnett et al., 2021; Gobbi et al., 2022a). Currently, one approach is using e_{max} and e_{min} indices, assuming that estimated FC_{th} represents the fines content proportion where e_{max} and e_{min} of the mixture

arrive at minimum values (Lade et al., 1998; Rahmani and Naeini, 2020; Zuo and Baudet, 2020). Our results of $e_{\max}$ and $e_{\min}$ suggest that the estimated FC_{th} is between 30% ~ 40%. The other approach is using semi-empirical equation presented by Rahman et al. (2009), as follows:

$$FC_{th} = 0.40 \times \left(\frac{1}{1+e^{\alpha-\beta\chi}} + \frac{1}{\chi} \right) \quad (5)$$

where α and β are curve-fitting constants ($\alpha = 0.50$, $\beta = 0.13$), and χ is the D_{10} / d_{50} . A value of estimated $FC_{th} = 32\%$ of tested samples by using Eq. (5), which is approximately consistent with the estimated results (around 30% ~ 40%) from the former approach. Considering silt content in our SP samples and the physical uncertainty around precise FC_{th} calculated by the semi-empirical formula and ring shear tests are under simulating natural landslide-stress level, here, in the following discussions, we use 40% as the value of estimated FC_{th} in the ring shear tests, namely, we use e , e_s/e_s^* for S7, M10, M20, M30, and M40.

The value of b could be calculated by using the semi-empirical formula approach (Rahman and Lo, 2008; Mohammadi and Qadimi, 2015), the formula is written as:

$$b = \left[1 - \exp \left(-\frac{0.3}{k} \right) \right] \times \left(r \frac{FC}{FC_{th}} \right)^r \quad (6)$$

where r is $1/\chi$, k is $1-r^{0.25}$. Here, b is computed as 0%, 33%, 40%, 45% and 49% for S7, M10, M20, M30, M40.

Furthermore, the relative density (D_r) can standardize the influence of the average grain dimensions in cases where the influence of FC could be ignored. Similarly, the equivalent granular void ratio (e_{ge}) can normalize FC 's influence in cases where the average size's effect could be ignored. Consequently, the equivalent granular relative density (Dr_{ge}) is introduced relying on both D_r and e_{ge} and is used for evaluating liquefaction resistance and steady-state behavior in triaxial tests (Thevanayagam 2007a; Watanabe et al., 2015; Porcino and Diano, 2017; Wang and Yoshimine, 2023). The definition of equivalent relative density (Dr_{ge}) involves the transformation of the formula for relative density (D_r) as follows:

$$Dr_{ge} = \frac{e_{\max(FC=0\%)} - e_{ge}}{e_{\max(FC=0\%)} - e_{\min(FC=0\%)}} \quad (7)$$

where $e_{\max}$ = maximum void ratio, $e_{\min}$ = minimum void ratio of coarser material containing 0% content of fines. In this study, we used density index (I_d) to replace relative density (D_r). Specifically, we calculated the intergranular density index (I_{ds}) and equivalent intergranular density index (I_{ds}^*) separately based on the corresponding values of e_s and e_s^* replacing e_{gc} .

These indices have been mostly studied and discussed within the context of triaxial compression tests, both undrained and drained, to investigate the influence of fine grains. For undrained large displacement shearing of mixtures, Wang et al. (2007) employed interfine void ratio (e_f) to assess the impact of loess content on liquefaction behaviors starting from the critical state perspective. To have a comprehensive understanding of the influences of FC on the shear behavior of the mixtures under undrained conditions, in the following, we assess peak strength (τ_p), steady-state strength (τ_{ss}), brittleness index (I_B), and energy dissipation to the steady state (W_{ss}) using various density variables, including void ratio (e), density index (I_d), intergranular void ratio (e_s), equivalent intergranular void ratio (e_s^*), intergranular density index (I_{ds}) and equivalent intergranular density index (I_{ds}^*).

4.2 Peak strength

The undrained peak strength (τ_p) is a key index commonly employed to evaluate a sample's liquefaction potential. τ_p is inherently dependent on the initial stress conditions and the sample's density (Wang and Sassa 2002). Given that all tests were conducted under the same initial applied stress state (normal stress is 200 kPa and shear stress is 0 kPa), the changes in the τ_p for each sample presented in Table 2 can be ascribed to differences in the sample's densities.

All the values of τ_p for Samples S7-M40 have been summarized and reviewed by using the initial void ratio (e , Fig. 7a), density index (I_d , Fig. 7b), intergranular void ratio (e_s , Fig. 7c), intergranular density index (I_{ds} , Fig. 7d), equivalent intergranular void ratio (e_s^* , Fig. 7e), and equivalent intergranular density index

(I_{ds}^* , Fig. 7f). Peak shear strengths in logarithm scale for each sample basically present a linear variation in general with each of these density variables including e/I_d and e_s/I_{ds} . As anticipated, τ_p became greater in general when the sample became denser (represented by a smaller e or greater I_d for each sample). From the variation tendency of τ_p shown in Fig. 7a and b, for a given e or I_d , τ_p decreased as the fine-particle content increased. It is also noted that the lines of $\log(\tau_p)$ against e , I_d , e_s , or I_{ds} , respectively, shifted in position significantly from the host sample of S7. Nevertheless, when viewed by e_s^* or I_{ds}^* , it is found that these lines moved to a narrow range (Fig. 7e and f), indicating that e_s^* and I_{ds}^* had better performance in evaluating the effects of FC on the undrained peak strength. In Fig. 7e, τ_p initially exhibited a sharp decline with increasing e_s^* from 0.90 to approximately 1.11. Subsequently, it remained an approximately slow descent in general when e_s^* exceeded 1.11.

From Fig. 7, it is noted that Fig. 7a and b, Fig. 7c and d, and Fig. 7e and f, basically provided similar variation patterns with the corresponding density variables, in the following sections, we will just view the results by e , e_s , and e_s^* .

4.3 Steady state strength

The steady-state strength (τ_{ss}) holds significant implications for interpreting the behaviors of a liquefied soil mass, particularly in the assessment of fluidized landslides. However, the determination of shear strength at steady-state has caused considerable discussions due to the varying approaches in its definition within the framework of triaxial tests (Poulos 1981; Vaid et al., 1990; Alamouti et al., 2023). Some research, for example, suggests that a steady state can be identified when the axial strain (in triaxial tests) reaches 30% in loose specimens, while dense specimens are recognized as reaching the steady state at the peak stress state following phase transformation (Wang and Yoshimine, 2023). Alternatively, other studies argue that extrapolating the stress-strain curve is essential to attain a critical state (Yilmaz et al., 2023).

Compared to conventional triaxial tests, the ring shear test offers distinct advantages in determining true steady-state strength (Wang and Sassa, 2002). This advantage arises from the significantly larger shear displacement employed in ring shear tests compared to conventional triaxial tests (Okada et al., 2005), enabling all grains' orientation and possible particle breakage to reach a complete steady state as defined by Poulos (1981).

Consequently, in our study, we have examined the shear strengths at the complete steady state in our analysis of undrained shear behavior of mixture soils characterized by varying FC content. These results are displayed in Fig. 8 against e (Fig. 8a), e_s (Fig. 8b), and e_s^* (Fig. 8c). As observed, τ_{ss} exhibited a consistent trend of increase as the sample's density increased (reflected by smaller e) across all samples (Fig. 8a). Furthermore, given e being the same, τ_{ss} decreased as fine content increased, showing good consistency with the results of Naeini and Baziar (2004) and Wang et al. (2007). From Fig. 8a, it is seen that the line of τ_{ss} against e showed different slopes, and the slope became steeper with an increase of FC.

However, when viewed by e_s , these lines are arranged in parallel approximately, although a significant distance could be identified among these lines (Fig. 8b). A further view of τ_{ss} by e_s^* indicates that these lines are all gathered into a very narrow domain (Fig. 8c), showing that e_s^* also had better performance in evaluating the FC's effects on the undrained steady-state shear strength. Notably, τ_{ss} displayed a continuous decrease with increasing e_s^* in general (Fig. 8c).

4.4 Brittleness index

The magnitude of reduction in the material's shear strength due to the onset of liquefaction can vary significantly. This reduction depends on the initial state of materials including void ratio and stress, and may greatly affect the post-failure behavior of the liquefied materials. In the following, we use the brittleness index (I_B), a concept introduced by Bishop (1973), to assess the post-failure behavior of liquefied

materials. The brittleness index (I_B) is written as:

$$I_B = \frac{\tau_p - \tau_{ss}}{\tau_p} \quad (8)$$

where τ_p = peak strength, τ_{ss} = steady state strength.

The brittleness index (I_B) for samples S7-M40 has been plotted against e (Fig. 9a), e_s (Fig. 9b), and e_s^* (Fig. 9c), respectively. As observed, for S7, as the sample became denser (indicated by a decrease in void ratio), I_B decreased to a certain value, after which I_B began to increase. Namely, an optimal density index existed where undrained I_B was the smallest. A similar phenomenon can also be found in the other samples but disappeared in the tests on M40. For M40, I_B increased continuously with the increasing e . This may result from that we failed to prepare denser specimens by M40 by the same sample-preparing method, forming a relatively small densities range. From Fig. 9 it is also noticed that when viewed by e , e_s the domain of I_B shifted rightward with an increase of these density variables. However, these plots of I_B against e_s^* are shifted to a relatively narrow range (Fig. 9c), and the variation tendency of I_B showed that there was an optimum e_s^* value of approximately 1.11 at which the I_B was the smallest for all the mixtures. Before this point, the I_B gradually decreased as e_s^* increased, but beyond this point, it sharply increased with further increases in e_s^* (Fig. 9c). Therefore, from the point of view of the brittleness index, it is clear that e_s^* also present good performance in evaluating FC's effects on undrained shear behaviors. The optimum e_s^* phenomenon was attributed to the rapid initial decrease in τ_p , followed by an approximately slow descent in general, while the corresponding τ_{ss} exhibited a continuous decreasing trend (Figs. 7c, 8c, and 9c).

4.5 Shear energy

The energy approach has proved to be a valuable method for assessing liquefaction potential (Figueroa et al., 1994; Sassa et al., 2005; Thevanayagam, 2007b; Polito et al., 2013; Yang and Pan, 2018; Jain et al., 2023; Fardad Amini and Yang, 2024). As reported, fines content has different effects on energy dissipation

during the liquefaction initiation (Baziar and Jafarian, 2007; Baziar and Sharafi, 2011; Javdanian, 2019), and different density variables, such as e , e_s/e_f , e_s^*/e_f^* , have been introduced to evaluate the liquefaction potential of mixtures from view of energy that required to induce liquefaction (Thevanayagam, 2007b). Nevertheless, few attempts have been made to study the dissipated energy during the whole shear process until the complete steady state (W_{ss}) under undrained shear tests, especially concerning the effects of fines on energy consumption throughout the full duration of shearing.

To comprehensively investigate the FC and sample density's effects on shear energy dissipation in mixtures, we have plotted the shear energy dissipation for all tested samples against e (Fig. 10a), e_s (Fig. 10b), and e_s^* (Fig. 10c), respectively. It is noted that in our study, the total shear energy dissipation to steady state (W_{ss}) was calculated as follows:

$$W(\text{J/m}^2) = \sum_{i=1}^{n-1} \frac{1}{2} (\tau_i + \tau_{i+1}) (l_{i+1} - l_i) \quad (9)$$

where τ is shear resistance; l is shear displacement; and n is the number of points recorded.

The total shear energy to steady state (W_{ss}) depends on two influencing factors: shear stress variations of mixtures and the corresponding total shear displacement (l_{ss}), the detailed results are presented in Table 2. From Fig.10, W_{ss} increased as the sample became denser (with a smaller e) for each sample (Fig. 10a). In addition, given the void ratio (e) being the same, W_{ss} decreased with an increase in FC. It would be also noted that for each specimen, W_{ss} in the log scale presents a linear relationship with all these density variables, and the lines of W_{ss} against e , e_s for each sample kept a significant distance (Fig. 10a and b). However, when viewed by e_s^* (Fig. 10c), all the lines swarmed into a narrow domain, showing that for all the mixtures, W_{ss} can be summarized by either e_s^* . Evidently, the log-scaled shear energy dissipated to the steady state exhibited a continuous and linear decrease with the increasing e_s^* . Moreover, with an increase in FC, the plots for each sample shifted toward the lower right in the respective figures (Fig. 10c).

4.6 Role of fines

From Figs. 7-10, it is seen that for a given e or I_d , the undrained peak shear strength, steady state shear strength, and dissipated energy to the steady state decreased with an increase in contents of the fine in the mixture, showing that the introduction of fine grains to the sand could enhance the liquefaction potential, and decrease the energy dissipated during the shearing process. Therefore, it can be inferred that in case of landslides, the materials with a high content of fines may have a higher potential to suffer from liquefaction phenomena, while the less energy dissipated during the shearing process to the steady state will enable the sliding mass have greater energy for its downslope movement, resulting in higher mobility.

For the mixtures tested in this study, the density variables of equivalent intergranular void ratio (e_s^*) present good performance in integrating the undrained shear behavior of these sand mixtures containing different contents of fines. Therefore, it is reasonably inferred that these no-plastic fines work play roles as follows. Namely, in the mixtures with $FC < \text{the threshold}$, the mechanical behavior is mostly influenced by the strong contacts of coarse grains, fine grains partially come into the contacts of coarse grains and then participate in the forces transferring structure (active fine particles) (Thevanayagam et al., 2002; Thevanayagam, 2007a; Chang et al., 2019; Mehdizadeh et al., 2021; Gobbi et al., 2022b; Yazdani et al., 2022). A small number of fine grains are located in the void space between coarse grains, making no significant contributions to material resistance, and work as a filler role (inactive fine particles). With increasing fines content, more fines participate in force transferring, then increase the liquefaction potential, and decrease the energy dissipated during the shearing process.

Further, we infer that whether the increasing internal erosion of fines could promote the landslide failure and fluidization proposed in the introduction may be associated with the competition of fine grains and void ratio. The increase of erosion of fine grains could change the inner structure of mixtures, simultaneously resulting in fines content decreasing and void ratio increasing, further leading to inhibiting or facilitating

landslide failure and fluidization. On the other hand, the presence of fines could decrease permeability and retard the dissipation of PWP (Iverson et al., 2015), thus internal erosion of fines may result in the increase of soil layer permeability, which will lower the water retention of soil layers, and elevate the rainfall threshold for initiation of soil layer instability, or enhance the dissipation of generated PWP and then lower the post failure mobility of landslide materials.

5. Conclusions

To explore the effect of non-plastic fines on the undrained large displacement shearing behavior of saturated mixtures, undrained ring-shear tests were performed on mixtures of silica sand with different contents of non-plastic fines under varying densities. Based on the results, the influences of fine-grain content on the undrained shear behavior were studied. The following conclusions can be drawn:

(1) Fines content (FC) and initial density jointly affect the liquefaction behaviors. In the tested density range, liquefaction potential increases (shifting from a “dilation-contraction” to a “pure contraction” pattern in the stress path) with increasing FC. Liquefaction potential increased with decreasing initial density (increase in void ratio (e)).

(2) For each sample, peak strength (τ_p) and steady-state strength (τ_{ss}) generally increased as the sample became denser (decreasing e). Given the initial density being the same, τ_p and τ_{ss} became smaller with the increase of FC. An optimal void ratio existed where the undrained brittleness index (I_B) was the smallest for clean sand S7, M10, M20, and M30. Nevertheless, M40 showed an increasing tendency with an increasing void ratio, probably because the range of densities was relatively small.

(3) Comparing void ratio (e) and intergranular void ratio (e_s), plots of τ_p , τ_{ss} , and I_B against the equivalent intergranular void ratio (e_s^*) or equivalent intergranular density index (I_{ds}^*) moved to a narrow range, indicating e_s^* or I_{ds}^* serve as proper indicators for evaluating FC's effects on undrained shear behavior of

saturated mixtures. The τ_p showed an initial rapid decrease followed by an approximately slow descent in general with increasing e_s^* , but the corresponding τ_{ss} continuously decreased with an increase in e_s^* . There exists an optimum e_s^* at which the I_B was minimal for experiments on mixtures of M10, M20, and M30.

(4) The shear energy (W_{ss}) dissipated throughout the entire shearing process to steady-state increases in log scale as the sample density increased (decreasing e) for each sample. The dissipated energy (W_{ss}) decreased with increasing FC at the same void ratio (e). The plots of W_{ss} against e_s^* were located in a relatively narrow band, and W_{ss} in the log scale continuously decreased with increasing e_s^* .

(5) The addition of fine particles could change the inter-particle force transfer. Fine particles partially come into the contacts between coarse grains, and then join in the forces transferring structure (as active fine particles). With FC increases, more fine particles actively participate in the mechanical transferring structure and then result in increasing liquefaction potential and reducing the energy dissipation throughout the entire shearing process of mixtures, thereby facilitating the fluidization progress of landslides, making landslides more mobile and potentially more hazardous.

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Caption of Figures

Fig. 1. (a) Grain size distributions of the sample with silica sand no. 7 (denoted by S7), silica powder (denoted by SP), and mixtures of M10, M20, M30, and M40. (b), (c) Scanning electron microscope (SEM) images for S7 and SP.

Fig. 2. Properties of the mixtures of S7 with 0%, 10%, 20%, 30%, and 40% of SP by weight. (a) Permeability, (b) Maximum and minimum void ratios ($e_{\max}$ and $e_{\min}$) of the mixtures of S7 with different contents of SP.

Fig. 3. (a) Annular ring-shaped sample. (b) The configuration of the ring-shear apparatus (no size scale, after Chang and Wang, 2022).

Fig. 4. Effects of initial density on the liquefaction behaviors of the clean sand S7 (varying initial void ratio e of 0.99, 1.02, 1.07, 1.12, and 1.14, corresponding to decreasing density indices I_d of 0.28, 0.21, 0.11, 0.00, and -0.04). Density index $I_d = (e_{\max} - e) / (e_{\max} - e_{\min})$, where e is void ratio, $e_{\max}$, and $e_{\min}$ are maximum and minimum void ratio. PT: phase transformation. SSP: steady state point. (a) Shear resistance versus shear displacement; (b) Normal stress and pore water pressure versus shear displacement; (c) Effective stress paths. C: contraction; D-C: dilation-contraction.

Fig. 5. (a), (b) Results of undrained ring shear tests on S7 and M10 samples in a loose state (for S7, $e = 1.14$ and $I_d = -0.04$; for M10, $e = 1.04$ and $I_d = 0.15$). (c), (d) Results of undrained ring shear tests on M20, M30 and M40 in loose state (for M20, $e = 0.96$ and $I_d = 0.26$; for M30, $e = 0.88$ and $I_d = 0.34$; for

M30, $e = 0.81$ and $I_d = 0.45$). SR: shear resistance; PWP: pore water pressure; ESP: effective stress path;
SSP: steady state point.

Fig. 6. Results of undrained ring shear tests on S7 ($e = 0.95$, $I_d = 0.38$), M10 ($e = 0.88$, $I_d = 0.45$), M20 ($e = 0.81$, $I_d = 0.50$), M30 ($e = 0.77$, $I_d = 0.52$), and M40 ($e = 0.77$, $I_d = 0.53$) samples in the “dense” state.
PT: phase transformation. (a) Shear resistance versus shear displacement; (b) Pore water pressure versus shear displacement; (c) Effective stress paths.

Fig. 7. Undrained peak strength (τ_p) versus void ratio e (a) and density index I_d (b), intergranular void ratio e_s (c) and intergranular density index I_{ds} (d), equivalent intergranular void ratio e_s^* (e) and equivalent intergranular density index I_{ds}^* (f) for different samples, respectively.

Fig. 8. Steady-state strength (τ_{ss}) versus void ratio e (a), intergranular void ratio e_s (b), and equivalent intergranular void ratio e_s^* (c) for different samples, respectively.

Fig. 9. Brittleness index (I_B) versus void ratio e (a), intergranular void ratio e_s (b), and equivalent intergranular void ratio e_s^* (c) and for different samples, respectively.

Fig. 10. Shear energy to steady state (W_{ss}) versus void ratio e (a), intergranular void ratio e_s (b), and equivalent intergranular void ratio e_s^* (c) for different samples, respectively.

Table 1. Properties of tested samples in the ring shear tests.

Table 2. Test conditions and summarized data in the all ring shear tests.

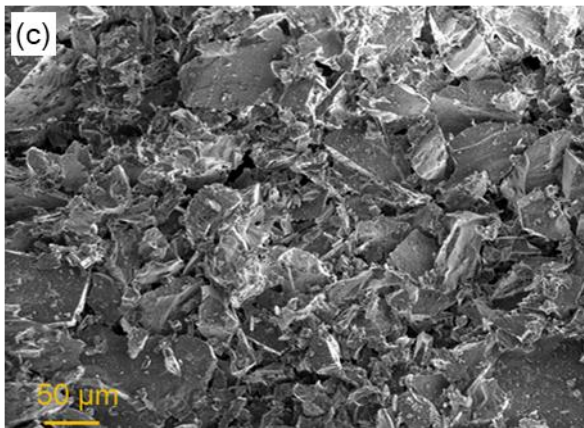
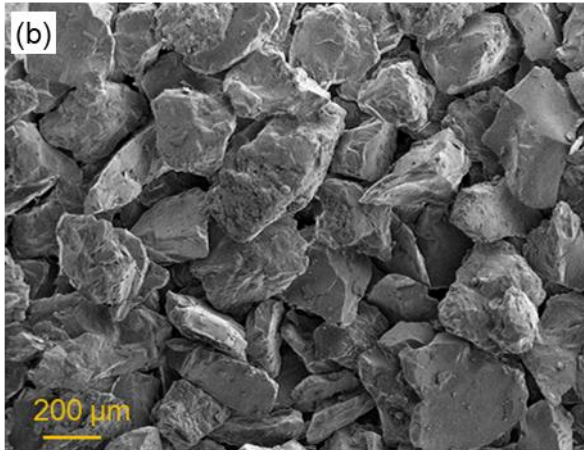
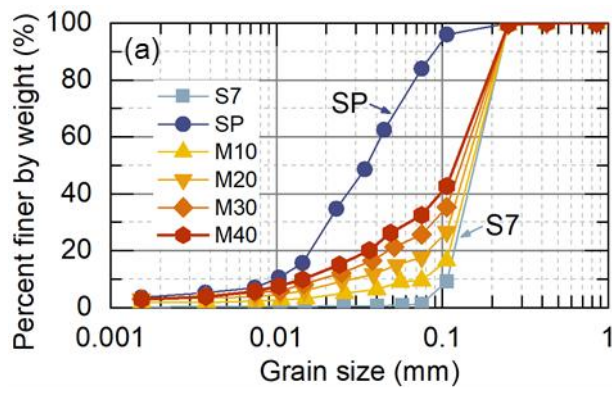


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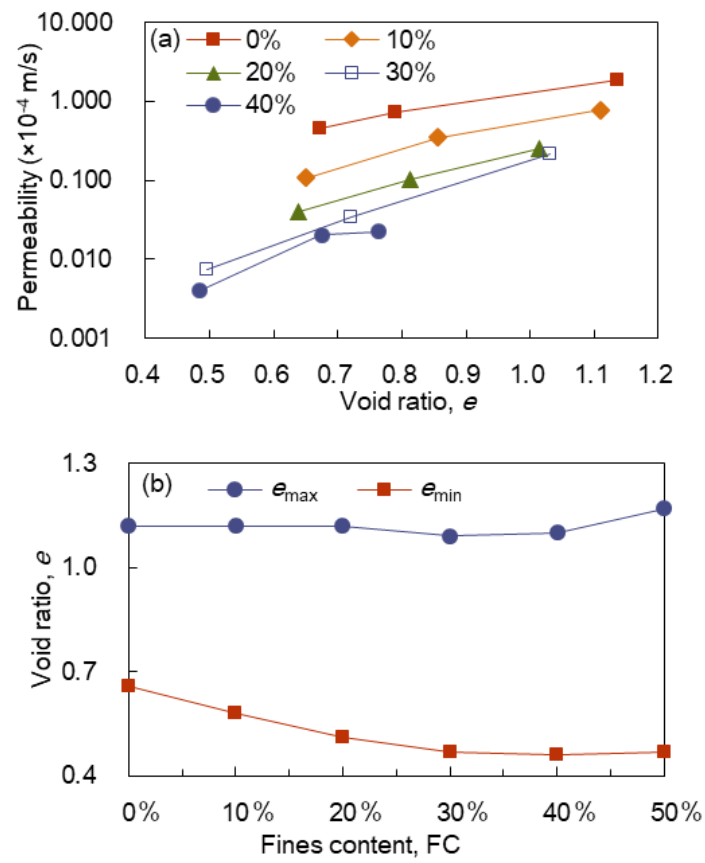


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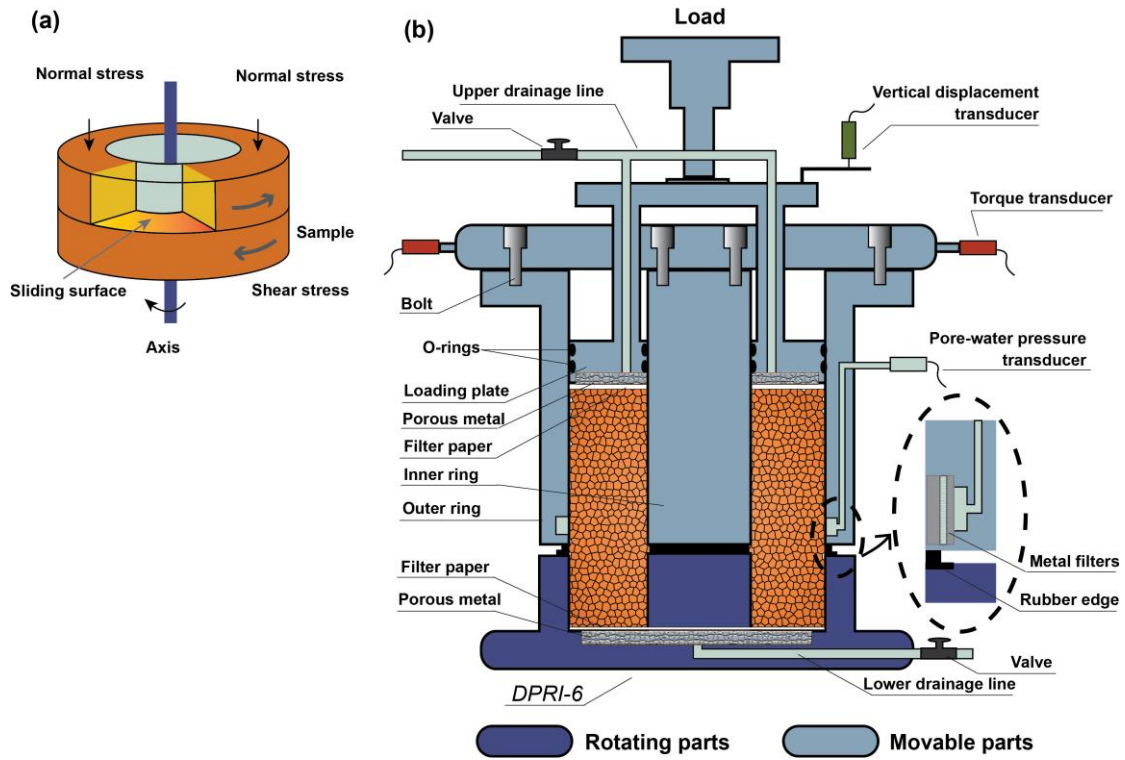


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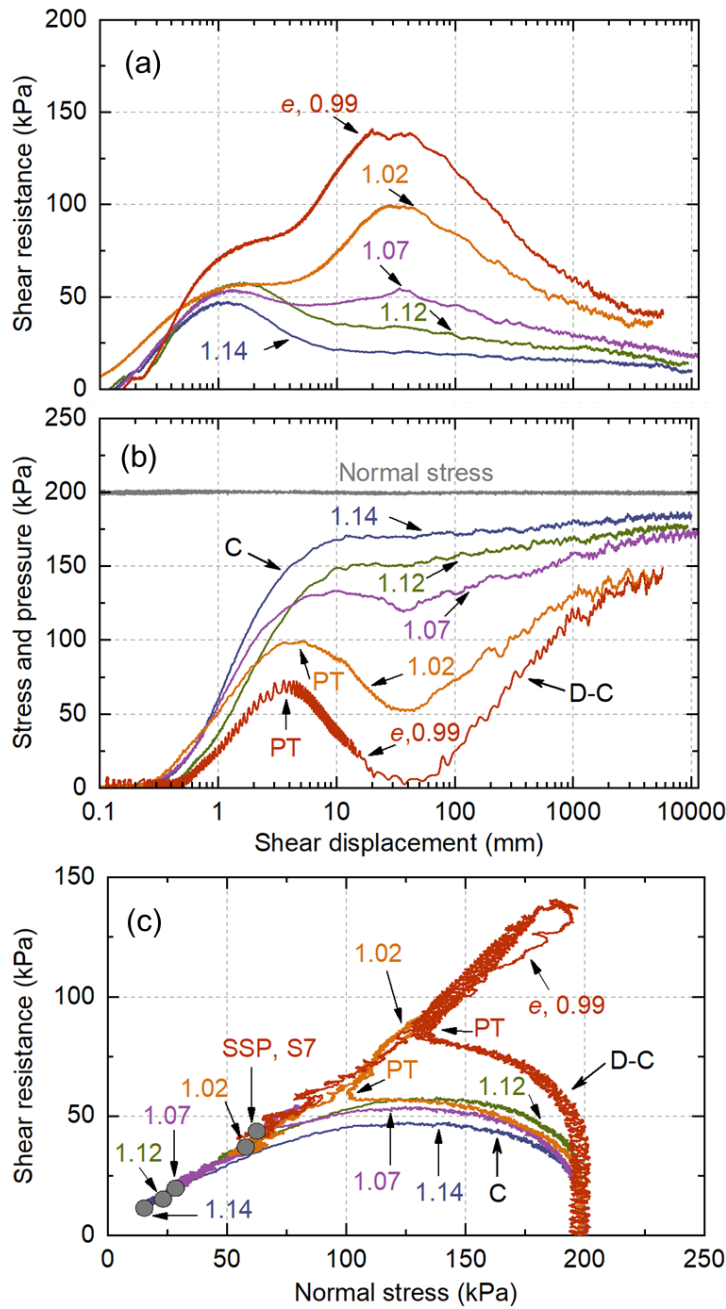


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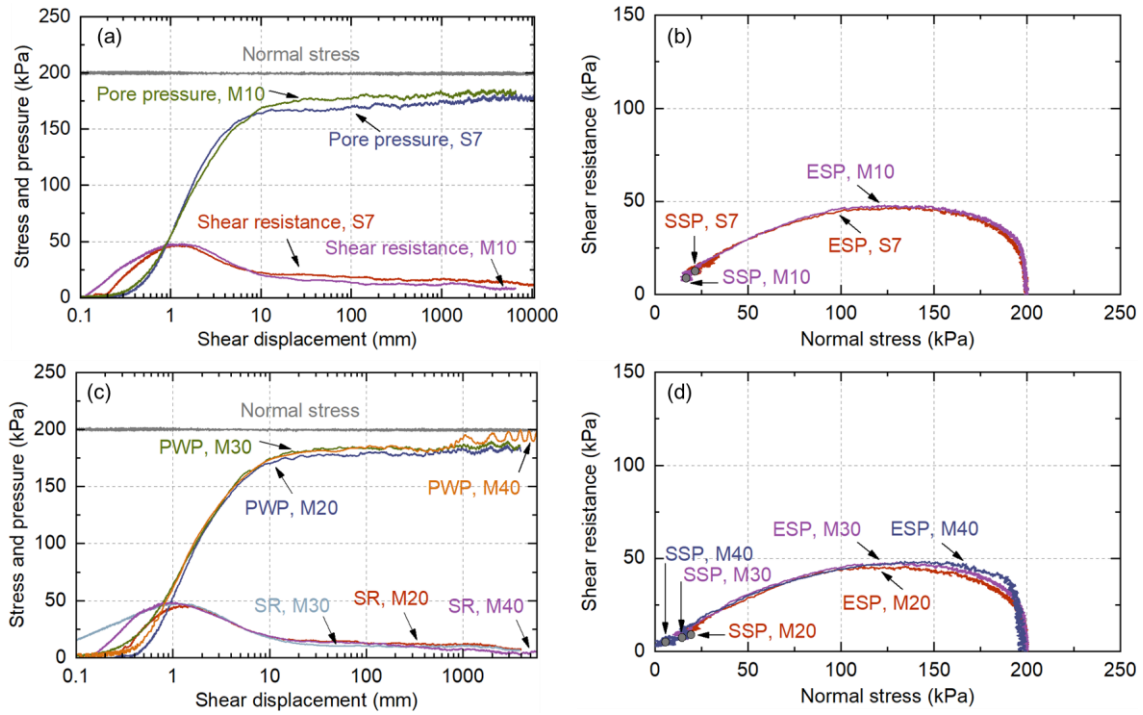


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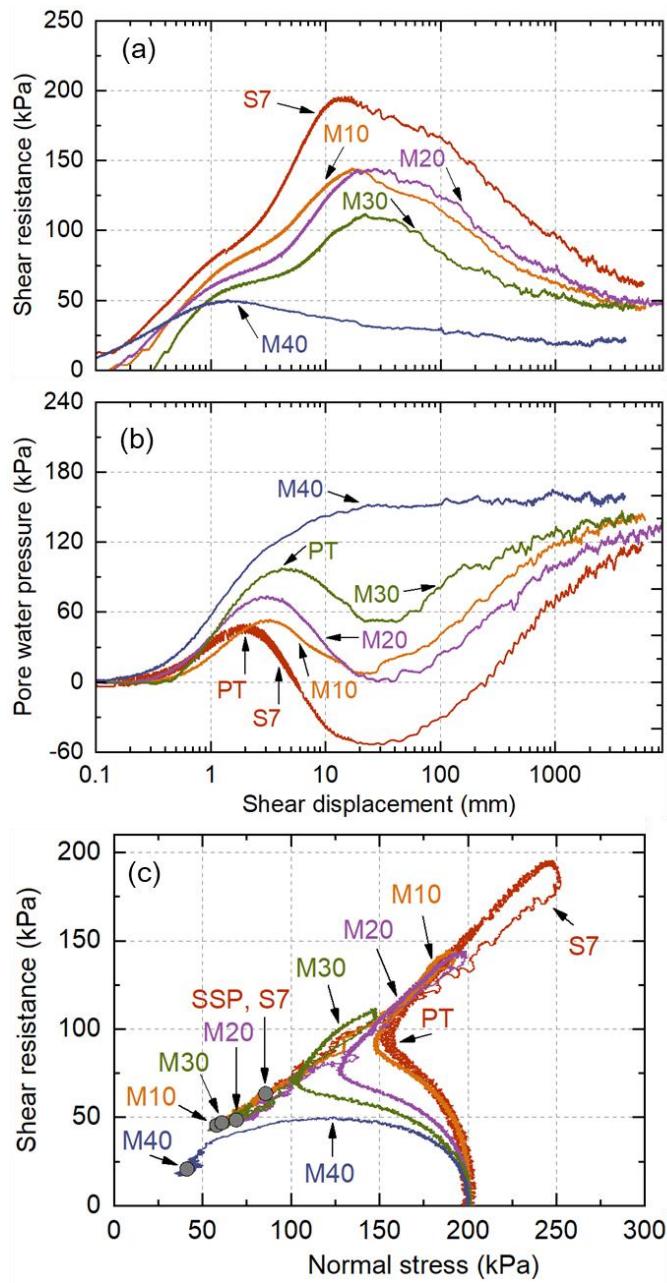


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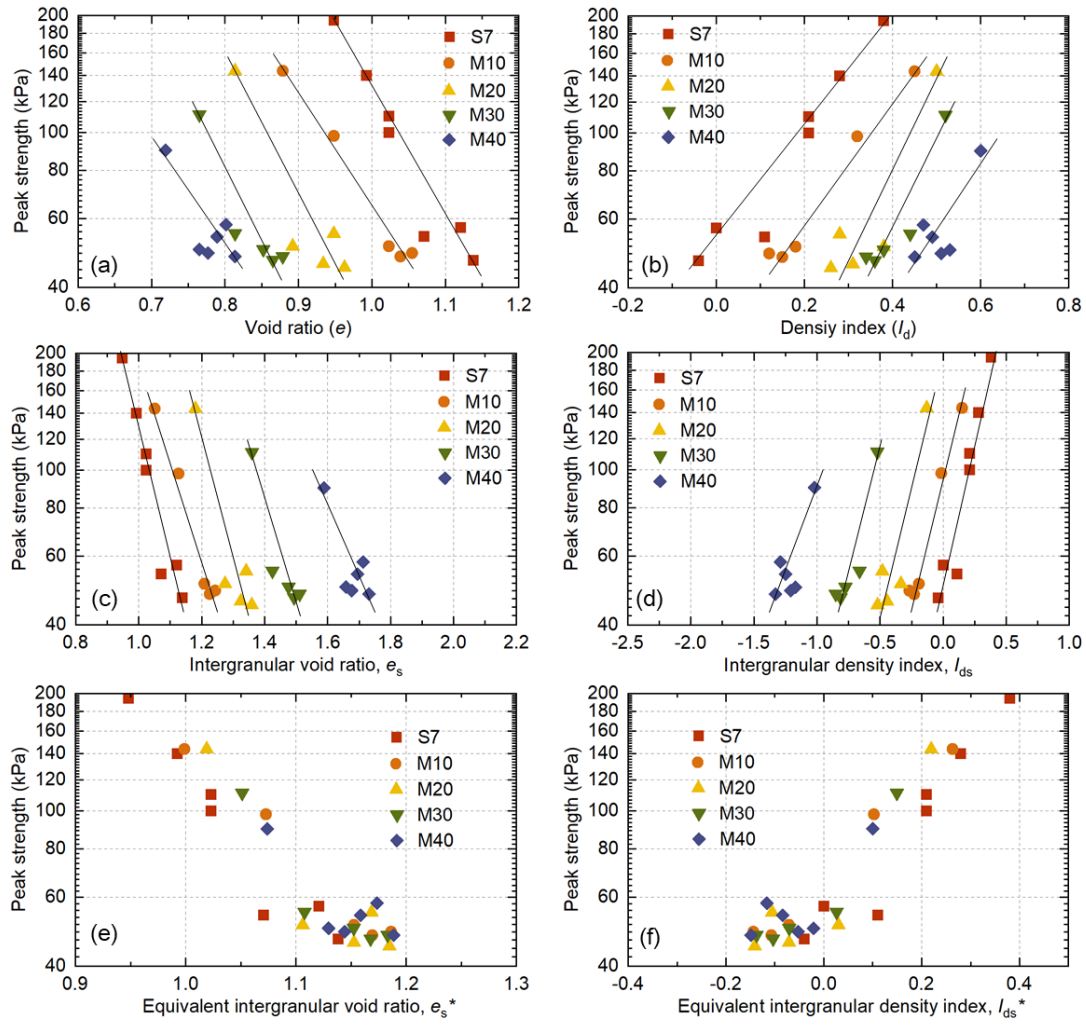


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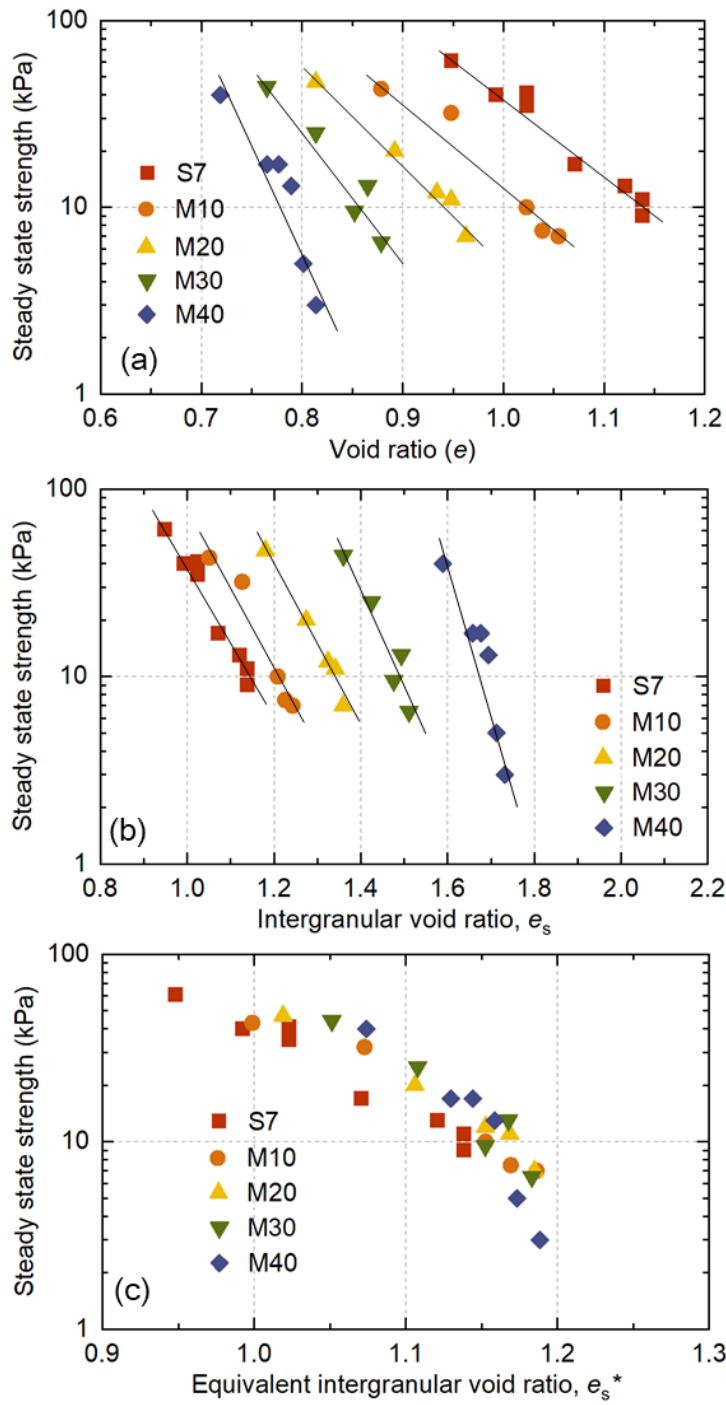


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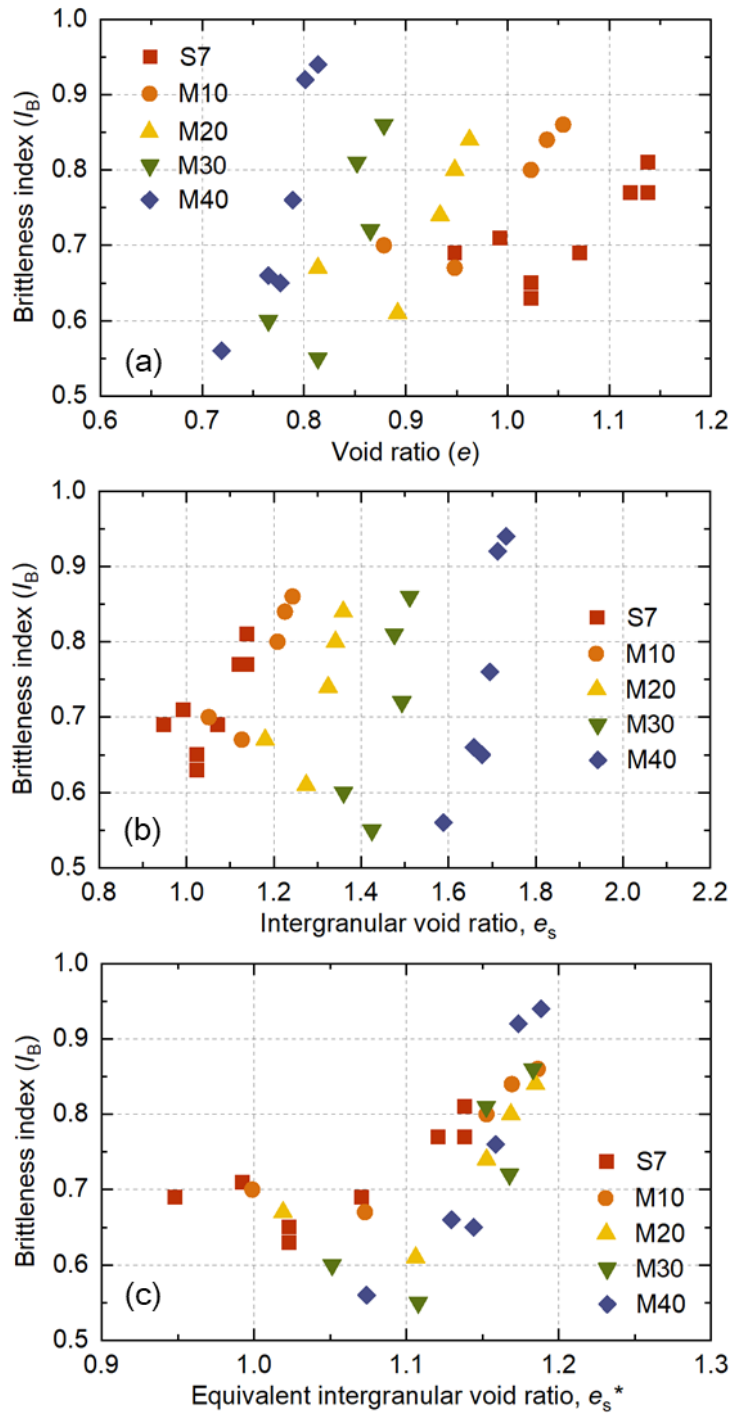


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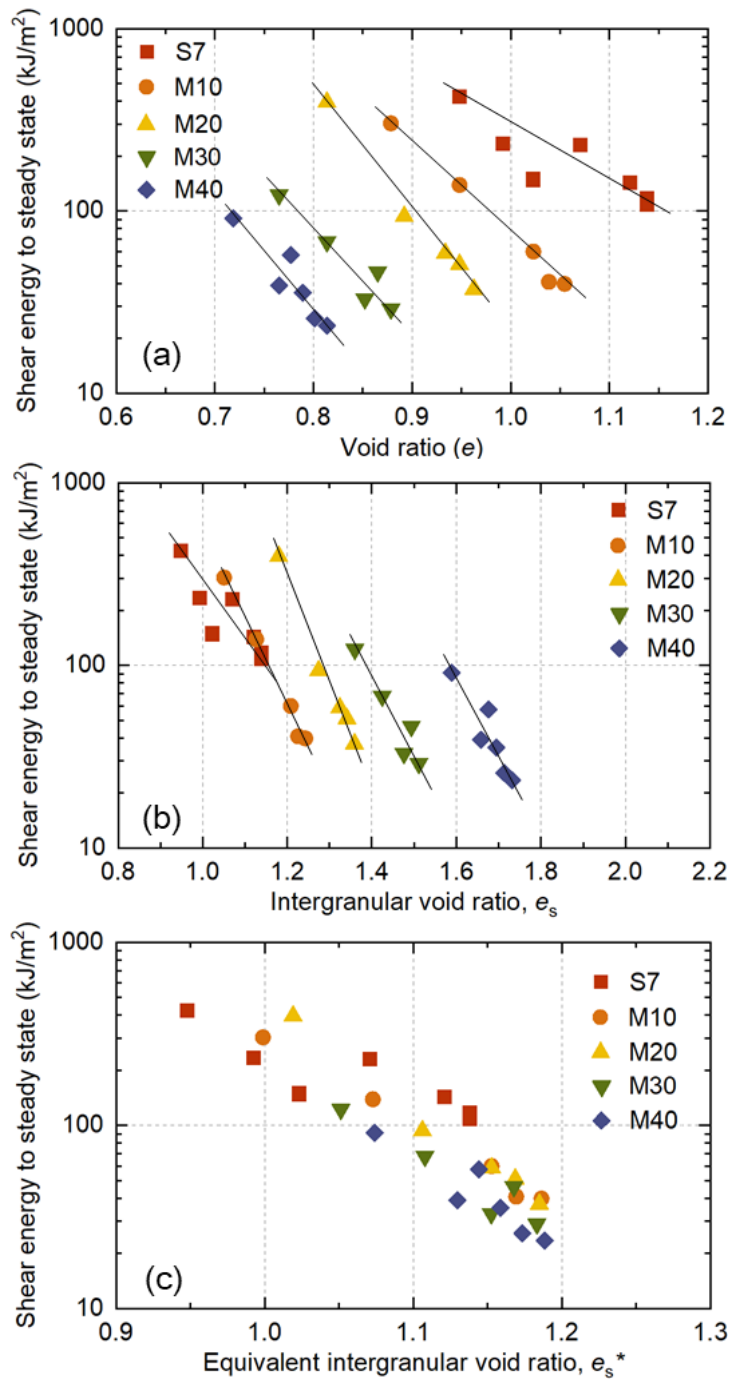


Fig. 10. Shear energy to steady state (W_{ss}) versus void ratio e (a), intergranular void ratio e_s (b), and equivalent intergranular void ratio e_s^* (c) for different samples, respectively.

745 **Table 1.** Properties of tested employed samples in the ring shear tests.

Sample	Particle density	Minimum dry density	Maximum dry density
	ρ_s (g/cm ³)	ρ_{dmin} (g/cm ³)	ρ_{dmax} (g/cm ³)
S7	2.63	1.24	1.58
M10	2.63	1.24	1.66
M20	2.63	1.24	1.74
M30	2.63	1.26	1.79
M40	2.63	1.25	1.80

746 **Table 2.** Test conditions and summarized data in the all ring shear tests.

Test. no.	Sample	Consolidation state				State value					Behavior	Note
		ρ_d	I_d	e	B_D	τ_p	τ_{ss}	I_B	W_{ss}	l_{ss}		
T-1	S7	1.23	-0.04	1.14	0.96	47	11	0.77	117111	8234	C	Fig. 5
T-2	S7	1.23	-0.04	1.14	0.96	47	9	0.81	108203	8139	C	Fig. 4
T-3	S7	1.24	0.00	1.12	0.96	57	13	0.77	142883	7891	C	Fig. 4
T-4	S7	1.27	0.11	1.07	0.99	54	17	0.69	229951	9959	D-C	Fig. 4
T-5	S7	1.30	0.21	1.02	0.95	110	41	0.63	150099	2820	D-C	
T-6	S7	1.30	0.21	1.02	0.98	100	35	0.65	147477	3244	D-C	Fig. 4
T-7	S7	1.32	0.28	0.99	0.96	140	40	0.71	233697	4301	D-C	Fig. 4
T-8	S7	1.35	0.38	0.95	0.96	194	61	0.69	423413	5213	D-C	Fig. 6
T-9	M10	1.28	0.12	1.05	0.96	49	7	0.86	39841	3885	C	
T-10	M10	1.29	0.15	1.04	0.96	48	7.5	0.84	40837	3711	C	Fig. 5
T-11	M10	1.30	0.18	1.02	0.96	51	10	0.80	59919	4396	C	
T-12	M10	1.35	0.32	0.95	0.97	98	32	0.67	138941	3346	D-C	
T-13	M10	1.40	0.45	0.88	0.95	144	43	0.70	302042	5440	D-C	Fig. 6
T-14	M20	1.34	0.26	0.96	0.99	45	7	0.84	37229	3619	C	Fig. 5
T-15	M20	1.35	0.28	0.95	0.96	55	11	0.80	51056	3708	C	
T-16	M20	1.36	0.31	0.93	0.97	46	12	0.74	58634	3874	C	
T-17	M20	1.39	0.38	0.89	0.99	51	20	0.61	93532	3906	D-C	
T-18	M20	1.45	0.50	0.81	0.97	144	47	0.67	395846	6834	D-C	Fig. 6
T-19	M30	1.40	0.34	0.88	0.96	48	6.5	0.86	29001	3172	C	Fig. 5
T-20	M30	1.41	0.36	0.87	0.99	47	13	0.72	46330	2789	C	
T-21	M30	1.42	0.38	0.85	0.95	50	9.5	0.81	32945	2748	C	
T-22	M30	1.45	0.44	0.81	0.96	55	25	0.55	67750	2090	D-C	
T-23	M30	1.49	0.52	0.77	0.99	111	44	0.60	122444	2199	D-C	Fig. 6
T-24	M40	1.45	0.45	0.81	0.98	48	3	0.94	23539	3871	C	Fig. 5
T-25	M40	1.46	0.47	0.80	0.98	58	5	0.92	25780	2749	C	
T-26	M40	1.47	0.49	0.79	0.98	54	13	0.76	35553	2150	C	
T-27	M40	1.48	0.51	0.78	0.97	49	17	0.65	57411	2630	C	
T-28	M40	1.49	0.53	0.77	0.96	50	17	0.66	39103	1836	C	Fig. 6
T-29	M40	1.53	0.60	0.72	0.95	90	40	0.56	91088	1858	D-C	

747 Density index, I_d ; Void ratio, e ; Peak strength, τ_p (kPa); Steady state strength, τ_{ss} (kPa); Brittleness index, I_B ; Shear energy to steady

748 state, W_{ss} (J/m²); Shear displacement to steady state, l_{ss} (mm); Contraction, C; Dilation-contraction, D-C.