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Assessing the influence of non-plastic fines on rainfall-induced landslide initiation and propagation: flume test findings

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Abstract:

Numerous fluidized landslides with long runout distances and fast movements have been triggered by heavy rainfall under climate change. Non-plastic fines are frequently involved in many fluidized landslides. However, the role of fines in the initiation and movement of landslides has not been well understood. This study aimed to investigate the role of non-plastic fines in the initiation and movement of rainfall-induced landslides in a flume test setup. The experiments were conducted using mixtures of silica sand ($D_{50} = 0.16$ mm) with different non-plastic fines ($D_{50} = 0.035$ mm) content under different initial densities. The results demonstrated that the behavior of landsliding greatly depends on fines content (FC) and initial packing density (presented by density index, I_d) of the soil layers in the flume. The landsliding phenomena could be classified into five failure modes, including slow and fast retrogressive individual sliding, sudden multiple sliding, and deep-seated and shallow fluidized sliding. As the FC increased or I_d decreased, the landsliding behavior gradually evolved from slow retrogressive individual sliding to shallow fluidized sliding. For the tests with $I_d \geq -0.06$, the peak velocity (V_p) of the landsliding increased continuously with FC. Conversely, for the tests with $I_d \leq -0.19$, an optimal FC of 30% was identified at which V_p reached its maximum. Additionally, for $FC \leq 10\%$, V_p decreased with the increasing I_d , while for $FC \geq 20\%$, an optimal I_d existed at which V_p reached its maximum for a given FC, and the value of optimal I_d ($-0.2 \sim -0.04$) varied with FC ($20\% \sim 40\%$). The peak pore-water pressure (PWP) (u_p) generally increased with the increase of FC. Furthermore, an optimal I_d existed at which the PWP built up after landsliding occurrence (Δu) reached its maximum. The equivalent intergranular void ratio (e_s^*) and equivalent interfine void ratio (e_f^*), which are calculated by considering both FC and void ratio, are found to be appropriate indices for evaluating and explaining the role of fines in the observed landsliding behavior. In conclusion, the addition of non-plastic fines can change the structure of test materials and result in different failure modes and mobility of landslides by affecting both interparticle contacts and PWP responses. These findings provide insights into the mechanisms of landsliding behavior and have implications for the assessment and mitigation of

landslides in geotechnical engineering.

Keywords: Geologic hazards; Climate change; Fines; Flume test; Pore pressure; Sliding velocity

1. Introduction

Rainfall-induced landslides, especially those fluidized ones, are a major concern due to the potential significant threat they pose to human lives and properties (Hadji et al., 2013; Kallel et al., 2018). Notable examples of such disasters include the 1999, 2009, and 2014 landslides in Hiroshima, which resulted in extensive damage to densely populated areas and a significant number of casualties (Wang et al., 2003; Tsuchida et al., 2014; Wang et al., 2015). Over the years, numerous studies have been carried out to investigate the initiation and motion mechanisms of fluidized landslides, revealing that liquefaction is the main cause of their rapid movement (Castro and Poulos, 1977; Ishihara, 1993; Wang and Sassa, 2001; Wang et al., 2003).

Liquefaction is a process whereby soils experience a build-up of high pore-water pressure (PWP) and a significant loss of shear resistance. This phenomenon has been widely studied since the 1964 Niigata earthquake, which triggered widespread liquefaction and resulted in significant damage to local infrastructure (Seed and Idriss, 1967; Ishihara and Koga, 1977; Seed, 1979; Ishihara, 1993). It is known that liquefaction can be caused by dynamic loading (e.g., earthquakes), static loading (e.g., rainfall, snowmelt, and groundwater variation), or other factors (Castro, 1969; Hutchinson and Bhandari, 1971; Castro and Poulos, 1977; Sassa et al. 1996; Sassa, 1998).

Despite extensive research, most of the understanding of the liquefaction mechanism has been obtained from triaxial tests using small specimens under idealized conditions. To gain a better understanding of the fluidized landsliding behavior, many researchers have employed flume tests to trigger laboratory-scale mass

movement. These studies have revealed that landslide movements involve the generation of high pore pressure, which depends on the initial soil porosity and the soil deformation rate (e.g., Eckersley, 1990; Iverson et al. 2000; Wang and Sassa, 1998, 2001, 2003; Moriwaki et al., 2004; Olivares and Damiano, 2007; Zhang et al., 2020).

The studies mentioned above have significantly improved our understanding of how fluidized landslides begin and move. However, most of these studies have focused on clean sands, leaving little research on the effects of adding fine materials like silt and clay. Although a few exceptions exist, such as studies by Eckersley (1990), Ishihara et al. (1990), Thevanayagam (1998), and Wang and Sassa (2003).

It's common to find fine materials mixed in with fluidized landslide debris. These fines can either be originally present in the landslide materials (Ishihara et al, 1990; Fourie et al., 2001; Wang et al., 2015; Yin et al., 2016; Zhang et al., 2017), or they can result from the fragmentation of the landslide material during sliding (McSaveney and Davies, 2007; Perinotto et al., 2015; Delannay et al., 2017; Dufresne and Dunning, 2017). These fine materials may play a significant role not only in the initiation and movement of the original landslide but also in secondary landsliding that occurs on the initial landslide deposits. For example, the 2008 Wenchuan earthquake caused numerous landslides, leading to the formation of massive landslide deposits in the mountainous areas (Huang et al., 2011; Zhang and Zhang, 2017; Hu et al., 2018; Fan et al., 2019). After the earthquake, numerous secondary landslides were triggered on these deposits, with non-plastic fines being significantly involved (Fan et al., 2017; Hu et al., 2018; Li et al., 2021; Wasowski et al., 2021).

The importance of fines in the initiation and movement of landslides has been established by field evidence and laboratory experiments. However, there is still a limited and sometimes conflicting understanding of the role of non-plastic fines in this progress. Some studies suggest that the presence of non-plastic fines could increase the liquefaction resistance of sands (Pitman et al., 1994; Amini and Qi,

2000; Kwa and Airey, 2017), while others show a decrease (Yamamuro and Lade, 1997; Murthy et al., 2007; Monkul et al., 2017). To classify the opposite effects of fines on liquefaction, some studies adopt new concepts, such as fines content threshold (FC_{th}) or limiting fines content (LFC) (Thevanayagam et al., 2002; Bahadori et al., 2008; Chen et al., 2020; Rahman and Sitharam, 2020; Rahmani and Naeini, 2020). It is reported that fines can reduce the collision rate between coarser particles and form a dust suspension system with a fine-grained matrix during landslide motion, allowing the landslide to travel further (HSÜ, 1975; Perinotto et al., 2015). Further, some studies examined whether fine particles could promote the stability of a slope. Cui et al. (2017) suggested fine particle migration in soil may contribute to soil failure and debris flow initiation, while Hu et al. (2017, 2018) stated that the fines could be eroded and transported within large particle pores, which can produce the grain coarsening, elevate the permeability of slope, and the strengthen the stability against rainfall. On the other hand, Furuya et al. (1999) pointed out that fine materials can be eroded and transported by groundwater flows, leading to the production of enlarged voids and instability of the landslide mass, and resulting in the creeping movement of a large-scale landslide in crystalline schist, Southwestern Japan.

This study aims to investigate the impact of non-plastic fines on rainfall-induced landslides. Considering that much knowledge on the liquefaction behavior of sandy materials is based on simple element tests on small specimens under idealized conditions (such as fully saturated, undrained, and stress isotropy), while fluidized landsliding phenomena may involve in the movement of partially saturated materials under naturally drained condition, in this study we employed a medium-sized flume. It is noted that although this kind of flume tests has the shortage in simulating the landslides under the same stress level as natural landslide cases, it has been widely used to trigger small-scale landslides under laboratory condition, because it has the advantages, such as idealizing the natural physical processes, reducing the complexity faced by field experiments and almost field-scale landsliding models, isolating the effects of key variables, and

allowing for real-time and in-situ detecting of the involved processes (Moriwaki et al., 2004; Ochiai et al., 2004; Dufresne, 2012; Iverson, 2015; Brož et al., 2020; Zorn et al., 2020; Baselt et al., 2021). We conducted the tests on sand mixtures with varying amounts of non-plastic fines and different initial densities. To obtain accurate data, we measured the pore-water pressure, tilting, and displacement using various transducers, and recorded the landsliding behavior through cameras. Our findings shed light on how non-plastic fines affect pore-water pressure and landslide movements. We also analyzed the role of fines by introducing the concepts of equivalent intergranular void ratio (e_s^*) and equivalent interfine void ratio (e_f^*).

2. Methods

2.1 Material

In this study, we used mixtures of silica sand No.7 (S7) with different contents of silica powder #100 (SP). S7 and SP were chosen as the coarse and fine materials, these materials have accessibility in industrial sand and reproducibility in laboratory experiments. Both S7 and SP comprise quartz and feldspar. SP originates from silica sand by mechanical grinding, which could be classified as rock flour, belonging to the non-plastic fines. To study the effect of non-plastic fines, the percentage of SP in the mixture was varied as 0%, 10%, 20%, 30%, and 40% by weight (hereinafter these mixtures are termed S7, M10, M20, M30, and M40, respectively). Fig. 1a presents the grain size distributions, while Figs. 1b and 1c are SEM images of S7 and SP, respectively. The mean grain sizes (D_{50}) are 0.16 mm and 0.035 mm for S7 and SP, respectively. The permeability, maximum and minimum void ratios ($e_{\max}$ and $e_{\min}$) of tested samples were measured following the Japanese Geotechnical Society standards (Japanese Geotechnical Society, 2011) and present in Fig. 2. Based on void ratio indices, a clear transitional behavior, with estimated fines content threshold (FC_{th}) around 30~40% is observed. The value of $e_{\max}$ reaches the lowest point at 30% earlier than that of $e_{\min}$ at 40%, although the $e_{\min}$ drops faster until reaching FC_{th} , then afterward grows more slowly

than $e_{\max}$. Other density properties of these samples are listed in Table 1.

2.2 Experimental setup

As mentioned above, the flume test had been widely used as an effective physical modeling method to simulate laboratory-scale mass movement, including landslide motion, fluvial erosion, debris flow, and debris avalanche evolution and deposition (Wang and Sassa, 2001; Okura et al. 2002; Chen et al., 2010; Iverson et al., 2011; de Haas et al., 2015; Lamb et al., 2015; Delannay et al., 2017; Hu et al., 2017; Sasahara and Sakai, 2017; Lube et al., 2019; Yang et al., 2019; Xie et al., 2020; Baselt et al., 2021; Ng et al., 2022). In this study, we employed the same flume as used by Lourenço et al. (2006), and followed the same procedures as introduced in Wang and Sassa (2001) for the preparation of the sample. Fig. 3 shows the employed experimental apparatus (Figs. 3a and 3b) and arrangement of the samples together with the setup of the monitoring system (Figs. 3c and 3d). The flume is 200 cm in length, 25 cm in width, and 40 cm in height. The slope of the flume is changeable following the test requirement and kept at 25 degrees in all the tests for this study. The side walls of the flume are made of transparent plexiglass plates, and the base of the flume was covered by a rubber sheet glued with sand grains to ensure the same friction as that between the grains themselves.

As well known, the initial conditions of a physical model, such as rainfall intensity, bulk density, initial moisture content, and model shape, are essential in laboratory flume experiments (Govers et al., 1990; Pelletier, 2003; Jomaa et al., 2012; Cogan and Gratchev, 2019). In our test, the sample in the flume was designed as a right trapezoid with the size of 87 cm in upper base length, 120 cm in lower base length, and 15 cm in height (as illustrated in Fig. 3c). It is noted that Peng and Zhang (2012) proposed two dimensionless coefficients, H_d/W_d and $(V_d)^{1/3}/H_d$ (here H_d is height, W_d is width and V_d is the volume of model), to check the representativeness of the physical model for the landslides in natural-scale. The H_d/W_d

and $(V_d)^{1/3}/H_d$ were suggested to be among 0.02~1 and 1~10 (Zhou et al., 2019; Zhou et al., 2022), respectively for the representativeness. In our tests, all the samples were shaped the same dimension, which results in the values of H_d/W_d and $(V_d)^{1/3}/H_d$ being 0.6 and 2.4, correspondingly. Namely, the physical model in our tests can represent natural-size landslide events. Different sensors, including ten pore water pressure (PWP) transducers, seven tilt sensors, and displacement-measuring sensors, were installed at ten locations (P1-P10). Among them, six PWP transducers were installed within the soil mass at different depths (5 and 10 cm from the bottom of the flume), and four PWP transducers were at the bottom of the flume. Seven tilt sensors were used to monitor the rotation inside the soil mass. At P6, a styrene foam ball of 2 cm in diameter was used to capture the sliding displacement of soil mass. This foam ball was connected with a linear displacement sensor that was placed at the upslope of the flume through a stiff wire. A laser displacement sensor with a precision of 0.003 mm was used to capture the movement of the foam ball precisely by measuring the displacement of the fixed target connecting with the ball. A 150 g counterweight was utilized to balance the pulling resistance of the linear displacement sensor. Two sprinklers were mounted above the flume to simulate rainfall (Fig. 3c). Two video cameras were installed to observe the sliding behavior from both the side and the downstream end of the flume.

2.3 Test procedures and conditions

The samples were prepared at an initial gravimetric water content of 10% by adding water to the dried samples. Pore water pressure transducers and displacement sensors at the bottom of the flume (P4-P7) were installed first, and then, the samples were placed in the flume by layers. To make the samples uniform, the samples were placed in layers (parallel to the base of the flume) with a thickness of 2 cm, and then were tamped successively. To ensure the free rotation of the tilt sensor at the flume bottom (P4-P7), we installed four tilt sensors close to the base at the same locations as the PWP sensors (P4-P7) after placing and tamping

the first thin soil layer. Other transducers were installed successively when the soil mass was prepared. Finally, we shaped the soil layers as shown in Fig. 3c and inclined the flume to 25 degrees. Because the tilt sensor can measure both the clockwise and counterclockwise rotations, in the study, the increase/decrease in the tilting angle presents the forward (clockwise) / backward (counterclockwise) rotation of the soil mass. We offset the initial value of the tilt sensors, the initial value was defined as 0 after lifting the flumes. Thus, the tilting angle is the angle measured from the experimental set-up.

After the setup of the flume, we turned on the sprinklers to simulate rainfall-induced landslides. The rainfall intensity was kept at 100 mm/h, a value that was used in Wang and Sassa (2001, 2003), and is also similar to those rainfall events that triggered shallow landslides in volcanic areas, such as in Aso and Izu-Ohshima events (Yano et al., 2019; Tsunetaka, 2021; Wang et al., 2019). During the tests, displacements, tilting, and pore-water pressure were measured at a sampling frequency of 20 Hz. The experimental landslide event was recorded by video cameras and analyzed by means of a particle image velocimetry (PIV) approach developed by Thielicke and Stamhuis (2014). For the slow landslide, the velocity was calculated by displacements captured by the linear transducer. Due to the limitation of linear transducer capacity, the velocity was calculated by particle image velocity (PIV) analysis within the visible range by the video-recording system for those landslides with long traveling distances, the frame rate used for the PIV analysis was 10 Hz. Table 2 summarizes the initial test conditions and some test results for all the tests. For each sample, we varied the initial dry density by changing the tamping method, and the concept of density index $I_d = (e_{\max} - e) / (e_{\max} - e_{\min})$ was employed to present the density (Spence and Guymer, 1997), where e is void ratio, $e_{\max}$ and $e_{\min}$ are maximum and minimum void ratio.

3. Results

3.1 Observed landsliding phenomena and failure modes

Examining the video recordings revealed that the landsliding phenomena in all the tests mainly involved four stages, i.e., wetting, precursory slides, main failure, and successive movement after the main failure. In the wetting stage, the soil mass became wet after rainfall, and the wetting front gradually went downwards to the flume base, which was approximately parallel to the base of the flume. Then, in the stage of precursory slides, small sliding events occurred at the toe part successively and slowly. Thereafter, the main failure occurred with the soil layer. In this stage, tilting and displacement measurements showed significant variations, and the sliding behavior differed among tests, which greatly depended on the initial density and the fine content in the mixtures. Fig. 4 summarizes the observed failure modes based on real images of the experiment in the main failure stage for all the tests. The failure modes were classified into five types based on the global morphology, volume and velocity of the events, and the time intervals between landsliding events. Type I presents slow retrogressive individual sliding that is characterized by inconsistent retrogressive slides. In this type, individual soil blocks slid slowly with a small volume following the former block, and the duration of each sliding event is normally longer than ten seconds. Type II is fast retrogressive individual sliding, which presents similar sliding behavior as Type I, but the individual block triggered a longer distance within a shorter duration (normally less than 1 second). Type III is termed as sudden multiple sliding where the displaced soil mass disintegrated into several blocks almost simultaneously within very short periods (about 1 s), and then these blocks reposed but finally slid downslope slowly. Type IV is characterized by deep-seated fluidized sliding, in which the displaced materials behaved rapid fluid-like movement with a large volume and a deep sliding surface. Type V is shallow fluidized sliding, which is characterized by rapid flow-slide with a small volume and a shallow sliding surface. In addition, evident soil compaction preceding the main failure was observed in the initial

stage of the tests in Type V.

3.2 Tilting of soil mass in different failure modes

The above-mentioned five types of failure modes involve complex internal deformation within the soil mass. Compared to the bottom measurements of displacement, the utilization of tilt sensors in different locations could provide detailed information on the sequence of internal rotation and translation during the landslide for a better understanding of the initiation and movement of the landslides. Fig. 5a presents the time series data of the tilting angle for a test on S7 (T-4 in Table 2) as an example of tests showing retrogressive sliding. Figs. 5b-5e provide enlarged views for those data inside the dashed windows shown in Fig. 5a. The soil mass at location P7 showed slow forward rotation with an increase in the tilting angle at 1420 s (Fig. 5a-1), and after a temporal reduction (backward rotation), it was followed by a quick forward rotation change at 1489 s with the occurrence of a toppling failure on the toe part. The tilting angle of the soil mass at P6 started to increase at 1566 s (Figs. 5a-2 and b) but finally showed a sharp decrease together with the soil mass at P3 at the moment of 1603 s (Figs. 5a-3, b, and c) with the occurrence a rotational landslide. Consequently, tilt sensors at P2 and P5 started slow rotation at 1727 s and 1769 s but rotated together at 1769 s with the occurrence of a new rotational sliding event (Figs. 5a-4, a-5, c, and d). The soil mass located at P4 and P1 began to tilt at 2087 s and 2094 s slowly and finally showed quick variation in the tilting angle with the occurrence of the third event of a rotational landslide.

Although the surficial soil mass may show different tendencies in the variation of tilting angle, the soil layers at the bottom show temporal increase before the occurrence of each rotational landslide event, suggesting the presence of precursory deformation preceding the main failure event. Details on this will be discussed in a later section.

The variations of tilting angles in the soil layers suffered from the failure of Type IV (deep-seated fluidized sliding) are exemplified by test T-23 (M40) in Fig. 6. Fig. 6a presents the responses of the tilt sensors

for a long duration, while Figs. 6b-6h for clear views of the data within short durations. The landsliding sequences and individual events captured by the tilting sensors are largely different from those shown in Fig. 5. As shown in Figs. 6(a-1)~6(a-3) and 6b-6d, half of the soil mass on the downslope side showed very rapid motion simultaneously within a short period (1528.20-1528.25 s), which could be identified by tilting variations at P7 at 1528.20 s, and P3 and P6 (right-middle part) at 1528.25 s. This first event is illustrated by Type IV in Fig. 4. After detachment of the soil mass on the downslope half of the model, the middle portion (P2 and P5) slid fast at about 1539.65 s, which was identified as the second landslide event through the video recordings (Figs. 6(a-4), 6(a-5), 6e and 6f). With continuous rainfall, the third event occurred two seconds after the second event, which was recorded by the tilt sensors at P1 and P4 (Figs. 6(a-6), 6(a-7), 6g, and 6h). It is worth noting that several late events were also captured by the tilt sensors that were placed on the path of the movements, as revealed by the sharp variations (Figs. 6(a-1)-6(a-7)). The simultaneous responses of the tilt sensors demonstrate fast initiation and movement of the landslide material and also indicate that a large volume of the material was involved during a short period (1528.2-1541.6 s). In contrast to the precursory deformation in Test T-4 (cf. Figs. 5b and e), the precursory tilting commonly shows a trend of backward rotation (Figs. 6b-h). It is also noted that all the tilting angles shown in Figs. 6b-6h presented backward rotation by showing decreases in the tilting angles.

3.3 Pore water pressure responses in different failure modes

Because the main failure usually involved the movement of soil mass in the middle portion of the landslide model, in this study we focused on examining the pore-water pressure responses along with the measurements at locations around P6 and P9 (cf. Fig. 3d). The results revealed that the response of pore-water pressure depends on the tested sample and the initial density of the soil layer, and also on the location of the sliding surface at each landsliding event. This could be exemplified by the results of tests T-25 (shown in Fig. 7) and T-23 (shown in Fig. 8), in which the soil masses showed failure modes of Type I and Type

IV, respectively. As shown in Fig. 7a, the PWP at P6 gradually increased at about 1372 s and reached a constant value transitorily, and then showed a small build-up, finally, it sharply decreased with the runaway of the displaced soil masses. The PWP at P9 gradually increased first and then decreased. The tilting angle showed a slow increase after the initial build-up of PWP and then fluctuated in the sliding process. The shear displacement measured by the laser sensor increased continuously with time, while the sliding displacement measured by the linear transducer showed a quick increase after the sliding occurred. For a detailed view, the time series data shown in Fig. 7a for the duration of 1515 ~ 1545 s are presented in Fig. 7b correspondingly. PWP at P6 increased with the soil rotation first, then decreased with the occurrence of the accelerating movement that could be identified by the tilting angle and the shear displacement. However, the PWP at P9 decreased from its peak. It is also seen that the tilting angle and shear displacement showed the same increasing tendency with time in the short period before the main failure occurred at about 1543s. Once the main failure was initiated, the tilting angle decreased sharply, indicating that the rotation direction of the soil mass was changed, while the linear transducer arrested this rapid acceleration and showed an instantaneous increase in displacement. These results indicate that tilting is more sensitive than macro sliding, namely, the small internal rotation that occurred before macroscopic sliding behavior and was captured by the tilt sensor may provide more promising precursory signals for the prediction of the occurrence of the major failure.

For test T-23, Fig. 8a presents the measurements at P6 and P9 from 1350 s and 1550 s, while Fig. 8b is the enlarged view of the data for the duration of 1525 s ~ 1530 s to provide a clearer view of the rapid motion. As shown in Fig. 8a, the PWP at P6 gradually increased slightly and then sharply decreased with the sudden change in the title angle and displacements, namely, with the occurrence of the fluidized sliding. However, during this period, PWP at P9 showed a sharp increase. Detailed examination of the time series data shown in Fig. 8b suggests that the PWP at P6 had already started to decrease before the onset of tilting,

but PWP at P9 showed a temporary decrease with the onset of the backward tilting (decrease in tilting angle), and then increased when the tilting shifted from backward to forward tilting. It is noted that we checked the data for all the tests and found that in most cases, the PWP at P6 (bottom) showed increases before the onset of landsliding, but decreased quickly when the main failure event occurred. However, during the main failure, the PWP at P9 (top of P6) increased.

3.4 Displacement and velocity of landslides in different failure modes

The mobility differed significantly in five types of failure modes, which could be revealed by the sliding velocities. As mentioned above, the sliding distance obtained by the displacement transducer was sampled at a frequency of 20 Hz, while those obtained from the images dispatched from the video recordings for PIV analysis were obtained at a rate of 10 Hz, the time intervals for the calculation of velocities are 0.05 s and 0.1 s, for the data from displacement transducer and video recordings, respectively. In Fig. 9, we summarized the sliding velocities in the main failure events against sliding displacement for different types of failure modes. Fig. 9a presents the results of test T-4 (S7), in which the landsliding behavior is grouped into Type I. It is seen that the sliding mass accelerated to the first peak velocity, and then behaved with repeated accelerations and decelerations with a maximum velocity of about 8 mm/s. Fig. 9b shows the results for T-25 on M40 in Type I, the sliding mass accelerated until reached a certain velocity (260 mm/s), and then decreased within the capacity of the displacement measuring system by the linear displacement sensor. Fig. 9c shows the results for T-7 on M10 with the failure mode of Type III. In this test, the sliding mass accelerated to a certain velocity (260 mm/s), which was followed by deceleration and coming to a stop. However, the once-stopped soil mass slid again and finally moved slowly after experiencing accelerating and decelerating processes. The first acceleration and deceleration resulting from the sudden disintegration of the entire soil mass are the typical feature of Type III. Fig. 9d summarizes the results for

the tests showing other types of failure modes. In this figure, all the velocities are calculated by means of PIV for the available monitoring range of sliding distance by the video camera (about 1.0 m). Unlike the sliding behaviors as shown in Figs. 9a-c, within the available monitoring range of sliding distance, the sliding mass behaved in continuous accelerating movement, and the peak velocities were normally > 1.5 m/s, much greater than those in Type I and Type III.

4. Discussion

4.1 Tilting angle change and initiation of landsliding

For the slow landslide in Type I, as shown in Figs. 5b, 5e, and 7b, the soil mass at the bottom slightly rotated forward at first, which was followed by dramatic backward rotation accompanied by the occurrence of a major failure. This kind of variation in the tilting angles indicates that the soil mass suffered from local toppling deformation at first and then was followed by rotational sliding at the bottom. Further, it is noticed that the initiation of rotation was prior to the response in the macroscopic displacement (Fig. 7b), indicating that tilting monitoring provided better performance than the macroscopic displacement monitoring on the soil mass along the sliding surface (on the base). In this sense, our experimental results show good consistency with these phenomena occurring in natural landslide events (Doi et al. 2020). In addition, the retrogressive landslide phenomenon had been well illustrated by the sequences of soil rotation, indicating that the tilt sensor can also provide information on the next sliding area after one landslide event for those retrogressive landslides.

The event that tilting is prior to the sliding of mass not only occurred in slow-moving landslides but also was observed in the fluidized landslides in our tests. As exemplified by the results shown in Fig. 8b, for the fluidized landslide in Type IV, the tilt sensor (at location P6) recorded a sudden rotation within the soil layers at 1528.10 s, while the displacement measuring system started to respond about 0.2 s late. Although

the time difference is very small, the inclination change could be regarded as the precursor of the initiation of the fluidized landslide.

4.2 Effects of fines content and density index on the landslide failure mode

As shown in Fig. 4, our test results indicated that the landslide failure modes might be regulated by both the fines content (FC) and the density of the soil mass. To clarify the effects of FC and density on the landslide failure modes, we tabulate the observed failure modes with variations of FC and I_d in Fig. 10, from which it is clearly noticed that the failure modes were regulated by FC and I_d . When the I_d was approximately the same ($-0.54 \sim -0.52$), the landslide type transformed from sudden multiple sliding (Type III) to deep-seated fluidized sliding (Type IV) with the increase of FC. With a further increase of FC (30% and 40%), the landslide type transformed from Type IV to Type V of shallow fluidized sliding. However, this variation tendency of failure mode with the increase of FC changed with the increase of density. It is noticed that with the increase of density, the failure modes of Type I and Type II occurred at the I_d of $-0.19 \sim -0.21$, while Type V did not appear at tests on M30. In general, the failure mode of Type I appeared more frequently, but other types of failure modes decreased with the increase of I_d or decrease of FC.

Previous studies have revealed that the landslide failure modes are greatly dependent on the fine particles and initial density (Furuya et al., 1999; Wang and Sassa, 2001, 2003; Hu et al., 2017, 2018). However, these researchers discussed the effects of FC or initial density on the resultant failure modes individually. In this sense, our results suggest that to figure out the effect of FC on the initiation and movement of landslides, an integrated analysis of the observed landsliding phenomena with consideration of the joint effects of FC and sample density will be essential, because the observed landsliding phenomena are regulated by both the FC and initial density.

4.3 Effects of fines content and density index on the sliding velocities

To quantitatively evaluate the effects of FC and I_d on the sliding velocities, we examined the peak velocity (V_p) during the main failure as a function of FC or density index (I_d) in Fig. 11. Fig. 11a presents the V_p against FC for all the tests. For the tests with I_d being -0.54 ~ -0.52 and -0.21 ~ -0.19, V_p increased at first with the increase of FC when FC < 30%, and then decreased with a further increase in FC of 30~40%. However, for the tests with higher I_d of -0.06 ~ 0.25, the V_p increased continuously with increased FC (0~40%). Among them, it is noted that for the tests with the densest sample ($I_d = 0.24 \sim 0.25$) in the study, the velocity exhibits a nearly two-order magnitude change with increasing FC. This trend is also depicted in Figs. 9a and 9b for the T-4 on S7 and T-25 on M40 in Type I characterized by slow individual sliding. Despite this observed difference, the magnitude of velocity remains relatively small compared to other landslide events characterized by fast sliding. However, it is still evident that fines can have a positive influence on landslide motion, even in the densest samples examined in this study. These results suggested that landsliding velocity has an acute sensitivity to the initial fines content. There is an optimal FC at which V_p reached a maximum, and the value of the optimal FC increased from 30% to 40% with the increases in I_d . Fig. 11b shows the relation between V_p against I_d . When FC is low (at 0~10%), V_p decreased with the increase in I_d . However, with a further increase of FC in the sample, V_p increased at first with the increase of I_d and then decreased with a further increase in I_d . Namely, an optimal I_d existed in these tests on M20, M30, and M40. It is also noticed that this threshold value of I_d is -0.2 for the samples of M20 and M30, and -0.04 for the tests on M40, indicating the FC-dependent feature of the optimal I_d .

Previous studies have revealed that initial density and FC could play a significant role in the landsliding rates (e.g., Iverson et al., 2000; Wang and Sassa 1998, 2001, 2003). Some studies suggest that the high mobility of landslides is related to optimal relative densities of the material in flume tests (Wang and Sassa 2001; Zhang et al., 2020), and higher content of fines can promote landslide mobility (Wang and Sassa,

2003; Hu et al., 2017, 2018). These results are based on the individual effects of FC and density, lacking comprehensive research on these two effects. However, these two effects may influence each other. In this research, we found that an optimal FC of 30% could contribute to the maximum V_p of landslides in the relative looser sample. While the V_p of landslides increased monotonically with increased FC in the relatively denser sample, the V_p reached the maximum when FC is 40%. On the other hand, an optimal I_d is related to the maximum V_p of landslides in the sample with a relatively higher FC, and V_p decreased monotonically with increased I_d in the sample with a relatively lower FC. These results indicate that FC and I_d can jointly regulate the peak sliding velocity also.

4.4 Effects of fines content and density index on the pore water pressure

To examine the role of pore water pressure in the initiation and movement of landslides, we analyzed the pore-water pressure (PWP) in all the tests during pre- and post-main failure time periods. Fig. 12 schematically summarizes the variation of pore-water pressure before and after the onset of the tilting angle within the soil mass near the bottom (P6 and P7) of the flume and within the soil mass (P9) in all the tests. Some of the tests with failure modes of Type II~V showed that PWP increased and accelerated to the peak before the start of tilting, then it sharply dropped with the sudden rotation of the soil mass (Fig. 12a). Some tests with failure modes of Type I-II showed that the PWP gradually increased, kept relatively stable for a short period. Thereafter, the PWP showed a certain build-up after the occurrence of tilting, and finally decreased remarkably (Fig. 12b). Fig. 12c presents the variation of PWP for some tests showing failure modes of Type IV-V. The PWP continuously increased to a high value and then decreased slightly. It increased again and finally showed a sharp drop with the occurrence of tilting. Fig. 12d presents a relatively gradual trend of PWP response (observed in Type I), which increased continuously and then started decreasing before the tilting of the soil mass. Within the soil mass (P9), two patterns of PWP variations

were recorded. One is illustrated in Fig. 12e (observed in Type I), in which the PWP increased and then started decreasing when tilting started. The other one (Fig. 12f) was observed in the tests showing failure modes of Type II-V. In these tests, the buildup of PWP after the occurrence of tilting was observed.

Many studies examined the build-up of pore water pressure in the landsliding soil mass (Eckersley, 1990; Wang and Sassa, 2001, 2003; Olivares and Damiano, 2007; Hisada and Nakata, 2016; Hu et al., 2017, 2018; Zhang et al., 2020). For example, by using a flume, Wang and Sassa (2003) triggered flowslides by rainfall on the mixture of silica sand ($D_{50} = 0.050$ mm) with different contents of loess ($D_{50} = 0.0185$ mm) taken from China and found that there was no obvious difference in the pore water pressure built-up after the failure when the initial densities of the soil layers were approximately the same. Hu et al. (2017) carried out flume tests on granular mixtures of coarse grains ($D_{50} = 3$ mm) with small particles ($D < 0.5$ mm) by infiltrating water into the back of the slope towards the flume floor and pointed out that the peak value of PWP increases with the content of small particles in the tested range of 0% to 12%. Using a flume, Hisada and Nakata (2016) simulated rainfall-induced landslides on the decomposed granite soil with fines and found that the build-up of pore water pressure became larger when fines content increased from 5% to 25%. Although the grain sizes of the fine materials used in these two exemplified studies are different, it could be concluded that the peak value of PWP increases with the increase of fine materials in the studied sandy materials. However, as shown in Fig. 12, the peak value of PWP may include the PWP built up after the initiation of the failure in some tests, we examined both the peak magnitude of PWP (u_p) and the PWP built-up after the beginning of tilting (Δu) in all of our tests measured at the locations of P6, P7, P9, and P10. The results are presented against the fines content in Figs. 13a and b, and against the density index in Figs. 13c and d, respectively. From Fig. 13a, it is seen that although u_p presents a random tendency for the tests with I_d being around $-0.54 \sim -0.52$, it becomes greater in general with the increase of FC for the tests under other values of I_d , irrespective of some exceptions. In this sense, Fig. 13a shows consistency with the

findings reported by [Hu et al \(2017\)](#). These suggest that fines may facilitate the static liquefaction development of mixtures. Fine grains could suspend or float in the pore fluid and act as part of the fluid, the generated PWP may dissipate slowly due to the lower permeability of the mixture, maintaining high PWP during motion for the soils containing high amounts of fines ([Iverson, 1997; Wang and Sassa, 2003; Okada and Ochiai, 2008; Iverson et al., 2015; Nishiguchi and Uchida, 2002](#)). Nevertheless, no clear tendency could be identified between Δu and FC from [Fig. 13b](#). From [Fig. 13c](#), it is noticed that for tests on S7, u_p decreased with the increase of I_d , while tests on other samples showed the existence of an optimal I_d , at which u_p reached its maximum, although there was one exception in each of the tests on M20 and M30. The results shown in [Fig. 13d](#) show that although there is an exception for M30, there is an optimal I_d (-0.06 ~ -0.04) in general for the tests on each sample, below which Δu increased with the increase of I_d , then it decreased with a further increase in I_d . [Fig. 13d](#) shows good consistency with the results reported by [Wang and Sassa \(2001\)](#), in which the existence of the optimal density was reported and was inferred as the combined result of the pore-pressure generation rate and pore-pressure dissipation rate.

4.5 Role of fines

To clarify the role of fines, it is essential to compare the mechanical behaviors of these mixtures with different fines contents under the same initial condition index, such as initial density or void ratio. By now, the global void ratio (e) or void-ratio-based density index had been widely employed to evaluate material properties and related landsliding behavior (i.e., [Wang and Sassa, 2001; Zhang et al., 2020](#)). However, the void ratio may not be an appropriate variable for characterizing sand with fines under the critical state soil mechanics framework, because it could not reasonably assess the actual effects of fines content ([Thevanayagam et al., 2002; Rahman et al., 2008; Chen et al., 2020; Rahman and Sitharam, 2020; Rahmani and Naeini, 2020; Porcino et al., 2021](#)). Thus, the intergranular void ratio (e_s) and interfine void ratio (e_i)

were proposed as alternative density variables to interpret the behavior of silty sands (Thevanayagam, 1998; Thevanayagam and Mohan, 2000). e_s and 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 is the global void ratio, FC is the fines content. These variables are proposed based on the assumption that when the fines content in the mixture is low, all the fines can be approximated as void and their contribution to the force structure can be neglected, and then the usage of intergranular void ratio will be reasonable. However, when the fines content in the mixture increase to a certain value (termed threshold FC_{th}), the force structure of the mixture will be mainly controlled by the contacts between the fines and the coarser grains will float within the finer-grains matrix, and then the usage of the interfine void ratio is suggested.

Considering that in the silty sands, there are both inactive fine particles (stay in the voids created by coarse grains and do not contribute to material resistance) and active fine particles (touch and separate the coarse grains), Thevanayagam et al. (2000) further proposed two density variables, i.e., equivalent intergranular void ratio (e_s^*) and equivalent interfine void ratio (e_f^*), to include the effects of either active or inactive fines. e_s^* and e_f^* write 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 is the fraction of fines that actively contribute to the force structure; $R_d = D_{50}/d_{50}$ (D_{50} and d_{50} : the mean grain sizes of coarse and fine grains, respectively) and presents the size disparity ratio; m is the coefficient that depends on grain characteristics and fine grain packing ($0 < m < 1$) (Kanagalingam and Thevanayagam, 2005).

In the framework of these variables, the determination of FC_{th} for the use of different variables and b is essential (Ni et al., 2004; Mohammadi and Qadimi, 2015; Chang and Deng, 2019; Barnett et al., 2020). By now, two approaches had been proposed to determine the FC_{th} . One is based on the assumption that FC_{th} presents the FC percentage at which e_{max} and e_{min} of the mixture reach their minimums (Lade et al., 1998; Yang and Wei, 2012; Zuo and Baudet, 2015; Rahmani and Naeini, 2020). As another one is a semi-empirical approach introduced by Rahman et al. (2009), in which FC_{th} is calculated by:

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

where α ($= 0.50$) and β ($= 0.13$) are curve-fitting constants; and χ is the particle-size disparity ratio (D_{10}/d_{50}). The application of Eq. (5) for the tested samples gives a value of $FC_{th} = 32\%$ for the mixtures used in our study. To check the applicability of these approaches, we further conducted tests to measure e_{max} and e_{min} for the mixture with $FC = 32\%$. The results are included in Fig. 2b, from which it is noticed that the e_{max} is the smallest compared with other mixtures. Therefore, we use 32% as the value of FC_{th} in the following discussions, namely, we use e for all the tested samples, e_s and e_s^* for S7, M10, M20, and M30, and e_f and e_f^* for M40.

The parameter, b , represents the fraction of fines that are active in the force structure of the soil skeleton and can be determined by the semi-empirical equation (Rahman and Lo, 2008; Mohammadi and Qadimi, 2015), which writes as:

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

where $r = 1/\chi$ and $k = 1 - r^{0.25}$. Therefore, b is calculated as 0%, 34%, 42%, 48% and 53% for S7, M10, M20, M30, M40, respectively.

To examine the role of fines on landsliding behaviors, we first evaluate the mobility of displaced landslide materials in different types of failure modes by adopting different void ratio variables of e , e_s/e_f , and e_s^*/e_f^* .

Fig. 14 presents the results in the forms of V_p against e (Fig. 14a), e_s/e_f (Fig. 14b), and e_s^*/e_f^* (Fig. 14c),

respectively. It could be noticed from Figs. 14a-14c, irrespective of the types of void ratio, V_p increased with the increase of void ratio for S7 and M10. However, for other mixtures, V_p increased with the increase of void ratio but decreased with further increase of void ratio after the void ratio is greater than a certain value, i.e., for each mixture, there is an optimal void ratio at which the V_p reaches its maximum. Compared to the plots of V_p against e (shown in Fig. 14a), the plots of V_p against e_s/e_f for S7, M10, M20, and M30 shift to a narrow domain, while those for M40 shift away from other mixtures. Nevertheless, these plots of V_p against e_s^*/e_f^* shrink to a relatively narrow range, indicating that e_s^* and e_f^* are more appropriate as the indexes to provide a unified interpretation for the variation of V_p with void ratio. In Fig. 14c, an overall tendency was observed, where V_p first generally increased with increased e_s^*/e_f^* . However, V_p started to fall overall when e_s^*/e_f^* exceeded 1.8. This may result from the evolution of the mixture's structure. Based on the framework of the equivalent void ratio by Thevanayagam et al. (2002), initially, a small number of fine particles act as fillers. With the increase of FC when FC is less than the threshold, fine grains partially come in between the strong contacts of coarse particles and participate in the mechanical transferring, playing a secondary role in the structure (the behavior of the mixture is primarily controlled by the contacts of coarse grains, namely, coarse-material-dominated state). However, when FC is high enough to exceed the threshold, fines can separate and disperse the contacts of coarse grains, leading to the dominance of fine particle contacts (fine-material-dominated state). From Figs. 4 and 10, it is observed that V_p generally became greater with the increase of e_s^* and e_f^* , and the landsliding phenomena also transformed from slow slides to deep-seated fluidized slides. These suggest that the role of fine particles in affecting the mechanical behavior of the mixture becomes stronger in the coarse-material-dominated mixture with the increase of e_s^* and e_f^* . However, From Figs. 4 and 10, it is also observed that V_p started to decrease when e_s^*/e_f^* exceeded 1.8, and the displaced materials were characterized by fluidized sliding with shallow sliding surfaces. This may result from the fact that the mixtures become a fine-material-dominated state, in which the fine

particles started to play a primary role in mechanical behavior, while the permeability of the mixture becomes smaller, favoring the initiation of shallow sliding.

On the other hand, it is also noted that in this study, although we respectively employed two indexes, e_s^* and e_f^* , to mathematically represent the coarse-material-dominated state and fine-material-dominated state, these indexes may be not fully suitable for explaining our experimental findings. This discrepancy arises due to the intricate nature of granular systems and the wide range of densities involved, making it challenging to precisely determine the dominant grain type when the fines content (FC) is high around the fines content threshold (FC_{th}), which is sensitive to the varied density of granular materials. Besides, an estimated FC_{th} of around 30~40% was observed based on void ratio indices, and the FC_{th} of 32% was calculated and used based on the semi-empirical formula. Consequently, an equivalent interfine void ratio (e_f^*) was employed for the mixtures containing 40% of FC. These factors introduce some uncertainty regarding the dominance of coarse or fine grains in the mixtures with high FC near the FC_{th} . Notably, the dominant particles may undergo a transition in a mixture with the same high FC around FC_{th} as the packing density changes from very loose to very dense, leading to a shift in dominance from fines to coarse particles. For instance, in this study, an FC of 40% is considered relatively high and is situated close to the FC_{th} , indicating proximity to the transitional state of the mixture structure. As the density increases, the structure of the mixture may shift from fines-dominant to coarse-grain-dominant, and vice versa. Consequently, V_p begins to decrease with a further increase in $e_s^*/e_f^* > 1.8$. This transition is accompanied by changes in observed landslide behavior, transitioning from deep-seated fluidized slides to shallow fluidized slides, as the mixture structure transforms from coarse grain-dominant to fines-dominant due to decreasing density. However, in cases where FC is very low (much lower than FC_{th}), coarse particles exert a key influence in both dense and loose mixtures. Conversely, when FC is very high (much higher than FC_{th}), fine particles could play a dominant role in both dense and loose mixtures.

Fig. 15 presents the plots of u_p and Δu against e (Figs. 15a and 15b), e_s/e_f (Figs. 15c and 15d), and e_s^*/e_f^* (Figs. 15e and 15f). Basically, for each sample, the variation trend of u_p and Δu against these void ratios shown in Fig. 15 are similar to those shown in Figs. 13c and 13d. However, it is noticed that through using e_s/e_f and e_s^*/e_f^* , u_p can be integrated into a relatively smaller domain, but did not integrate all samples for Δu , basically indicating the appropriateness of using these void ratios in the examination of roles of fines in the mixture. It is further seen that e_s^*/e_f^* provides better performance (Figs. 15e) than e_s/e_f (Figs. 15c) in evaluating the values of u_p under the same range of void ratios. Therefore, it is reasonable to infer that the roles of fines on the landsliding behavior of sandy materials with fines are properly interpreted by the assumptions for the concept of e_s^* and e_f^* . In this sense, the fines may affect the landsliding behavior by affecting the microstructure of the mixtures and also the mechanical behavior of fines working as contacts. It is noted that to unravel the possible effect of fines on the contacting points, examining the shear behavior of the mixtures under different normal stresses will be needed and are under plan.

From Fig. 14c and Fig. 15e, it is noticed that both V_p and u_p showed the same general variation trend with e_s^*/e_f^* . Therefore, we infer that the landsliding behavior is positively regulated by the maximum pore-water pressure that has been built up within the landsliding mass. As reported by previous studies (Iverson, 1997; Wang and Sassa, 2003; Okada and Ochiai, 2008; Iverson et al., 2015; Nishiguchi and Uchida, 2022), in those fluidized landslide materials with fines, the fine materials could remain suspension/rotation and act as part of the interstitial fluid, and then retard the dissipation of PWP and elevate the mobility of the sliding materials. Although the trend of peak velocity against e_s^*/e_f^* is generally consistent with the observed PWP (u_p), we noticed that in some cases u_p and Δu did not present good consistency with the variation of the landslide mobility. This may result from the facts that: (1) the measured PWP in our study was the PWP in the initial stage of landsliding failure progress, but the V_p was obtained during the long-distance travel progress in the landsliding path after failure; (2) the fine grains can facilitate the landsliding

by reducing the roughness of assemblies of granular particles, as reported by some studies (Roche et al., 2005; Perinotto et al. 2015; Bernard and Le Pennec, 2016; Hyodo et al., 2017; Rahmani and Naeini, 2020).

5. Conclusions

The goal of this study was to investigate the impact of non-plastic fines on rainfall-induced landslides. To achieve this, we conducted 25 laboratory flume tests using mixtures of silica sand with varying amounts of non-plastic fines and initial densities. The results led to the following conclusions:

(1) The occurrence and characteristics of landslides were highly dependent on the amounts of fines in the mixture and the initial soil density. We identified five failure modes: slow and fast retrogressive individual sliding events characterized by inconsistent retrogressive behavior (Types I and II), sudden multiple sliding characterized by the disintegration of the soil mass into several blocks (Type III), and deep-seated or shallow fluidized sliding characterized by rapid flowslides with the deep or shallow sliding surface (Types IV and V). The failure mode shifted from Type I to Type V as the fines content increased or the density index decreased.

(2) Based on the soil rotation within the soil mass, five types (Type I ~ Type V) of landslides were identified. In Type I, the tilting angle gradually changed as the landslide progressed, while in other types, the tilting angle changed sharply. We observed that tilting before the mass sliding occurred not only in slow-moving landslides but also in fluidized landslides. This suggests that monitoring the change in tilting angle within the soil layer can help in detecting the precursors of various types of landslides, including fluidized ones.

(3) The presence of non-plastic fines has a significant impact on the mobility of landslide and pore-water pressure (PWP) responses. When the initial density index (I_d) is greater than or equal to -0.06, the peak velocity (V_p) of landslides increases with the fines content. However, when I_d is less than or equal to -0.19,

an optimal fines content of 30% results in maximum V_p . Additionally, the maximum value of pore-water pressure (u_p) generally increases with the increase of fines content (FC).

(4) Density of the mixture also plays a critical role in the movement of landslides and pore-water pressure (PWP) responses. For samples with low fines content ($\leq 10\%$), V_p decreased with increasing density index of I_d . Conversely, for samples with high fines content ($\geq 20\%$), an optimal I_d exists at which the V_p reached its maximum, and this optimal value of I_d varies from -0.2 to -0.04 with increasing FC. Additionally, for sample S7, u_p decreases with the increasing I_d , whereas for other samples, an optimal I_d exists at which u_p reaches its maximum, irrespective of some exceptions. For all samples, an optimal I_d around -0.04 exists at which the build-up of PWP after the initiation of failure (Δu) reached its maximum as the comprehensive results of generation and dissipation of the PWP.

(5) The study employed different concepts of void ratios and found that the equivalent intergranular/interfine void ratio (e_s^*/e_f^*) offers better insight into unraveling the role of fines in the landslide behavior of sandy materials with fines. In the mixtures with FC less than 30~40% (a threshold value), although the mechanical behavior is controlled by the coarse grains contacts in general, with further increase of FC, more fines will actively participate in the inter-particle force transfer and their effect on the mechanical behavior will become important gradually. However, as FC increases further beyond the threshold, the fines begin to separate and disperse the contacts of coarse grains, and the role of coarse grains becomes less important, and then the mechanical behavior of mixtures will be controlled by fine-grains contacts. The variation trend of the maximum sliding velocity (V_p) and maximum pore-water pressure (u_p) in landslide events under the same reference indicator of e_s^*/e_f^* indicates that V_p is generally positively regulated by u_p .

(6) The appropriateness of e_s^* and e_f^* , as the reference indicators in identifying the behavior of sand-fine mixtures, highlights the need to understand the possible effect of fines on the formation of void space and the contacting points of the assemblies to gain a better understanding of the roles of fines in the mixture.

It is noted that further research will be needed to explore the effects of different fines contents and density indices on the landslide mobility and pore-water pressure responses of sandy materials with fines, and then provide a more comprehensive understanding of the behavior of these materials. The findings of this study suggest that the equivalent intergranular/interfine void ratio (e_s^*/e_f^*) could be a useful reference indicator for studying the role of fines in the landsliding behavior of sandy materials with fines. Future studies could investigate this concept to determine its applicability to other types of soils and geologic settings. It is also noted that the optimal fines content and density index values identified in this study could be used as guidelines for designing and constructing structures in areas prone to landslides. By taking into account the role of fines content and density index on landslide mobility and pore-water pressure responses, engineers and geologists could potentially mitigate the risks associated with landslides. The study highlights the importance of considering both the formation of void space and the contacting points of the assemblies when examining the role of fines in sandy materials. This could inform future research on the mechanisms that govern the behavior of these materials and could help to improve the accuracy of models used to predict landslides and other geohazards.

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Fig. 1. (a) Grain size distributions of silica sand no. 7 (S7), silica powder (SP), and mixtures of M10, M20, M30, and M40. S7 and SP were chosen as the coarse and fine materials, the mean grain sizes (D_{50}) are 0.16 mm and 0.035 mm for S7 and SP, respectively. (b), (c) Scanning electron microscope (SEM) images for S7 and SP. SP originates from silica sand by mechanical grinding.

Fig. 2. (a) Permeability of different mixture samples of S7 with 0%, 10%, 20%, 30%, and 40% of SP by weight. The permeability of mixtures decreases with increasing SP. (b) Maximum and minimum void ratios ($e_{\max}$ and $e_{\min}$) of mixtures of S7 with different contents of SP. Based on void ratio indices, a transitional behavior, with estimated fines content threshold (FC_{th}) of around 30-40% is observed. The value of $e_{\max}$ reaches the lowest point at 30% earlier than that of $e_{\min}$ at 40%. Fines content of 32% is the calculated fines content threshold (FC_{th}).

Fig. 3. (a), (b) Side view and front view of the employed experimental flume. The flume is 200 cm in length, 25 cm in width, and 40 cm in height. (c) The schematic of the employed flume. The sample in the flume is designed as a right trapezoid, the rainfall intensity is kept at 100 mm/h, and two video cameras are set at the side and front of the flume. (d) Arrangement of samples and monitoring system, which is the zoom from Fig. 3c. The pore water pressure (PWP) sensors and tilt sensors are set. The tilting angle is the angle measured from the experimental setup. At location P6, a foam ball is used to monitor the displacement of the soil mass by the linear displacement transducer and laser displacement sensor. A 150 g counterweight is utilized to balance the pulling resistance of the linear displacement sensor.

Fig. 4. Summarized failure modes in the main failure process of landslides (the left side is the schematic

diagram, and the right side is the corresponding picture of failure processes). Type I, slow retrogressive individual sliding. Type II, fast retrogressive individual sliding. Type III, sudden multiple sliding. Type IV, deep-seated fluidized sliding. Type V, shallow fluidized sliding. The time began with the start of rainfall. 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.

Fig. 5. (a) Tilting variation in the landslide Type I showing slow retrogressive individual sliding (T-4). (b) ~ (e) Enlarged views of Fig. 5a at locations P6, P3, P2, and P5, respectively. The arrow shows the start of the variation in tilting.

Fig. 6. (a) Variation of tilting angle in the test showing deep-seated fluidized sliding (Type IV) (T-23). (b) ~ (h) Enlarged views of Fig. 6a at locations P7, P3, P6, P2, P5, P1, and P4, respectively. The arrow shows the start of the variation in tilting.

Fig. 7. (a) Results of pore-water pressure, tilting angle, and sliding distance at the locations P6 and P9 during the slow retrogressive individual sliding in Type I (T-25). (b) Enlarged part of Fig. 7a.

Fig. 8. (a) Time series of pore-water pressure, tilting angle, and sliding distance at the locations P6 and P9 for the test with deep-seated fluidized sliding in Type IV (T-23). (b) Enlarged part of Fig. 8a.

Fig. 9. Sliding velocity against sliding distance during the main failure. (a) For T-4 on S7 with Type I. (b) for T-25 on M40 with Type I. (c) for T-7 on M10 with Type III. In the Type I, III, the sliding mass behaved with repeated accelerations and decelerations, showing fluctuation features. (d) For T-12 on M20 with Type

II, T-17 on M30 with Type IV, T-23 on M40 with Type IV, and T-16 on M30 with Type V. In Type II, IV, and V, the sliding mass behaved in continuous accelerating movement.

Fig. 10. Fines content (FC) and density index (I_d) and landslide types. The failure modes were regulated by FC and I_d , with the FC increasing and density decreasing, the landslide modes gradually transfer from Type I to Type V.

Fig. 11. (a) Peak velocity (V_p) of landslides against fines content (FC) with different density indexes (I_d). For the tests with lower I_d , there is an optimal FC at which V_p reached a maximum. For the tests with higher I_d , the V_p increased continuously with increased FC. (b) The V_p of landslides against density index (I_d) with different FC. For the tests with lower FC, the V_p decreased with the increase in I_d . For the tests with higher FC, an optimal I_d existed at which V_p reached a maximum.

Fig. 12. Summarized pore water pressure (PWP) variation pattern corresponding to soil mass tilting in each sliding mode. u_p : peak PWP; Δu : build-up of PWP. The vertical line shows the initiation of tilting within the soil mass.

Fig. 13. (a), (b) Peak value of PWP (u_p) and the build-up of PWP (Δu) against fines content (FC), respectively, on samples with different densities. (c), (d) u_p and Δu against density index (I_d), respectively, on samples with different FCs.

Fig. 14. (a), (b), (c): Peak velocity (V_p) against global void ratio (e), intergranular/interfine void ratio (e_s/e_f), and equivalent intergranular/interfine void ratio (e_s^*/e_f^*) for different samples, respectively. Compared to

997 the plots of V_p against e and e_s/e_f , the plots of V_p against e_s^*/e_f^* shrink to a relatively narrow range.

998

999 **Fig. 15.** Peak PWP (u_p) (a, c, e) and build-up of PWP (Δu) (b, d, f) against global void ratio (e),

1000 intergranular/interfine void ratio (e_s/e_f) and equivalent intergranular/interfine void ratio (e_s^*/e_f^*) for different

1001 samples, respectively.

1002 **Table 1.** Properties of employed samples.

Sample	Particle density	Minimum dry density	Maximum dry density
	$\rho_s \text{ (g/cm}^3\text{)}$	$\rho_{dmin} \text{ (g/cm}^3\text{)}$	$\rho_{dmax} \text{ (g/cm}^3\text{)}$
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

1003 **Table 2.** Test conditions and summarized data in all flume tests

Series no.	Test. no.	Sample	Density index	V_p	u_p	Δu	Note
			I_d	mm/s	kPa	kPa	
1	T-1	S7	-0.54	59.4	4.4	0.6	Show in Figs. 5 and 9
	T-2	S7	-0.20	19.3	1.3	0.6	
	T-3	S7	-0.04	10.0	1.7	1.1	
	T-4	S7	0.07	7.6	1.6	0.7	
	T-5	S7	0.25	6.7	1.3	0.3	
2	T-6	M10	-0.54	1101.5	2.0	0.7	Show in Fig. 9
	T-7	M10	-0.20	260.3	4.1	1.2	
	T-8	M10	-0.06	26.1	2.5	1.7	
	T-9	M10	0.09	19.5	2.6	1.4	
	T-10	M10	0.24	13.5	1.4	0.6	
3	T-11	M20	-0.52	1074.5	6.8	0.9	Show in Fig. 9
	T-12	M20	-0.21	1582.0	2.0	0.7	
	T-13	M20	-0.06	340.3	2.4	1.4	
	T-14	M20	0.08	51.9	1.1	0.5	
	T-15	M20	0.24	28.2	1.3	0.2	
4	T-16	M30	-0.53	1868.0	2.8	1.3	Show in Fig. 9
	T-17	M30	-0.2	2088.0	3.0	1.0	Show in Fig. 9
	T-18	M30	-0.05	1688.0	1.5	1.1	
	T-19	M30	0.08	85.1	1.7	0.8	
	T-20	M30	0.25	60.1	4.3	3.3	
5	T-21	M40	-0.52	1229.5	1.6	0.8	Show in Figs. 6, 8 and 9
	T-22	M40	-0.19	1980.0	6.4	1.5	
	T-23	M40	-0.05	2011.0	7.7	0.8	
	T-24	M40	0.08	1210.0	2.1	0.7	
	T-25	M40	0.24	260.3	2.0	0.9	Show in Figs. 7 and 9

1004 I_d : Density index; V_p : Peak velocity; u_p : Peak PWP; Δu : Build-up of PWP.

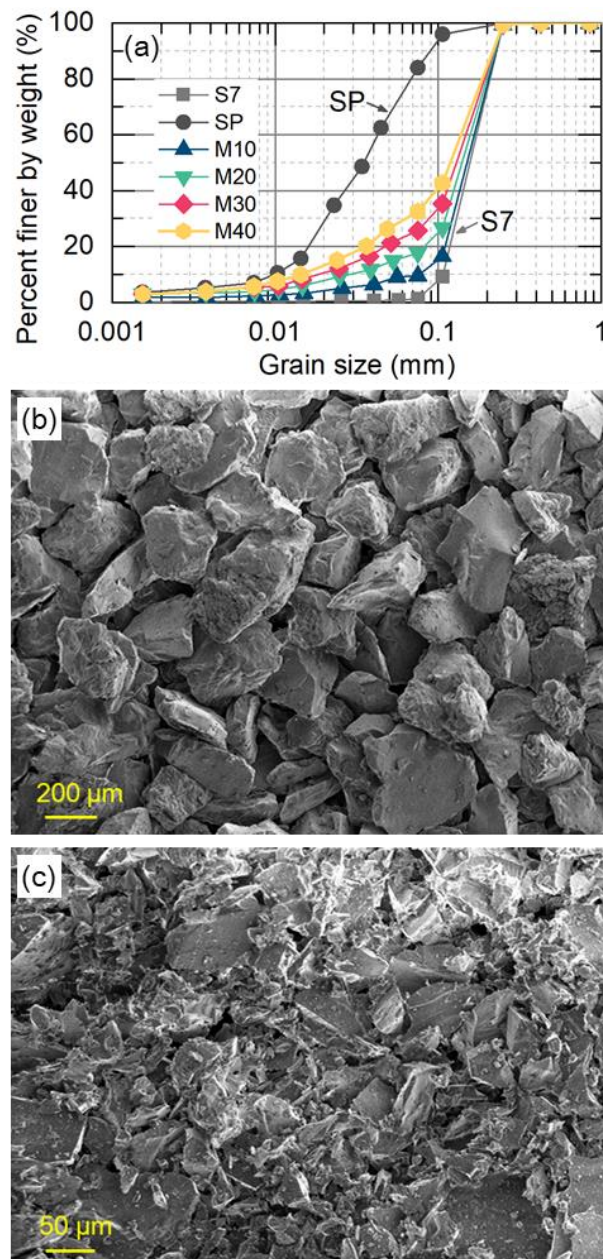


Fig. 1. (a) Grain size distributions of silica sand no. 7 (S7), silica powder (SP), and mixtures of M10, M20, M30, and M40. S7 and SP were chosen as the coarse and fine materials, the mean grain sizes (D_{50}) are 0.16 mm and 0.035 mm for S7 and SP, respectively. (b), (c) Scanning electron microscope (SEM) images for S7 and SP. SP originates from silica sand by mechanical grinding.

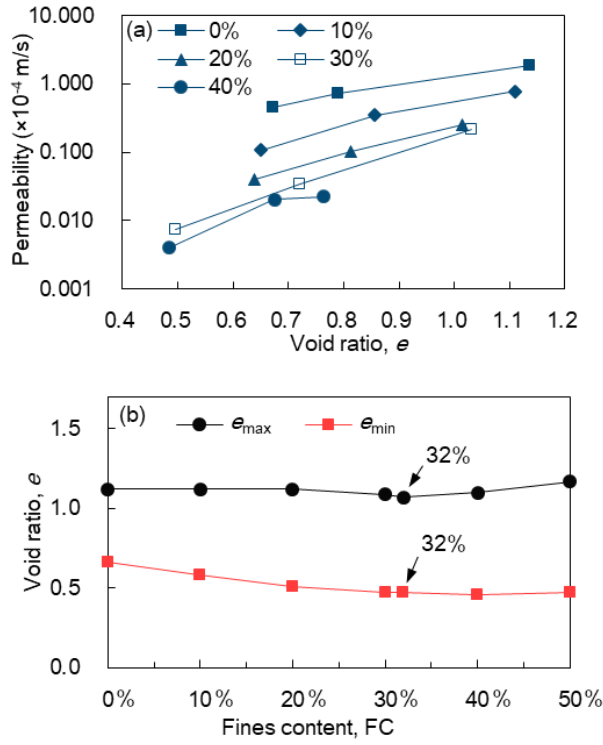


Fig. 2. (a) Permeability of different mixture samples of S7 with 0%, 10%, 20%, 30%, and 40% of SP by weight. The permeability of mixtures decreases with increasing SP. (b) Maximum and minimum void ratios ($e_{\max}$ and $e_{\min}$) of mixtures of S7 with different contents of SP. Based on void ratio indices, a transitional behavior, with estimated fines content threshold (FC_{th}) of around 30-40% is observed. The value of $e_{\max}$ reaches the lowest point at 30% earlier than that of $e_{\min}$ at 40%. Fines content of 32% is the calculated fines content threshold (FC_{th}).

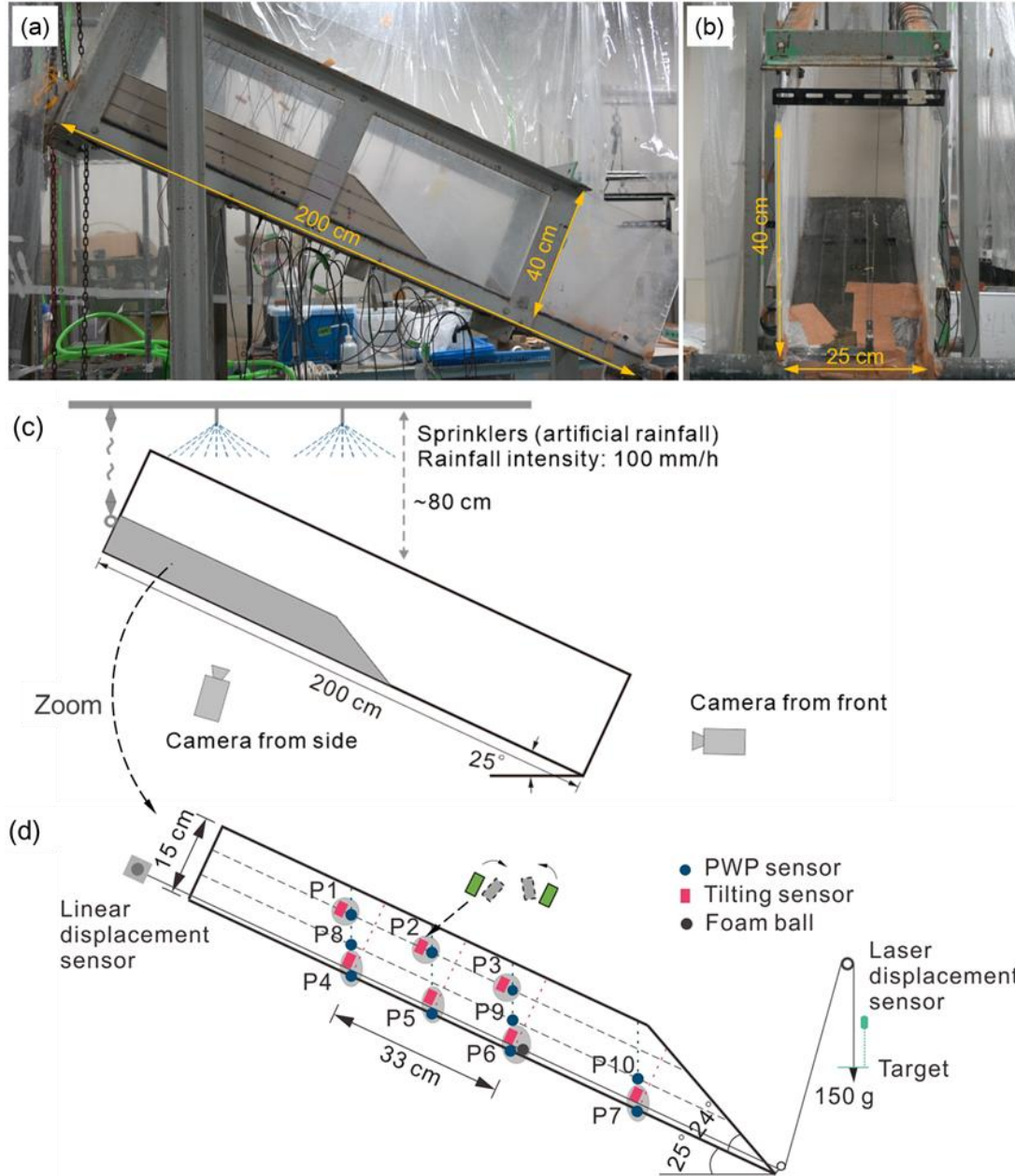


Fig. 3. (a), (b) Side view and front view of the employed experimental flume. The flume is 200 cm in length, 25 cm in width, and 40 cm in height. (c) The schematic of the employed flume. The sample in the flume is designed as a right trapezoid, the rainfall intensity is kept at 100 mm/h, and two video cameras are set at the side and front of the flume. (d) Arrangement of samples and monitoring system, which is the zoom from Fig. 3c. The pore water pressure (PWP) sensors and tilt sensors are set. The tilting angle is the angle measured from the experimental setup. At location P6, a foam ball is used to monitor the displacement of soil mass by the linear displacement transducer and laser displacement sensor. A 150 g counterweight is utilized to balance the pulling resistance of the linear displacement sensor.

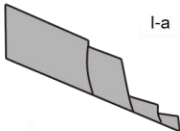
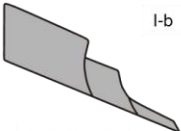
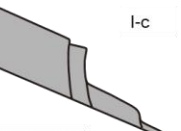
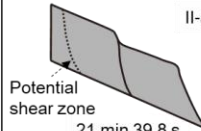
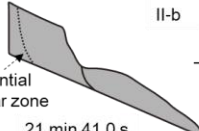
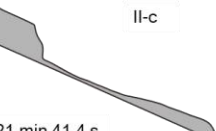
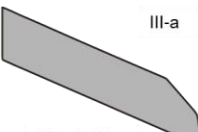
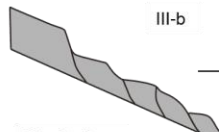
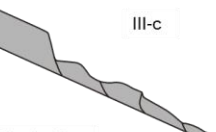
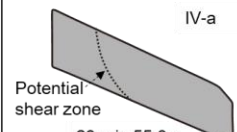
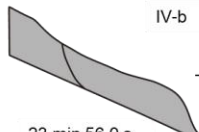
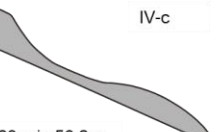
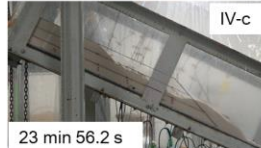
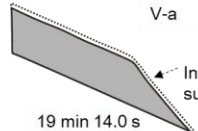
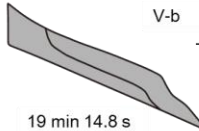
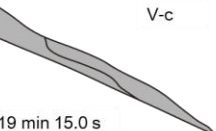
Sample and density	Failure mode	Failure processes in the main failure					
S7, $-0.20 \leq I_d \leq 0.25$ M10, $-0.06 \leq I_d \leq 0.24$ M20, $0.08 \leq I_d \leq 0.24$ M30, $0.08 \leq I_d \leq 0.25$ M40, $I_d = 0.24$	Type I: Slow retrogressive individual sliding Example: T-4	 25 min 8 s	 25 min 23 s	 26 min 5 s	 25 min 23 s	 26 min 5 s	
M20, $-0.21 \leq I_d \leq -0.06$ M30, $I_d = -0.05$ M40, $I_d = 0.08$	Type II: Fast retrogressive individual sliding Example: T-12	 Potential shear zone 21 min 39.8 s	 Potential shear zone 21 min 41.0 s	 21 min 41.4 s	 21 min 41.0 s	 21 min 41.4 s	
S7, $I_d = -0.54$ M10, $I_d = -0.20$	Type III: Sudden multiple sliding Example: T-7	 20 min 59 s	 21 min 0 s	 21 min 40 s	 21 min 0 s	 21 min 40 s	
M10, $I_d = -0.54$ M20, $I_d = -0.52$ M30, $I_d = -0.20$ M40, $I_d = -0.05$	Type IV: Deep-seated fluidized sliding Example: T-23	 Potential shear zone 23 min 55.0 s	 23 min 56.0 s	 23 min 56.2 s	 23 min 56.0 s	 23 min 56.2 s	
M30, $I_d = -0.53$ M40, $-0.52 \leq I_d \leq -0.19$	Type V: Shallow fluidized sliding Example: T-16	 Initial surface 19 min 14.0 s	 19 min 14.8 s	 19 min 15.0 s	 19 min 14.8 s	 19 min 15.0 s	

Fig. 4. Summarized failure modes in the main failure process of landslides (the left side is the schematic diagram, and the right side is the corresponding picture of failure processes). Type I, slow retrogressive individual sliding. Type II, fast retrogressive individual sliding. Type III, sudden multiple sliding. Type IV, deep-seated fluidized sliding. Type V, shallow fluidized sliding. The time began with the start of rainfall. 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.

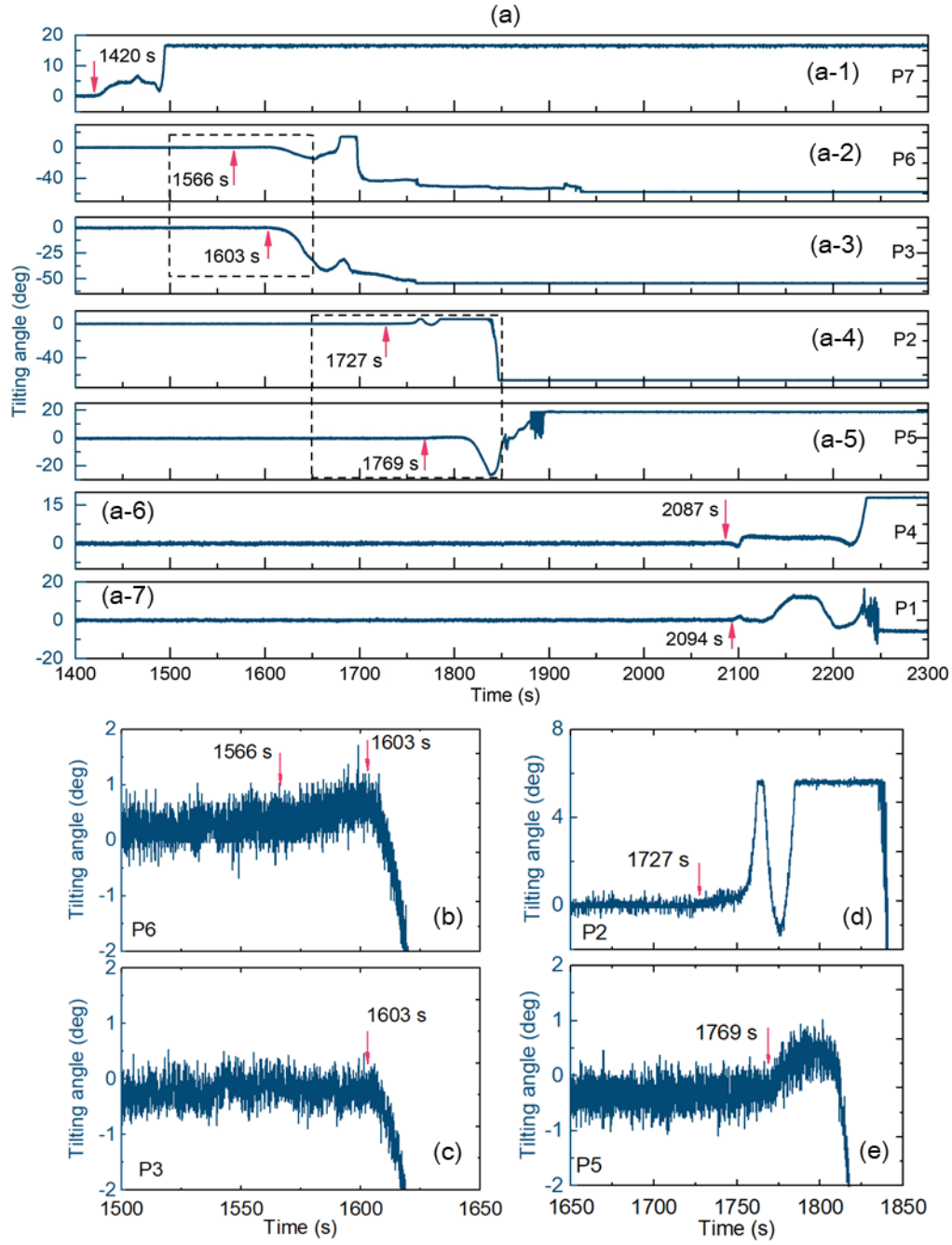


Fig. 5. (a) Tilting variation in the landslide Type I showing slow retrogressive individual sliding (T-4).

(b) ~ (e) Enlarged views of Fig. 5a at locations P6, P3, P2, and P5, respectively. The arrow shows the start of the variation in tilting.

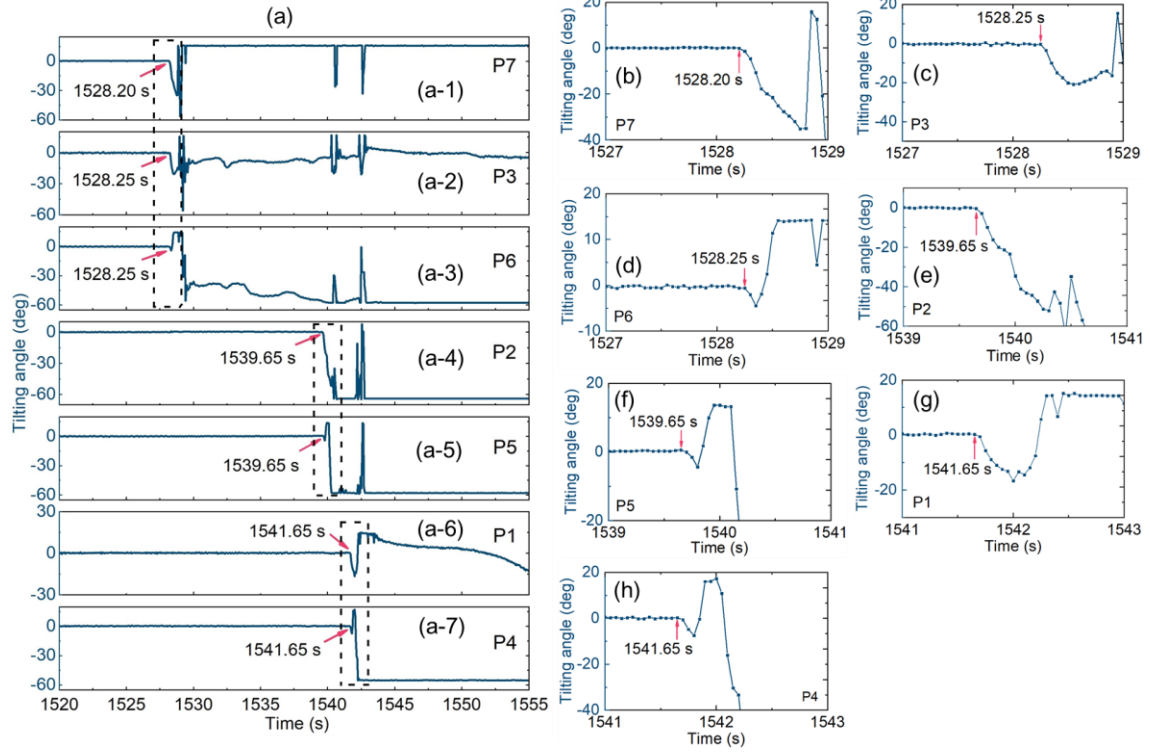


Fig. 6. (a) Variation of tilting angle in the test showing deep-seated fluidized sliding (Type IV) (T-23). (b)

~ (h) Enlarged views of Fig. 6a at locations P7, P3, P6, P2, P5, P1, and P4, respectively. The arrow shows

the start of the variation in tilting.

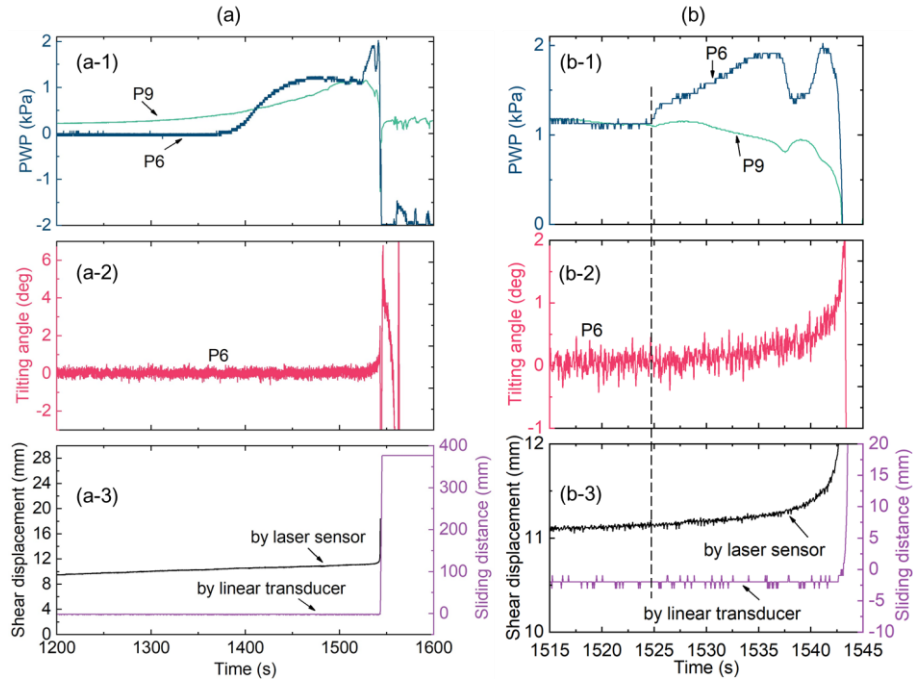


Fig. 7. (a) Results of pore-water pressure, tilting angle, and sliding distance at the locations P6 and P9

during the slow retrogressive individual sliding in Type I (T-25). (b) Enlarged part of Fig. 7a.

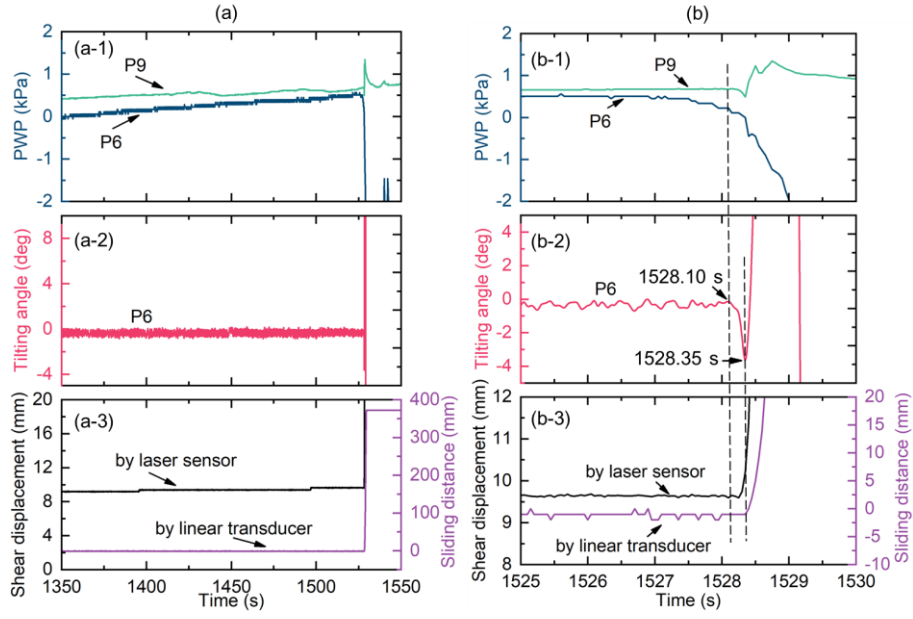


Fig. 8. (a) Time series of pore-water pressure, tilting angle, and sliding distance at the locations P6 and P9 for the test with deep-seated fluidized sliding in Type IV(T-23). (b) Enlarged part of Fig. 8a.

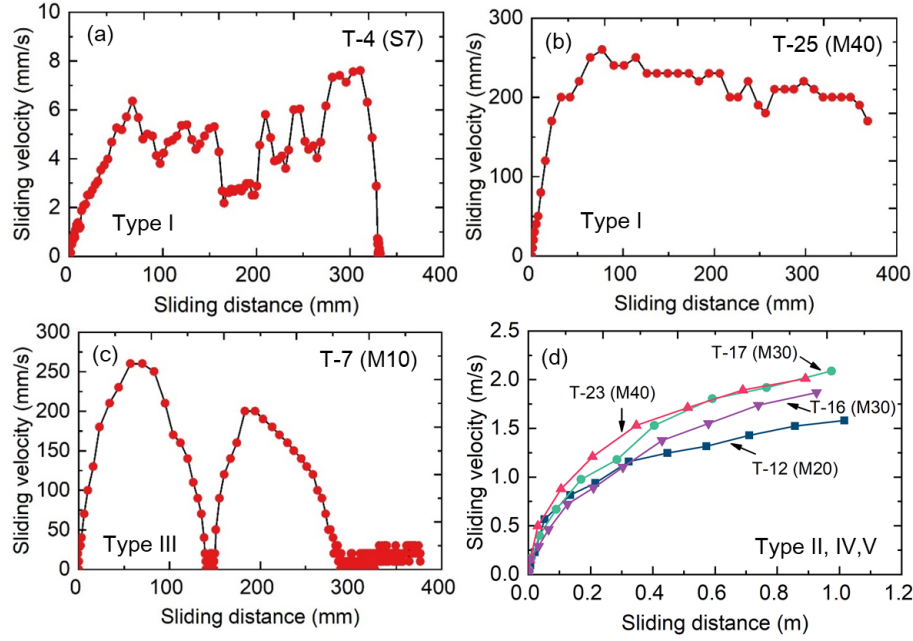


Fig. 9. Sliding velocity against sliding distance during the main failure. (a) For T-4 on S7 with Type I. (b) for T-25 on M40 with Type I. (c) for T-7 on M10 with Type III. In the Type I, III, the sliding mass behaved with repeated accelerations and decelerations, showing fluctuation features. (d) For T-12 on M20 with Type II, T-17 on M30 with Type IV, T-23 on M40 with Type IV, and T-16 on M30 with Type V. In Type II, IV, and V, the sliding mass behaved in continuous accelerating movement.

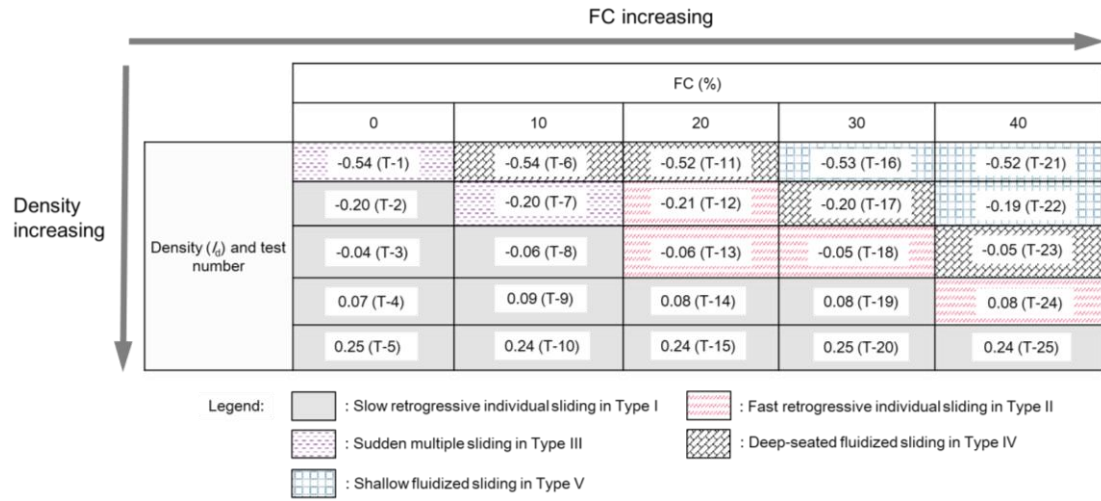


Fig. 10. Fines content (FC) and density index (I_d) and landslide types. The failure modes were regulated by FC and I_d , with the FC increasing and density decreasing, the landslide modes gradually transfer from Type I to Type V.

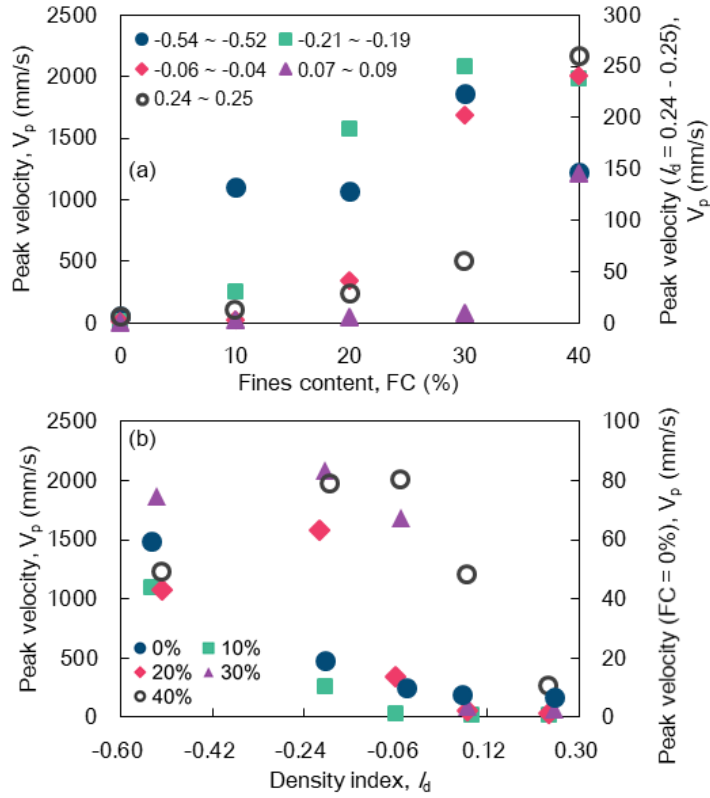


Fig. 11. (a) Peak velocity (V_p) of landslides against fines content (FC) with different density indexes (I_d).

For the tests with lower I_d , there is an optimal FC at which V_p reached a maximum. For the tests with higher I_d , the V_p increased continuously with increased FC. (b) The V_p of landslides against density index (I_d) with different FC. For the tests with lower FC, the V_p decreased with the increase in I_d . For the tests with higher FC, an optimal I_d existed at which V_p reached a maximum.

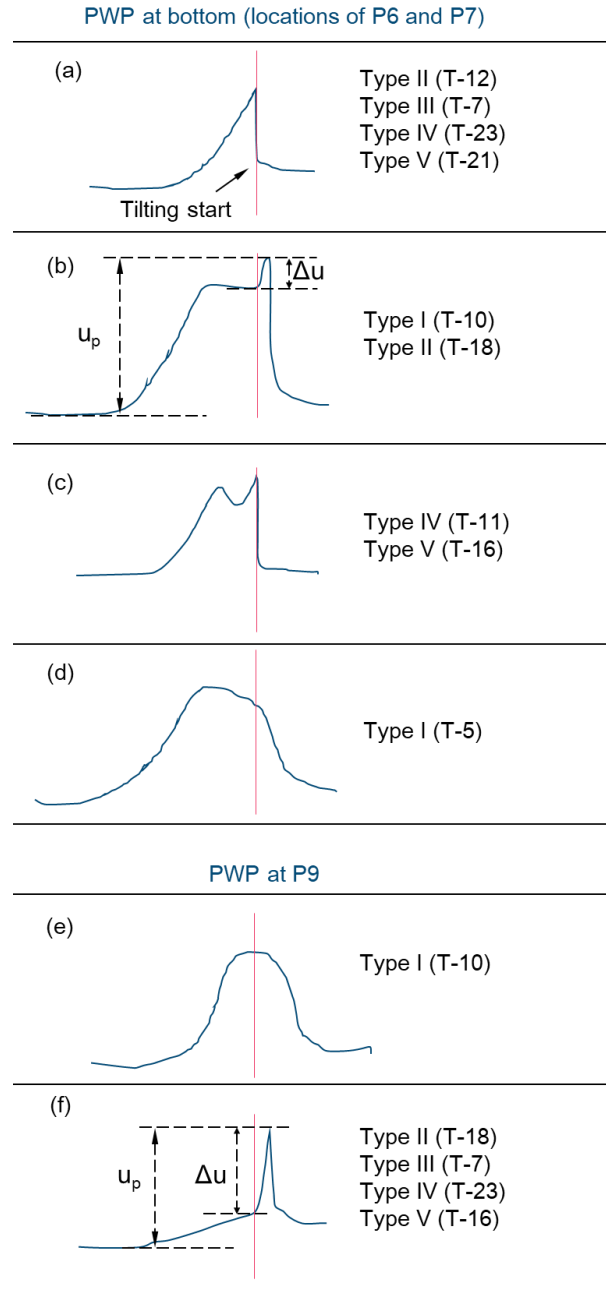


Fig. 12. Summarized pore water pressure (PWP) variation pattern corresponding to soil mass tilting in each sliding mode. u_p : peak PWP; Δu : build-up of PWP. The vertical line shows the initiation of tilting within the soil mass.

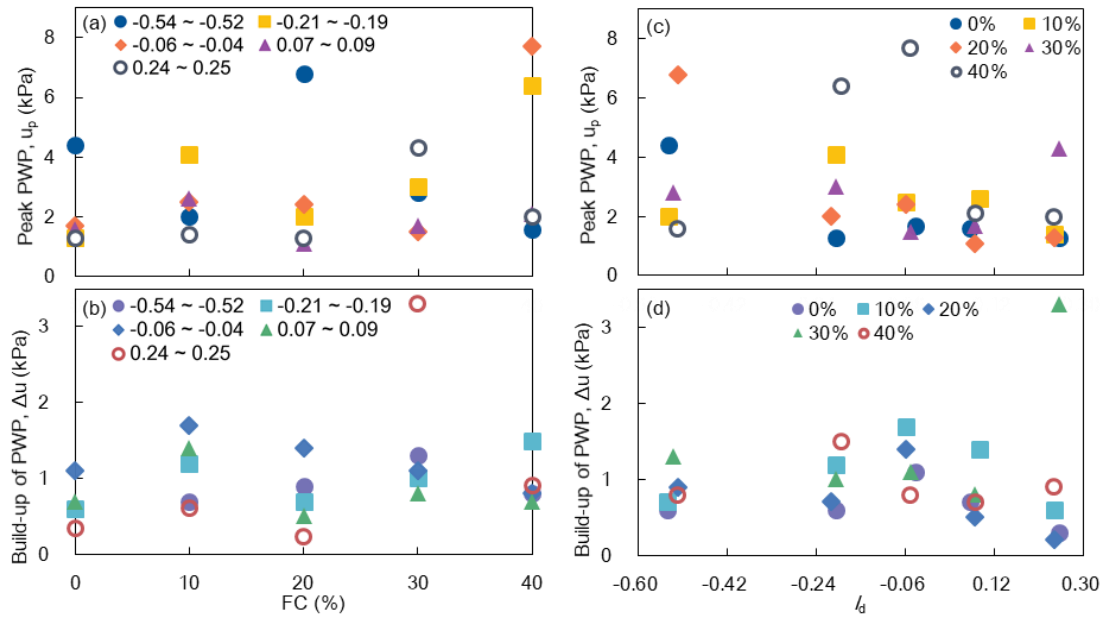


Fig. 13. (a), (b) Peak value of PWP (u_p) and the build-up of PWP (Δu) against fines content (FC), respectively, on samples with different densities. (c), (d) u_p and Δu against density index (I_d), respectively, on samples with different FCs.

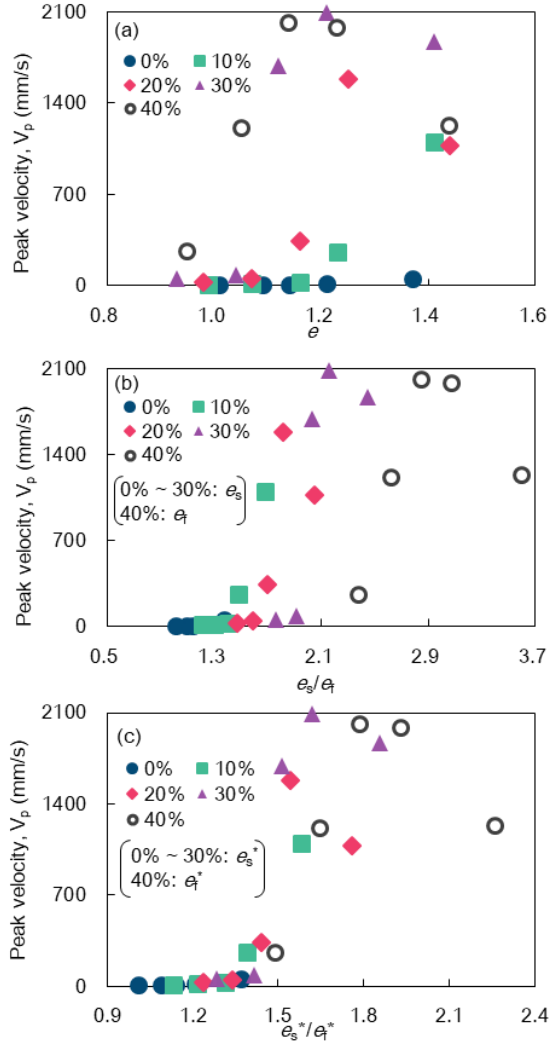


Fig. 14. (a), (b), (c): Peak velocity (V_p) against global void ratio (e), intergranular/interfine void ratio (e_s/e_f), and equivalent intergranular/interfine void ratio (e_s^*/e_f^*) for different samples, respectively.

Compared to the plots of V_p against e and e_s/e_f , the plots of V_p against e_s^*/e_f^* shrink to a relatively narrow range.

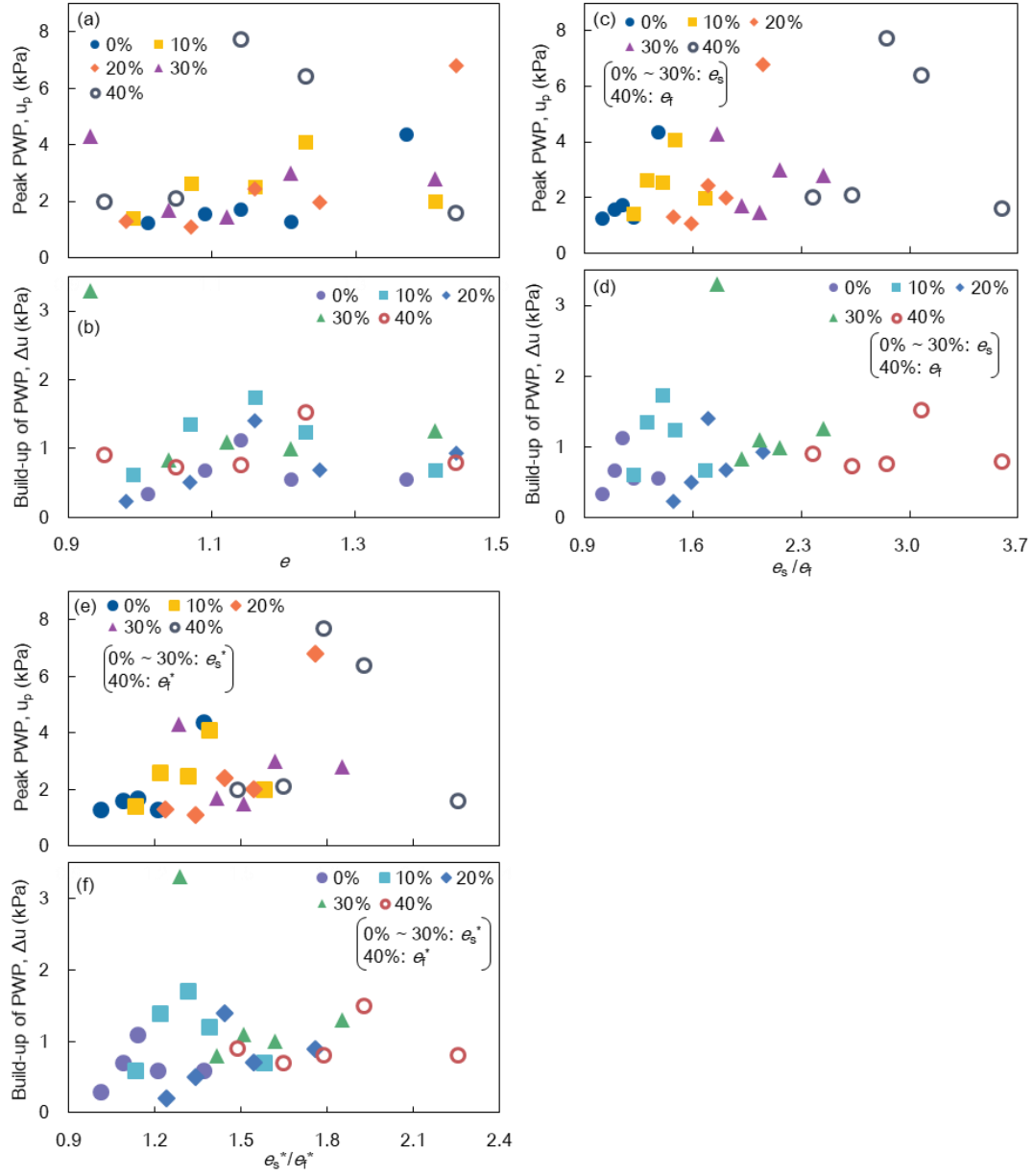


Fig. 15. Peak PWP (u_p) (a, c, e) and build-up of PWP (Δu) (b, d, f) against global void ratio (e), intergranular/interfine void ratio (e_s/e_f) and equivalent intergranular/interfine void ratio (e_s^*/e_f^*) for different samples, respectively.

Table 1. Properties of employed samples.

Sample	Particle density	Minimum dry density	Maximum dry density
	$\rho_s \text{ (g/cm}^3\text{)}$	$\rho_{dmin} \text{ (g/cm}^3\text{)}$	$\rho_{dmax} \text{ (g/cm}^3\text{)}$
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

Table 2. Test conditions and summarized data in all flume tests

Series no.	Test.	Sample	Density index	V_p	u_p	Δu	Note
	no.		I_d	mm/s	kPa	kPa	
1	T-1	S7	-0.54	59.4	4.4	0.6	
	T-2	S7	-0.20	19.3	1.3	0.6	
	T-3	S7	-0.04	10.0	1.7	1.1	
	T-4	S7	0.07	7.6	1.6	0.7	Show in Figs. 5 and 9
	T-5	S7	0.25	6.7	1.3	0.3	
2	T-6	M10	-0.54	1101.5	2.0	0.7	
	T-7	M10	-0.20	260.3	4.1	1.2	Show in Fig. 9
	T-8	M10	-0.06	26.1	2.5	1.7	
	T-9	M10	0.09	19.5	2.6	1.4	
	T-10	M10	0.24	13.5	1.4	0.6	
3	T-11	M20	-0.52	1074.5	6.8	0.9	
	T-12	M20	-0.21	1582.0	2.0	0.7	Show in Fig. 9
	T-13	M20	-0.06	340.3	2.4	1.4	
	T-14	M20	0.08	51.9	1.1	0.5	
	T-15	M20	0.24	28.2	1.3	0.2	
4	T-16	M30	-0.53	1868.0	2.8	1.3	Show in Fig. 9
	T-17	M30	-0.2	2088.0	3.0	1.0	Show in Fig. 9
	T-18	M30	-0.05	1688.0	1.5	1.1	
	T-19	M30	0.08	85.1	1.7	0.8	
	T-20	M30	0.25	60.1	4.3	3.3	
5	T-21	M40	-0.52	1229.5	1.6	0.8	
	T-22	M40	-0.19	1980.0	6.4	1.5	
	T-23	M40	-0.05	2011.0	7.7	0.8	Show in Figs. 6, 8 and 9
	T-24	M40	0.08	1210.0	2.1	0.7	
	T-25	M40	0.24	260.3	2.0	0.9	Show in Figs. 7 and 9

I_d : Density index; V_p : Peak velocity; u_p : Peak PWP; Δu : Build-up of PWP.

Declaration of Competing Interest

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.