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Prediction of landslide velocity and displacement from groundwater level changes

considering the shear rate-dependent friction of sliding zone soil

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Abstract:

In reactivated slow-moving landslides, the prediction of velocity and displacement is a crucial issue for understanding landslide kinematics and realizing warning systems. A variety of data-based numerical models have been used to predict landslide displacement; however, kinematics-based models, which consider the shear behaviors of soils within the sliding zone that fundamentally control landslide movement, have been relatively limited.

In the present research, the Tangjiao landslide in the Three Gorges Reservoir (TGR) in China was taken as a case study. The monitored ground displacement from March 2007 to September 2016 and deformation signs observed in the field survey show that the Tangjiao landslide experienced slow movement with the ground displacement increasing stepwise in response to seasonal rainfall and periodic fluctuation of the reservoir water level. We performed a rate-stepped continuous ring shear test on two specimens remolded from the sliding zone soil at shear rates ranging from 0.01 to 10 mm/s. The test results show that the residual strengths of both specimens show a nonmonotonic shear rate dependency, i.e., a weak negative shear rate dependency (fitted by an exponential law) followed by a significant positive dependency (fitted by a linear law). The transition from a negative shear rate dependency to a positive shear rate dependency may be attributed to the change in

shear mode when the shear rate exceeds a critical value. This complex shear rate-dependent residual behavior regulates the movement of the Tangjiao landslide in the reactivated state. On this basis, a kinematics-based model for predicting landslide velocity and displacement was proposed by taking into account the shear rate-dependent friction of the sliding zone soil. This model can reproduce the velocity and displacement of the Tangjiao landslide from the change in groundwater level (i.e., stress perturbations in this paper). The results suggest that by using this model, the well-investigated material parameters can enhance the prediction reliability and accuracy of landslide behavior. Our proposed approach, with the ability to reliably predict landslide velocity and displacement, such as the Tangjiao landslide in this work, has important implications for predicting movement trends and developing early warning systems for reactivated slow-moving landslides, especially translational landslides under similar geological conditions.

Keywords: Reactivated slow-moving landslide; Prediction of displacement; Sliding zone soil; Shear rate dependency; The Three Gorges Reservoir

1. Introduction

As the most widespread global geological disasters, landslides give rise to serious threats to human life, property and infrastructure due to their frequent occurrence triggered by climate change, earthquakes, human engineering activities, etc. (Petley, 2012; Gariano and Guzzetti, 2016; Froude and Petley, 2018). According to the scale of the displacement rate or movement velocity, the movement of a landslide can be generically defined as extremely slow, very slow, slow, moderate, rapid, very rapid and extremely rapid (Hungr et al., 2014). Differing from rapid movement (sometimes catastrophic failure) with high kinetic energy mobilization, a slow-moving landslide often evolves over several years to decades with a velocity of millimeters to several meters every year and is dominated by frictional sliding and/or viscos-plastic flow along the discrete sliding zone (Agliardi et al., 2020; Lacroix et al., 2020a). Although slow-moving landslides rarely take lives, they can cause serious damage to landforms, houses, infrastructure and agricultural production (e.g., Froude and Petley, 2018; Nappo et al., 2019; Lacroix et al., 2020b). Notably, a slow-moving landslide may also accelerate to rapid movement and catastrophic failure, resulting in fatal consequences, for instance, the Vaiont landslide in Italy (Genevois and Ghirotti, 2005) and the Xinmo landslide and the Baige landslide in China (Fan et al., 2017; Ouyang, et al., 2019). Therefore, the reliable prediction of a slow-moving landslide movement trend and, in particular, its critical transition to rapid movement is a key issue for preventing and mitigating landslide hazards.

The velocity and/or displacement can not only provide insight into the movement patterns of a landslide (e.g., [Leroueil, 2001](#); [Petley et al., 2002, 2005](#)), but also contribute to forecast its failure time (i.e., critical time of the transition from slow-moving to catastrophic failure) by employing the well-known inverse-velocity model (e.g., [Saito, 1965, 1969](#); [Fukuzono, 1985a, 1985b](#); [Voight, 1988](#)) and relevant modified models (e.g., [Intrieri et al., 2018](#); [Zhou et al., 2020](#); [Sharifi et al., 2022](#)). In recent years, various data-based numerical methods have been used to predict landslide displacements based on different mathematical and/or statistical models, e.g., machine-learning models and neural networks. This kind of approach compares the simulated displacements by a reference model to the measured displacements to confirm the feasibility and capability of the model to predict the future displacement ([Intrieri, et al., 2019](#)). [Liu et al. \(2014\)](#) used computational intelligence techniques, including the support vector machine (SVM), the relevance vector machine (RVM) and the Gaussian process (GP), to compare the fitting and predicting displacement series of the Baishuihe landslide in the Three Gorges Reservoir (TGR) in China and the Super-Sauze landslide in the French Alps. In the same area (i.e., the TGR in China), similar investigations were carried out by using machine-learning models such as the extreme learning machine (ELM) model, particle swarm optimization and support vector machine (PSO-SVM) model, support vector regression (SVR) model, and multifeature fusion transfer learning (MFTL) model (e.g., [Huang et al., 2017](#); [Zhou et al., 2018](#); [Miao et al., 2018](#); [Liu et al., 2020](#); [Li et al., 2020](#); [Li et al., 2021](#); [Zhang et al., 2021](#); [Long et al., 2022](#); [Ma et al., 2022](#); [Wang et al., 2022](#)). In addition, neural networks and related improved

models were also exploited to predict the displacement of landslides in the same study area (e.g., [Du et al., 2013](#); [Lian et al., 2015](#); [Yang et al., 2019](#); [Gao et al., 2020](#); [Guo et al., 2020](#)). For the landslide cases in the above literature, the results indicate that the data-based numerical methods are particularly suitable for landslides with displacement rates that vary periodically. Although data-based numerical methods can achieve ideal predictions, they are usually applicable to well-calibrated landslides. Moreover, data-based numerical methods only predict the future deformation trend of a slow-moving landslide because the reference model is dependent on the training of the measured displacement. Therefore, it remains challenging to predict the transition from slow-moving to catastrophic failure due to the lack of clear physical and mechanical mechanisms.

It is well understood that shear strength fundamentally controls the initiation and mobility of a landslide (e.g., [Bromhead, 2013](#); [Schulz and Wang, 2014](#)). From this perspective, some authors have predicted the velocity and/or displacement, as well as the transition to catastrophic failure of a slow-moving landslide based on kinematics-based models, in which the shear behaviors of the soil within the sliding zone have been taken into account. For instance, the Bingham model (e.g., [Angeli et al., 1996](#); [Bernardie, et al., 2015](#)) recognized that the shear strength of a clayey sliding zone soil is composed of the inherent static strength (i.e., cohesion and internal friction angle) and the viscous strength. [Corominas et al. \(2005\)](#) separately used Bingham model and power law model to describe the viscous term in landslide dynamics. Other models regulated by the shear rate-dependent friction law or rate-and-state friction law (RSFL, [Ruina, 1983](#)) were proposed to predict the

landslide velocity and the transition to catastrophic failure of some slow-moving landslides (Handwerger et al., 2016; Alonso et al., 2016; Alonso, 2021). However, to date, studies that reference this kind of kinematics-based model have been relatively limited. Clearly, these models take into account the rate dependency of the shear strength, such as rate-strengthening and rate-weakening, which have been demonstrated by a large number of experimental studies (e.g., Lemos, 1986; Tika et al., 1996; Saito et al., 2007; Wang et al., 2010; Ferri et al., 2011; Wang et al., 2014; Schulz and Wang, 2014; Kimura et al., 2014; Miao et al., 2014; Wang et al., 2017; Hu et al., 2017; Scaringi et al., 2018; Wang et al., 2020; Miao and Wang, 2021; Kang et al., 2022; Miao and Wang, 2022; Wang et al., 2023; among others).

Jurassic strata are well-known slide-prone strata in the TGR in China (Li et al., 2018; Tang et al., 2019), where most landslides are reactivated to move slowly and their cumulative displacements display a stepwise increase with time; for instance, landslides as study cases were investigated in Guo et al. (2020), Zhang et al. (2021), Zhou et al. (2022) and Long et al. (2022). The stepwise increasing displacement indicates that the reactivated landslide in Jurassic strata in the TGR can experience alternating acceleration and deceleration in movements, that is, intermittent reactivation, along an existing sliding surface (Miao et al., 2014). Consequently, it is expected that the sliding zone soil undergoes shearing with different displacement rates. In this case, to predict the landslide velocity using a kinematics-based model, it is valuable to take into account the possible rate-dependent residual behaviors of the sliding zone soil.

In this paper, the Tangjiao landslide, which is a large-scale reactivated landslide in Jurassic strata in the TGR (China), was taken as a case study. We first analyzed the deformation characteristics of this landslide and then prepared two remolded specimens from two boreholes. Rate-stepped continuous ring shear tests were carried out on the two specimens to examine the possible shear rate dependency of the residual strength. The residual strength indices were also determined. To predict the landslide velocity and displacement under stress perturbations, a kinematics-based model, which takes into account the shear rate-dependent friction of the sliding zone soil, was proposed. Finally, the implications of shear rate-dependent residual behaviors for landslide movement and the applicability of the prediction model were discussed.

2. Tangjiao landslide

2.1. Geological setting

The Tangjiao landslide (108°27'49.26"E, 30°49'32.60"N) is located on the right bank of the Yangtze River in Wanzhou District, Chongqing city in the TGR, China (see Fig. 1). The landslide area is an eroded hilly slope landform with a natural slope angle of 5-25°. This area exhibits a dustpan shape in plan view with two gullies that delineate the left and right boundaries (see Fig. 2a). According to the geotechnical investigation report submitted by CREEC Chongqing Survey, Design and Research Institute Co., Ltd., in August 2011, the Tangjiao landslide covered an area of $125 \times 10^4 \text{ m}^2$ with a main sliding direction of 354° and

has a volume of $25 \times 10^6 \text{ m}^3$ with an average thickness of 20 m (thickness ranging from 5-35 m).

The materials of the sliding mass consist of Quaternary deluvial deposits, which include silty clays and gravels (gravel size ranging from 10-50 cm) in a disordered state. The underlying rock stratum is the Middle Jurassic Shaximiao Formation (J_2s), and the lithology is sedimentary rocks with outcrops of interbedded purple red silty sandstone and sandy mudstone. The general occurrence is $170^\circ \angle 5^\circ$. This lithology is characterized as the most slide-prone strata in the TGR and Sichuan Basin, China (Tang et al., 2019; Liu et al., 2020; Xu and Tang, 2023). The sliding surface exposed by drilling (the arrangement of some boreholes, as shown in Fig. 2a) is the soil-rock interface with an average inclination angle of 15° . Fig. 2b shows a typical geological cross section exposed in 2011. The surface water is mainly distributed in gullies on the left and right boundaries and ponds on the landslide mass. The groundwater in the landslide area is mainly composed of pore water in Quaternary deposits, with the water level changing greatly with seasonal rainfall and periodic fluctuation of the reservoir water level (RWL).

2.2. Historical deformation characteristics

After the first impoundment of the TGR on June 15, 2003, and heavy rainfall throughout July 2003, deformation of the Tangjiao landslide began to occur. Afterward, under the joint action of reservoir operation and seasonal rainfall, obvious damage to the ground surface and damage to roads and houses occurred (see Fig. 3), which made it clear

that the Tangjiao landslide was failing in a reactivated state (Le et al., 2012). In March 2007, the government installed twelve Global Positioning System (GPS) monitoring stations (GPS1 to GPS12) and six inclinometer boreholes (QZK1 to QZK6), as well as one rain gauge on the landslide. Ten hydrological boreholes were installed by the China University of Geosciences (Wuhan) in August 2014 (Yang, 2019). Fig. 2a presents the layout of two GPS monitoring stations (GPS3 and GPS6), two inclinometer boreholes (QZK3 and QZK6) and three hydrological holes (SK1-SK3) in the front part of the landslide. According to the field survey and recorded deformation data, the Tangjiao landslide is divided into four deformation areas, i.e., deformation areas A, B, C and D, which are characterized by the largest, moderate, general and weak deformation, respectively (see Fig. 2a).

The time series of the cumulative displacements of GPS3 and GPS6 are plotted in Fig. 4 in the period of March 2007 to September 2016. The rainfall in each month and RWL in each day are also presented in Fig. 4 during this period. From Fig. 4, we find that both of the cumulative displacements of the two GPS stations increased stepwise. Fig. 4c plots the displacement rate of the largest deformation point of GPS6 with the fluctuation rate of the RWL. As shown in Fig. 4c, the displacement rate of GPS6 increased nine times from March 2007 to September 2016, which corresponded to nine decreases in the RWL. In June 2009, the maximum displacement rate reached approximately 7250 mm/month. Moreover, nine times the peak displacement rate is consistent with the rainy season in this period, which is shown in Fig. 4b. According to Fig. 4, for the front part of the Tangjiao landslide, the characteristics of deformation can be summarized as follows: (a) each increase in

cumulative displacement starts in the fall of the RWL and corresponds to the rainy season; and (b) the rise of the RWL did not result in the deformation of the front part of the Tangjiao landslide. These findings indicate that the falling RWL and heavy rainfall mainly caused deformation in the front part of the Tangjiao landslide. As discussed above, the Tangjiao landslide is now undergoing intermittent reactivation in response to seasonal rainfall and periodic fluctuation of the RWL.

2.3. Sampling and specimen preparation

The field and geotechnical surveys were conducted by CREEC Chongqing Survey, Design and Research Institute Co., Ltd., in August 2011. Twenty-six borehole profiles were arranged, with 243 boreholes on the landslide. Many borehole cores were examined, and sliding zone soil samples were collected from two boreholes (ZK1-111 and ZK1-139) in deformation area A (see Fig. 2a and Fig. 5). As shown in Fig. 5, in borehole ZK1-111, an obvious sliding surface with many parallel striations was observed at depths of 24.7-25.0 m. In borehole ZK1-139, a gray green smooth surface was observed at depths of 12.0-12.3 m, which is relatively close to the depth of the sliding surface exposed by QZK6 (see Fig. 2c). The sliding surfaces in the two boreholes indicate that a well-defined failure surface developed in deformation area A of the Tangjiao landslide. In other words, the landslide had experienced long-term reactivated movement along a sliding zone, and the soils within the sliding zone had already reached the residual state (Bromhead, 2013; Schulz and Wang, 2014). In the laboratory, first, the two soil samples were sieved to remove soil particles with

grain sizes greater than 2 mm, and two kinds of remolded soils labeled specimens S111 and S139 were obtained. It has been reported that the residual strength of clayey soils is mainly controlled by the matrix materials (e.g., [Yatabe et al., 1991](#)). Therefore, we assume that the removal of soil particles with grain sizes greater than 2 mm (i.e., 3.6% and 1.7% by weight of specimens S111 and S139, respectively) had a negligible effect on the residual strength of the sliding zone soil. Some basic physical properties are listed in Table 1. The grain size distributions and plasticity charts of the two specimens are presented in Fig. 6a and Fig. 6b, respectively. As a result, both specimens are low liquid limit clays (CL) with the clay-size fraction ($d < 0.002$ mm) of specimen S111 relatively lower than that of specimen S139. Although we did not examine the mineral composition of the two specimens, from our previous studies ([Miao and Wang, 2021, 2022](#)), it was suggested that the clay minerals of sliding zone soil originating in the J_{2s} stratum in the TGR are illite, kaolinite and a mixed layer of illite and smectite (I/S). Fig. 7 shows the soil particle morphologies ($d < 0.075$ mm) of specimens S111 and S139 by means of scanning electron microscopy (SEM). As shown in Fig. 7, the framework minerals of these two specimens exhibit mainly a flaky shape, with some other particles showing an agglomerate shape.

3. Ring shear test

3.1. Test apparatus

The ring shear apparatus enables an infinite shear displacement if no specimen is lost through the chamber gap and can subsequently examine the shear behaviors of a soil in a

residual state. In slope stability analysis, especially for landslides in a reactivated state and/or landslides in the postfailure stage, ring shear apparatuses have been widely used (e.g., Bishop et al., 1971; Hungr and Morgenstern, 1984; Lemos, 1986; Tika et al., 1996; Stark and Contreras, 1996; Sassa et al., 2003; Sassa et al., 2004; Sadrekarimi and Olson, 2009). In this study, the employed ring shear apparatus is DPRI-5 (Sassa et al., 2004) with a shear area of 141.37 cm² (12 cm and 18 cm in inner and outer diameter, respectively) and a maximum height of 11.5 cm. The maximum shear velocity of this apparatus is 100 mm/s. This kind of ring shear apparatus can simulate the formation of a shear zone so that the mobilized shear resistance can be observed, especially the generation of excess pore-water pressure in undrained conditions (Sassa et al., 2004; Fukuoka et al., 2006; Wang et al., 2007). Pore-water pressures near the shear surface (shear zone) are measured by transducers connected to a gutter extending along the whole circumference of the inner wall of the outer ring in the upper part of the shear box. The loss of sample and pore fluid can be precluded by a rubber gasket sealing the separation of the shear box. More details on the design, construction, operation method, and features of this kind of ring shear apparatus (DPRI series) can be found in Sassa et al. (2003, 2004, 2010).

3.2. Test procedure

For each specimen, three test procedures, i.e., specimen placement, consolidation and shearing, were carried out. Before each specimen placement, de-aired water was first added to the oven-dried specimen (dry mass = 2 kg) to make a soil slurry. The soil slurry was stirred

evenly and then kept in a moisturizing container for several days to make it fully saturated. After that, the slurry was placed into the shear box to a maximum filling height of 11.5 cm. The initial dry densities of the two specimens are the same (e.g., 1.23 g/cm³). Following specimen placement, we measured the friction resistance of the rubber gasket. The real shear strength of the specimen can be obtained by subtracting the friction resistance of the rubber gasket from the shear resistance measured by integrated force sensors.

Each specimen was first subjected to a total normal stress (σ_n) of 50 kPa for several minutes in undrained conditions to determine the B_D value (Sassa et al., 2004), which is usually used to check the degree of saturation. The B_D values of both specimens were greater than 0.95, indicating that they were saturated. After that, each specimen was consolidated under a final total normal stress (σ_n) of 310 kPa in drained conditions (by opening the upper and lower drainage lines, the same below). This value of σ_n corresponds to the average normal stress acting on the sliding surface due to the self-weight of the landslide mass. The consolidation processes of specimens S111 and S139 lasted approximately 28 h and 39 h, respectively. After the consolidation stage ended, each specimen was sheared by elevating the shear rate stepwise in drained conditions via a shear speed-controlled method. This continuous shearing enables the investigation of the possible shear rate dependency of the residual strength of the specimen. After that, the specimens were sheared under multiple total normal stresses with the same shear rate (0.01 mm/s) in drained conditions to estimate the residual strength index of each specimen.

3.3. Shear behaviors during continuous shearing by elevating the shear rate stepwise

In the continuous shearing of both specimens, the shear rates were 0.01, 0.02, 0.05, 0.10, 0.20, 0.50, 1.0, 2.0, 5.0 and 10 mm/s, which cover three orders of magnitude. During the completely continuous shearing, the total normal stress was always kept at 310 kPa. Fig. 8 and Fig. 9 plot the shear strength, variation in vertical displacement and pore-water pressure near the shear surface with increasing shear displacement for specimens S111 and S139, respectively. Note that in Fig. 8 and Fig. 9, an increase in the vertical displacement means a decrease in the specimen height and volume; conversely, the specimen height and volume increase.

Fig. 8a and Fig. 9a show that specimen S111 and specimen S139 exhibit similar shear behaviors, i.e., a weak negative shear rate dependency (or rate-weakening) appears at shear rates ranging from 0.01-0.50 mm/s, followed by a significant positive shear rate dependency (or rate-strengthening) occurring at shear rates ranging from 1.0-10 mm/s. As shown in Fig. 8b and Fig. 9b, at shear rates ranging from 0.01-0.50 mm/s, the vertical displacement variation shows continuous growth, indicating a decrease in specimen volume (i.e., shear contraction, see [Terzaghi et al., 1996](#); [Iverson, 2005](#)). In contrast, at shear rates ranging from 1.0-10 mm/s, the vertical displacement variation shows a slight decrease, suggesting an increase in specimen volume (i.e., weak shear dilatancy). It is noted that at the first shear rate of 0.01 mm/s, the decrease in specimen volume is the most significant. Some previous studies have confirmed that the variation in vertical displacement, specifically, the decrease in specimen volume, indicates the formation of a shear zone during shearing (e.g., [Agung et al.,](#)

2004; Fukuoka et al., 2006; Siman-Tov and Brodsky, 2018; Miao and Wang, 2022). These researchers stated that the crushing of coarse particles combined with the migration of fine particles results in the growth of a shear zone, which subsequently causes decreases in the specimen volume. As shown in Fig. 8b and Fig. 9b, at shear rates ranging from 0.01-0.50 mm/s, the decrease in the specimen volume suggests that a shear zone has been formed for each specimen. However, in fast shearing (1.0-10 mm/s), the specimen volume of each specimen shows a slight increase. This may be due to the disturbance of originally arranged flaky and agglomerate particles (see Fig. 7) during fast shearing. This kind of disturbance of grain orientation will change the mode of intergranular shearing (e.g., shear mode, Tika et al., 1996). In other words, for specimens S111 and S139, when the shear rate exceeds 1.0 mm/s, a transition of the shear mode may occur for both specimens.

To date, three main hypotheses have been proposed to interpret the mechanism of shear rate dependency of the residual strength of clayey soils, namely, (a) the build-up of excess pore-water pressure that subsequently results in a change in the residual strength, (b) the change in water content in the shear zone and (c) the change in shear mode with the variation in shear rate. For the first one, it is noted that during the entire shearing of specimen S111 and specimen S139, the pore-water pressure near the shear surface remained almost at zero under drained conditions (by opening the upper and lower drainage lines) (see Fig. 8c and Fig. 9c). Therefore, the effects of excess pore-water pressure on the observed shear rate dependency of both specimens can be excluded. For the negative shear rate dependency, Skempton (1985) and Tika et al. (1996) argued that the increasing water content in the shear zone with

increasing shear rate leads to the reduction in residual strength. As shown in Fig. 8b and Fig. 9b, at shear rates ranging from 0.01-0.50 mm/s, the volumes of the two specimens decrease gradually, leading to the discharge of pore water because of pore volume compression, which temporarily increases the water content of the soils within the shear zone. Therefore, we infer that the negative shear rate dependency of the two specimens may comply with the mechanism of increased water content in the shear zone. For the positive shear rate dependency, it is commonly hypothesized that the transition from the sliding shear mode to the turbulent shear mode is the main mechanism (e.g., [Lemos, 1986](#); [Tika et al., 1996](#); [Saito et al., 2007](#); [Wang et al., 2010](#); [Li and Aydin, 2013](#); [Li et al., 2013](#); [Miao and Wang, 2022](#); [Wang et al., 2023](#)). During fast shearing, the soil particles within the shear zone will lose their original alignment, which makes the turbulent shear mode dominant, leading to an increase in shear strength. Here, we observed a significant positive shear rate dependency of the mobilized shear strength of each specimen at shear rates ranging from 1.0-10 mm/s (see Fig. 8b and Fig. 9b). We infer that this phenomenon observed in our ring shear test results from the transition of the shear mode with increasing shear rate, e.g., from sliding shear to turbulent shear.

3.4. Shear rate-dependent residual strength

For clayey soils, once the deformation becomes localized during drained shearing with a shear rate slower than 0.1 mm/s, the residual state is developed within a shear zone, in which particles have been disaggregated, comminuted, and aligned ([Schulz and Wang, 2014](#)). At this

state, the shear strength of the soil reaches a minimum and is termed the residual strength (e.g., Bishop et al., 1971; Skempton, 1985; Agung et al., 2004; Mitchell and Soga, 2005; Bromhead, 2013). Notwithstanding the fluctuation of the measured shear stress (Li and Aydin, 2013) (Fig. 8a and Fig. 9a), the shear strength at the steady state, in which the volume, shear stress, and normal effective stress of a soil reach a constant (e.g., Poulos, 1981; Wang et al., 2007), was referred to as the residual strength in the present research.

Fig. 10a and Fig. 10b plot the steady state friction coefficient μ_{ss} (i.e., the residual friction coefficient in this study) at each shear rate for specimens S111 and S139, respectively. As shown in Fig. 10a and Fig. 10b, the steady-state friction coefficients μ_{ss} of both specimens S111 and S139 have two different relationships with the shear rate, namely, an exponential law followed by a linear law. We determined the critical shear rate v_c with a value of 0.5 mm/s, which distinguishes these two different shear rate dependencies.

3.5. Residual strength index

For each specimen, when the rate-stepped continuous shearing was finished, ring shear tests were performed under different total normal stresses at the same shear rate of 0.01 mm/s under drained conditions. The total normal stresses were 360 kPa, 410 kPa and 460 kPa. The shear strengths of the two specimens with increasing shear displacement under different total normal stresses are plotted in Fig. 11a and Fig. 11b. Fig. 11c presents the residual strength envelope determined by the linear fit. As shown in Fig. 11c, the effective residual cohesion (c'_r) of specimens S111 and S139 are 52.7 and 58.8 kPa, and the effective residual internal

friction angles (ϕ'_r) are 10.8° and 10.0°, respectively. According to the residual strength index, the effective residual internal friction angles of the two specimens are similar, while the effective residual cohesion of specimen S111 is slightly smaller than that of specimen S139. This result indicates that after experiencing a large shear displacement at a low shear rate (e.g., 0.01 mm/s or less), soil particles within the shear zone have formed a relatively fixed arrangement; meanwhile, the clay-size fraction (CF) will affect the residual strength indices of clayey soils. For instance, specimen S139 with a CF of 21.4% has a c'_r of 58.8 kPa, which is slightly higher than that of specimen S111 with a CF of 18.1% (see Table 1). Our previous study on two specimens remolded from the sliding zone soils of the Xingfusi landslide in the TGR demonstrated that an increase in the clay-size fraction will lead to a reduction in residual frictional resistance ([Miao and Wang, 2021](#)).

4. Landslide velocity and displacement prediction under stress perturbations

The change in groundwater level (GWL) leads to a transient pore-water pressure in the sliding zone resulting in stress perturbations. In fact, these stress perturbations may lead to a variety of landslide movements, such as occasional reactivation, episodic acceleration and deceleration and stable creep, as well as stationary movements in the case of driving forces less than resisting forces ([Davis et al., 1993](#)). To predict the velocity of the Tangjiao landslide attributed to stress perturbations, we developed a kinematics-based model, namely, the shear rate-dependent friction (SRF) model, which comprehensively expresses the shear

rate-dependent shear behaviors of sliding zone soils under the conditions of an infinite slope in planar sliding.

Fig. 12 conceptually presents the diagrammatic sketch of a reservoir-reactivated landslide. As shown in Fig. 12a, the landslide was reactivated along the preexisting sliding surface by seasonal rainfall and an increase in the RWL, forming tension cracks. The variation in landslide displacements was recorded by GPS and the potential sliding zones were exposed by inclinometers. Fig. 12b shows slope components under a 1D infinite slope geometry and stress perturbations resulting from transient precipitation and the fluctuation of the RWL. It is often assumed that the sliding mass is rigid without internal relative movements in infinite slope conditions. Fig. 12c interprets the acting stresses on a sliding slice and relevant parameters. Note that as shown in Fig. 12c, the velocity shows a linear profile over the depth within the range of the sliding zone.

4.1. Description of the shear rate-dependent friction (SRF) model

From a mechanical viewpoint, the dynamic equilibrium equation of a landslide in the conditions of the infinite slope is stated as follows:

$$G \sin \alpha - R = \frac{G}{g} \frac{dv}{dt} \quad (1)$$

where G is the weight of the sliding slice (kN/m^2), α is the inclination of the sliding surface ($^\circ$), R is the resisting stress along the sliding surface (kN/m^2), g is the gravitational acceleration, v is the landslide movement rate or the velocity (m/s) and t is the time (s).

The weight of the sliding slice G is given by:

$$G = \gamma^* H \quad (2)$$

where γ^* is the equivalent unit weight of the sliding slice (kN/m^3) and H is the thickness of the sliding slice (m). It is noted that we did not consider the change in unit weight as a function of the GWL. In this study, the equivalent unit weight of sliding mass γ^* equals 20.0 kN/m^3 .

The resisting stress along the sliding surface R is:

$$R = N' \mu_{ss} = (G \cos \alpha - U) \mu_{ss} \quad (3)$$

where N' is the effective normal stress acting on the sliding surface and U is the pore-water pressure on the sliding surface, which is expressed as:

$$U = \gamma_w (H - h_w) \cos^2 \alpha \quad (4)$$

where γ_w is the unit weight of the water (9.8 kN/m^3) and h_w is the depth of the GWL below the ground surface (m).

Comparing the test results of specimen S111 and specimen S139 (see Fig. 10), we use the data of specimen S139, which has a better statistical fit than that of specimen S111, to represent the whole landslide. As a result, the residual frictional coefficient (μ_{ss}) of the sliding zone soil of the Tangjiao landslide is stated as:

$$\mu_{ss} = 0.30 + 0.05 \exp\left(-12.5 \frac{v}{v_c}\right) \quad \text{for } 0 \leq v \leq v_c \quad (5a)$$

$$\mu_{ss} = 0.29 + 0.01 \left(\frac{v}{v_c}\right) \quad \text{for } v > v_c \quad (5b)$$

where v_c is $5.0 \times 10^{-4} \text{ m/s}$.

Therefore, the dynamic equilibrium equation (1) with the SRF model can be rewritten as:

$$\sin \alpha - \left(\cos \alpha - \frac{U}{\gamma^* H} \right) \mu_{ss} = \frac{1}{g} \frac{dv}{dt} \quad (6)$$

It is often convenient to introduce dimensionless variables. A dimensionless time t_d , velocity v_d and stress perturbations U_d , may be defined by

$$t_d = \frac{gt}{v_c}, \quad v_d = \frac{v}{v_c}, \quad U_d = \frac{U}{\gamma^* H} \quad (7)$$

In this case, the dynamic equilibrium equation (6) can be rewritten as:

$$\sin \alpha - (\cos \alpha - U_d) [0.30 + 0.05 \exp(-12.5v_d)] = \frac{dv_d}{dt_d} \quad \text{for } 0 \leq v_d \leq 1 \quad (8a)$$

$$\sin \alpha - (\cos \alpha - U_d) (0.29 + 0.01v_d) = \frac{dv_d}{dt_d} \quad \text{for } v_d > 1 \quad (8b)$$

Eq. (8) provides a piecewise differential equation describing the mobility of the Tangjiao landslide in a dimensionless form, and the exponential-law friction function renders the dynamics equation highly nonlinear when $0 \leq v_d \leq 1$ (i.e., $v < v_c = 5.0 \times 10^{-4}$ m/s). In Eq. (8), U_d represents the stress perturbations. For $v_d = 0$, using the average geometrical parameters of the front part of the Tangjiao landslide ($\alpha = 15^\circ$; $H = 15$ m, see deformation area A in Fig. 2b), the landslide will be statically stable for values of U_d equal to or less than approximately 0.226. To solve this piecewise differential equation, a fourth-fifth order Runge–Kutta numerical integration method (i.e., ode45 in MATLAB) can be used. Therefore, Eq. (8) can simulate a final velocity for each stress perturbation. As an illustrative example, Fig. 13 plots the simulating results with U_d values of 0.228, 0.236, 0.336 and 0.436. Note that the maximum value of U_d is approximately 0.457 according to Eq. (4) and Eq. (7).

As shown in Fig. 13, for each U_d , the dimensionless velocity v_d or velocity v maintains a stable value after a rapid increase within an extremely short time, i.e., less than one second according to Eq. (7), in which $g = 10$ m/s² and $v_c = 5 \times 10^{-4}$ m/s. The results indicate that if

instability occurs for the Tangjiao landslide (i.e., $U_d > 0.226$), it will keep moving at a certain final velocity. In addition, velocities calculated by Eq. (7) are approximately 262, 279, 522 and 857 m/d, corresponding to U_d values of 0.228, 0.236, 0.336 and 0.436, respectively. It is obvious that the calculated velocities appears to be too high compared with the field observations on the Tangjiao landslide (see Fig. 4), indicating an unreliable prediction effect with the current SRF model.

To achieve more realistic predicted landslide velocities, some approaches have taken into account the viscous component of resisting forces to govern the acceleration or deceleration of a slow-moving landslide with clayey sliding zones (e.g., [Vulliet and Hutter, 1988](#); [Van Asch et al., 1989](#); [Van Genuchten, 1990](#); [Van Asch and Van Steijn, 1991](#); [Angeli et al., 1996](#); [Corominas et al., 2005](#); [Van Asch et al., 2007](#); [Li et al., 2023](#)). In these studies, Bingham's viscous (BV) model is frequently used, which is expressed as:

$$\tau - \tau_0 = \eta \frac{dv}{dz} = \eta \dot{\gamma} \quad (9a)$$

$$\tau_0 = (\sigma - u) \tan \varphi'_r \quad (9b)$$

where τ is the shear stress (kPa), τ_0 is the yield strength (kPa), η is the dynamic viscosity (kPa·s), v is the velocity or shear rate (m/s), z is the depth and $\dot{\gamma}$ is the shear strain rate. σ is the total normal stress (kPa), u is the pore-water pressure (kPa) and φ'_r is the residual friction angle of the sliding zone soils (the residual cohesion is assumed to be zero).

For a narrow sliding zone, a linear relation between the velocity and the thickness of the sliding zone may be assumed in the BV model ([Yen, 1969](#); [Angeli et al., 1996](#); [Bogaard, 2001](#)), which is stated as:

$$v = \frac{z_m}{\eta} (\tau - \tau_0) \quad (10)$$

where z_m is the thickness of the sliding zone (m) and the dynamic viscosity (η) is assumed to be constant.

Many authors found a nonlinearity of v with τ for large ranges of $(\tau - \tau_0)$ both in laboratory tests and back-analysis in the field (e.g., [Vulliet and Hutter, 1988](#); [Bracegirdle et al., 1991](#); [Van Asch et al., 2007](#)). It can be expressed as:

$$v = \frac{z_m}{\eta_0} (\tau - \tau_0)^b \quad (11)$$

where b is interpreted as a variation of the viscosity with the applicable $(\tau - \tau_0)$, and η_0 is the intrinsic viscosity of the sliding zone soil (kPa·s).

[Van Asch et al. \(2007\)](#) have already pointed out that the validity of laboratory-scale soil viscosity should be considered when one uses the BV model to predict landslide velocities in the field. In their study, laboratory-scale intrinsic viscosities of sliding zone soils from three landslides were 10 to 1000 times lower than field-scale ones through back analysis. In reality, our ring shear test results have entirely reflected the shear behaviors of the sliding zone soil of the Tangjiao landslide, including the viscous component in the case of Bingham fluid. Here, to examine the intrinsic viscosity (η_0) of the sliding zone soil at the laboratory-scale, Eq. (11) was used by replacing z_m with the thickness of the shear zone (hereafter labeled z_s) that formed during rate-stepped continuous shearing. Note that we did not measure the thickness of the shear zone after the ring shear test was finished. However, from our previous studies with the DPRI ring shear apparatus ([Miao and Wang, 2021, 2022](#)), it was presented that an

approximately 10 mm thickness of a shear zone can be formed after the ring shear test on the sliding zone soil collected from a landslide in the same study area (i.e., J₂s stratum in Wanzhou in the TGR). Therefore, 10 mm of z_s is assumed in Eq. (11). Moreover, the range of shear rates from 0.5 to 10 mm/s was adopted due to the positive shear rate dependency of the residual strength, which is in line with Bingham's rheological behavior. Fig. 14 plots the fitting curve of shear rate v with $(\tau - \tau_0)$. From Fig. 14, the calculated laboratory-scale intrinsic viscosity ($\eta_{0(\text{lab})}$) of the sliding zone soil is 3.91×10^4 kPa·s. This value of η_0 seems to be rather lower than that usually used to predict landslide velocities in the field, for instance, $n \times 10^7$ kPa·s ($n = 1 \sim 2$) (e.g., [Corominas et al., 2005](#); [Van Asch et al., 2007](#)). This can also be verified for the Tangjiao landslide by comparing the laboratory value with the back-analyzed estimate based on field observations.

For estimating the field-scale intrinsic viscosity, the observed velocities recorded by GPS6 were adopted in the conditions of infinite slope with landslide geometrical parameters of $\alpha = 15^\circ$ and $H = 15$ m. $(\tau - \tau_0)$ was calculated with the given groundwater level (GWL) and the residual friction angle of 10.0° . However, due to the lack of one-to-one correspondence among the monitoring points of displacement and the GWL, the average recorded GWLs of SK1 to SK3 in the front part of the Tangjiao landslide ([Yang, 2019](#), see Fig. 2a) were utilized. The period covers one hydrological year from September 1, 2014, to August 31, 2015. The average daily groundwater depth below the ground surface (h_w) and rainfall, as well as the daily RWL, are plotted in Fig. 15. Note that the GWL data of SK1, SK2 and SK3 were digitized from [Huang et al. \(2018\)](#). Here, we performed a simple back analysis by using Eq.

(10) with the average h_w (i.e., 7.731 m, see Fig. 15a) and velocity of GPS6 (i.e., 6.23 mm/d, see Fig. 4) in the period from April 1, 2015, to June 30, 2015. This period covers the time area of the main deformation of the local point of GPS6 in the above one hydrological year. Note that the thickness of the sliding zone was assumed to be 10 mm, which is the same as that determined by the ring shear test. By using Eq. (10), the calculated field-scale intrinsic viscosity ($\eta_{0(\text{field})}$) is 5.06×10^6 kPa·s, which is approximately 130 times higher than the laboratory-scale viscosity obtained from the ring shear test.

Van Asch et al. (2007) presented several factors to explain the discrepancy between field-scale and laboratory-scale viscosity parameters. They stated that (1) the real stability of a landslide might be higher than that calculated in the 1D infinite slope conditions, resulting in larger viscosities; (2) the intrinsic viscosity η_0 increases linearly with the effective normal stress; and (3) the thickness of the sliding zone cannot readily be reproduced under laboratory conditions. The first factor is attributed to the disregarding of resistances that exist on the side of the slice in the landslide complex with a 1D infinite slope model. Moreover, the 1D infinite slope model simplifies the sliding surface to a plane. However, the basal and lateral sliding surfaces of landslides in soils are often irregular. In this case, the slide mass deformation during movement along non-planar sliding zones adds resistance, especially for clayey landslides (e.g., Mizuno, 1989; Baum and Johnson, 1993). Since the normal stress levels applied in the ring shear test in this study approached those experienced in the field, the second fact cannot give rise to the observed discrepancy between the field-scale and laboratory-scale values of the viscosity. However, as described above, the thickness of the

shear zone formed during the ring shear test was smaller than that observed in the field. This low value of the thickness of the shear zone leads to a low intrinsic viscosity η_0 when it was determined by fitting laboratory data (e.g., Fig. 14). In this case, one may expect this thickness to be dozens of times higher, for instance, approximately 30 cm in [Corominas et al. \(2005\)](#).

As discussed above, the discrepancy between field-scale and laboratory-scale viscosity parameters indicates a disparity between laboratory-scale and the field-scale sliding resistance, which will make unreliable predictions of landslide velocities at the field-scale by using the proposed SRF model described by Eq. (8). Therefore, to reduce the distance of the laboratory-scale velocities from the field-scale ones, some modifications should be made in Eq. (8). First, the viscous component at laboratory-scale is subtracted from the frictional strength measured by the ring shear test; in this way, a yield shear strength at each shear rate is obtained which is usually regraded as the threshold value for the materials showing Bingham's rheological behavior ([Bracegirdle, 1991](#); [Van Asch et al., 2007](#)). In addition, this yield shear strength can also be expressed by the Mohr–Coulomb yield criterion ([Li et al., 2023](#)). After subtracting the laboratory-scale viscous component, a field-scale viscous component was supplemented to the side of resisting stress to reflect the Bingham's rheological behavior of sliding zone soil. By such an approach, the final dynamic equilibrium equation with the SRF model is rewritten as follows:

for $0 \leq v_d \leq 1$,

$$\sin \alpha - \left\{ (\cos \alpha - U_d) [0.30 + 0.05 \exp(-12.5v_d)] - \frac{v_c \eta_{0(\text{lab})}}{\gamma^* H z_s} v_d \right\} - \frac{v_c \eta_{0(\text{field})}}{\gamma^* H z_m} v_d = \frac{dv_d}{dt_d} \quad (12a)$$

and for $v_d > 1$,

$$\sin \alpha - \left[(\cos \alpha - U_d)(0.29 + 0.01v_d) - \frac{v_c \eta_{0(\text{lab})}}{\gamma^* H z_s} v_d \right] - \frac{v_c \eta_{0(\text{field})}}{\gamma^* H z_m} v_d = \frac{dv_d}{dt_d} \quad (12b)$$

where $\eta_{0(\text{field})}$ is the field-scale intrinsic viscosity obtained through back analysis (kPa·s) and $\eta_{0(\text{lab})}$ is the laboratory-scale intrinsic viscosity determined by the ring shear test (kPa·s); z_s is the shear zone thickness obtained from the ring shear test.

To illustrate the capability of the SRF model described by Eq. (12), we also calculated the dimensionless velocity v_d and velocity v by using U_d values of 0.228, 0.236, 0.336 and 0.436 with the geometrical and viscosity parameters listed in Table 2. The results are plotted in Fig. 16. The velocities calculated by Eq. (7) are approximately 1.11, 6.83, 78.1 and 148.9 mm/d, corresponding to U_d values of 0.228, 0.236, 0.336 and 0.436, respectively. These values of the predicted velocities using Eq. (12) are more in line with the observed velocities in the field for the Tangjiao landslide compared with those predicted by Eq. (8). However, it should be noted here that the field-scale viscous strength in Eq. (12) is supplemented after subtracting the viscous term involved in the frictional strength measured in the laboratory. In this way, the discrepancy between field-scale and laboratory-scale viscosities can be considered, bringing predicted velocities close to that observed in the field.

4.2. Predicted velocity and displacement of the Tangjiao landslide

In the present research, we considered only the changes in groundwater level (GWL) that subsequently result in variations in pore-water pressure in the sliding zone regarded as the stress perturbations in the Tangjiao landslide. Accordingly, it is of great importance to

determine the GWL to predict the velocity and displacement of the landslide. As mentioned above, the average recorded GWLs of SK1 to SK3 (see Fig. 15a) in the front part of the Tangjiao landslide were utilized in the period from September 1, 2014, to August 31, 2015. As shown in Fig. 15a, the maximum variation in h_w is approximately 0.7 m. Meanwhile, it can be seen that when the RWL rises combined with rainfall, the h_w generally decreases, which indicates that the GWL rises. Moreover, with the slow fall of the RWL in the dry season, the h_w shows a fluctuating increase, while with the rapid fall of the RWL, the h_w shows an obvious increase despite the rainy season. The variation in h_w indicates that the influence of the RWL on the groundwater level of the Tangjiao landslide is more significant than that of rainfall.

We calculated the dimensionless stress perturbations U_d , which are plotted in Fig. 17a, by using Eq. (4) and Eq. (7) with the average daily h_w (Fig. 15a) and the parameters listed in Table 2. By solving Eq. (12) with ode45 in MATLAB, the dimensionless velocity v_d can be simulated, and then the landslide velocity is calculated according to Eq. (7). Fig. 17b plots the predicted daily velocity of the Tangjiao landslide. As shown in Fig. 17a and Fig. 17b, the landslide velocity has high consistency with the stress perturbations. Note that the velocity is zero when U_d is less than approximately 0.226 with the parameters listed in Table 2. Fig. 17c presents the predicted cumulative displacement and the recorded displacement of GPS6 in the period from September 1, 2014, to August 31, 2015. The predicted cumulative displacement is approximately 823 mm, which is slightly larger than that of GPS6 (i.e., 668.8 mm). It should be noted that we did not calibrate the parameters in our highly idealized model to

optimize the fit to the recorded displacement data due to the lack of one-to-one correspondence among the local recorded displacements and the GWL, as previously mentioned. Nevertheless, our model can successfully reproduce the general patterns of deformation that are observed in the front part of the Tangjiao landslide.

5. Discussion

5.1. Shear rate-dependent residual behavior and its implications for landslide reactivation

Our tests on two sliding zone soil specimens from the Tangjiao landslide present the existence of complex residual behaviors of the sliding zone soil, as shown in Figs. 8-10. Similar test results of sliding zone soils in Jurassic and Triassic strata in China's TGR and Sichuan basin (Hu et al., 2017; Scaringi et al., 2018; Miao and Wang, 2021; Kang et al., 2022; Miao and Wang, 2022), as well as landslides in Niigata in Japan (Wang et al., 2023), were also observed. The shear rate v_c , which is a critical rate distinguishing two different shear rate effects on the residual strength, is 5×10^{-4} m/s in this study. Our previous studies have demonstrated that the critical shear rate v_c is dependent on the clay fraction for clayey soil (e.g., the sliding zone soil in Jurassic strata in the TGR) (Miao and Wang, 2021, 2022).

Furthermore, the value of v_c (5×10^{-4} m/s) corresponds to a landslide velocity of 1.3 m/month, which can be characterized as slow movement within the range from 13 m/month to 1.6 m/year (Fell et al., 2000; Leroueil, 2001; Hungr et al., 2014). According to Fig. 10, it is suggested that the negative shear rate-dependent residual behavior with an exponential

law regulates the landslide in slow movement, while a linear positive shear rate-dependent residual behavior controls the rapid movement. During stress perturbations (Agliardi et al., 2020), the landslide will accelerate owing to the reduction in effective normal stress, and the acceleration reduces the residual strength, leading to a larger velocity because of the negative shear rate dependency. If the generated velocity is greater than 1.3 m/month, then the positive shear rate dependency of residual strength will lead to a deceleration of the landslide. This repeated acceleration and deceleration led to the intermittent reactivation of the Tangjiao landslide. However, due to the negative shear rate dependency, a relatively large velocity may occur during the slow movement of the Tangjiao landslide, for instance, a displacement rate of 7250 mm/month in June 2009 (see Fig. 4). Moreover, due to the positive shear rate dependency of residual strength in rapid movement, we infer that the Tangjiao landslide will not exhibit catastrophic failure with high velocity in the future. Nevertheless, severe deformation is unavoidable in the reactivated stage (see Fig. 3).

5.2. Applicability of the SRF model

The SRF model proposed in this study involves the shear rate-dependent frictional strength of the sliding zone soils, indicating that it has a clear kinematical mechanism for an active landslide. Actually, the frictional strength measured by the ring shear test has already reflected the shear behaviors of the sliding zone soils, including the viscous component if one regards the clayey sliding zone soil as the Bingham fluid. However, by comparing the intrinsic viscosity determined in the laboratory with that obtained through back analysis

based on field observations, it is suggested that there exists a considerable discrepancy between these two values. This discrepancy examined in the present research is not new; [Van Asch et al. \(2007\)](#) discussed the mechanism explaining the discrepancy between laboratory-scale and field-scale viscosities in depth. The predicted velocities in an illustrated example (see Fig. 13 and Fig. 16) have demonstrated that disregarding the discrepancy between laboratory-scale and field-scale viscosities will lead to an unreliable prediction of landslide velocities. Therefore, in our final proposed SRF model, the laboratory-scale viscous component is first subtracted from the frictional strength measured in the ring shear test, and then the field-scale viscous component is supplemented on the resistance side.

In this study, as previously mentioned, we used the average data of the GWLs from three hydraulic boreholes (SK1 to SK3 in the front part of the landslide) as the stress perturbations to predict the velocity and displacement. However, this average GWL cannot accurately represent that of any displacement monitoring point (e.g., GPS6). Therefore, accurate groundwater level data will be needed to contribute to the applicability of the SRF model.

It is also noted that an infinite slope scheme, widely employed in mechanical analyses of slope instability (e.g., [Duncan and Wright, 2005](#); [Montrasio and Valentino, 2008](#); [Lade, 2010](#)), was assumed in the SRF model. The scheme considers a landslide to be a rigid block with a planar sliding surface, as previously mentioned. In this case, once the landslide is initiated, the sliding mass will not disintegrate during its movement. However, as shown in

Fig. 3, the signs of deformation indicate that the movement of the Tangjiao landslide does not strictly comply with the assumption of an infinite slope; therefore, we suggest that this assumption affects the prediction accuracy and reliability of landslide behavior to some extent.

On the other hand, the SRF model was more suitable for predicting the velocity and displacement of a local point with a one-to-one correspondence between the observed displacement and the GWL. The extension of the results to other local points should be performed carefully if material parameters are well investigated. For a particular landslide, e.g., a translational landslide, the SRF model proposed here may become a promising approach for predicting landslide behavior and developing early warning systems. Finally, although the calibration of parameters in the SRF model has not been carried out to fit the recorded displacement data, the predicted velocity and displacement of the Tangjiao landslide suggest that for the prediction of translational landslides under similar geological conditions, the shear rate-dependent frictional strength of the sliding zone soil should be considered. Further studies will focus on the prediction performance of well-instrumented translational landslides from a global perspective instead of individual boreholes.

6. Conclusions

In this study, the deformation characteristics of a large-scale reactivated landslide in the Jurassic strata in the TGR, China, were analyzed first. Ring shear tests on two specimens remolded from the sliding zone soils in two boreholes were carried out to examine the shear

rate dependency of the residual strength by increasing the shear rate stepwise. The residual strength indices were also determined. On this basis, a kinematics-based model taking into account the shear rate-dependent frictional strength of the sliding zone soil was proposed to predict the landslide velocity and displacement. Finally, the implications for landslide reactivation from the shear rate dependency and the applicability of the prediction model were discussed. The following conclusions can be drawn:

(1) The Tangjiao landslide is a slow-moving landslide that is now undergoing intermittent reactivation with the ground displacement increasing stepwise as a result of the seasonal rainfall and periodic fluctuation of the RWL.

(2) The sliding zone soil of the Tangjiao landslide shows a nonmonotonic shear rate dependency of the residual strength, i.e., a weak negative shear rate dependency (fitted by an exponential law), followed by a significant positive shear rate dependency (fitted by a linear law). The transition of shear rate dependency may be due to the change in the shear mode when the shear rate exceeds a critical value (i.e., 0.5 mm/s). This complex shear rate dependency of the residual strength regulates the reactivated movement of the Tangjiao landslide.

(3) The intrinsic viscosity of the sliding zone soil of the Tangjiao landslide obtained by back analysis is approximately 130 times higher than that determined by the ring shear test. This discrepancy between the laboratory-scale and the field-scale intrinsic viscosities indicates a disparity between laboratory-scale and the field-scale sliding resistance, which will lead to unreliable prediction of the landslide velocities.

(4) The SRF model considering the shear rate-dependent frictional strength of sliding zone soil can reproduce the velocity and displacement of the Tangjiao landslide from the changes in the recorded groundwater level. When using this model, well-investigated material parameters and accurate recorded groundwater levels would enhance the prediction reliability and accuracy of landslide behavior.

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Captions:

Fig. 1. Location of the Tangjiao landslide.

Fig. 2. Geological setting, monitoring and surveying of the Tangjiao landslide. (a) Arrangement of monitoring and surveying and deformation area division on the image from Google Earth; (b) 9-9' cross section covering deformation areas A and B with the boreholes; (c) displacement versus depth measured by inclinometer in QZK6 (from Yang, 2019), indicating a depth of approximately 9.3 m of the located sliding surface. Note: CREEC Chongqing Survey, Design and Research Institute Co., Ltd., designed 26 borehole profiles with 243 boreholes during geotechnical surveying in August 2011. Fig. 2a shows only three borehole profiles. The magenta dots represent the locations of two inclinometer boreholes (QZK3 and QZK6). The pink dots represent the locations of three hydrological boreholes installed by the China University of Geosciences (Wuhan) (from Yang, 2019).

Fig. 3. Signs of deformation in deformation area A of the Tangjiao landslide: (a) left boundary (a gully); (b) ground surface cracks; (c) small-scale local collapse; (d) and (e) road damage; and (f) sliding of GPS6 station and QZK3 (inclinometer).

Fig. 4. Time series of the recorded reservoir water level (RWL), monthly rainfall and cumulative displacement as well as the calculated displacement rate of typical monitoring GPS stations of the Tangjiao landslide from March 2007 to September 2016. (a) RWL and cumulative displacement of GPS6; (b) monthly rainfall and cumulative displacement of GPS3; and (c) fluctuation rate of the RWL and displacement rate of GPS6.

Fig. 5. Sampling of sliding zone soil from the Tangjiao landslide: (a) borehole ZK1-111 and (b) borehole ZK1-139. Note: the locations of the two boreholes are shown in Fig. 2a.

Fig. 6. (a) Grain size distributions of specimens S111 and S139 and (b) plasticity charts of specimens S111 and S139.

Fig. 7. SEM micro-images of the soils with particle sizes less than 0.075 mm: (a) sample S111 and (b) sample S139.

Fig. 8. Drained ring shear test on the saturated specimen S111 with increasing shear displacement rate at the same total normal stress of 310 kPa. (a) Shear strength versus shear displacement during the entire shearing; (b) vertical displacement variation versus shear displacement; and (c) pore-water pressure near the shear surface versus shear displacement.

Fig. 9. Drained ring shear test on the saturated specimen S139 with increasing shear displacement rate at the same total normal stress of 310 kPa. (a) Shear strength versus shear displacement during the entire shearing; (b) vertical displacement variation versus shear displacement; and (c) pore-water pressure near the shear surface versus shear displacement.

Fig. 10. Relationship between the friction coefficient in the steady state (i.e., residual state in this study) (μ_{ss}) and the shear displacement rate. (a) Specimen S111 and (b) specimen S139. v_c is defined as the critical shear rate at which a negative shear rate dependency of the

residual strength transits into a positive shear rate dependency. In other words, v_c is the reference shear rate that distinguishes these two different shear rate dependencies.

Fig. 11. Drained ring shear test on the two saturated specimens under different total normal stresses (σ_n). (a) Shear strength of specimen S111 versus shear displacement under different total normal stresses; (b) shear strength of specimen S139 versus shear displacement under different total normal stresses; and (c) residual strength envelopes of the two specimens.

Fig. 12. Conceptual diagram for the reservoir ancient landslide in the stage of intermittent reactivation. (a) Reservoir ancient landslide with the preexisting sliding surface and the monitoring system such as GPS and inclinometer; (b) slope components under 1D infinite slope geometry and stress perturbations resulting from transient precipitation and reservoir water level fluctuation; and (c) interpretation of acting stresses on a sliding slice and relevant parameters. Note: as shown in Fig. 12c, a linear velocity profile over the depth is assumed within the range of the sliding zone.

Fig. 13. Calculated dimensionless velocity v_d and velocity v of the Tangjiao landslide corresponding to the imposed dimensionless stress perturbations U_d by using Eq. (8) with parameters $\alpha = 15^\circ$ and $H = 15$ m. (a) Dimensionless velocity v_d versus dimensionless time t_d , and (b) velocity v versus time t .

Fig. 14. The fitting curve of the shear rate $\dot{\gamma}$ with $(\tau - \tau_0)$ follows the form of Eq. (11). Note that the unit of the thickness of the shear zone (z_m) is mm, and τ_0 is calculated by using Eq. (9b) with a residual friction angle ϕ'_r of 10.0° , as shown in Fig. 11c. The data of τ and u are obtained from Fig. 9 with a total normal stress σ of 310 kPa.

Fig. 15. (a) Daily groundwater level depth below the ground surface (h_w), (b) daily rainfall and (c) daily reservoir water level (RWL) in one hydrological year (September 1, 2014 to August 31, 2015). Note: h_w was determined as the average groundwater level depth of SK1, SK2 and SK3, whose locations are shown in Fig. 2a; the value of h_w of 7.731 m is the average value in the period from April 1 to June 30, 2015.

Fig. 16. Calculated dimensionless velocity v_d and velocity v of the Tangjiao landslide corresponding to the imposed dimensionless stress perturbations U_d by using Eq. (12) with the parameters listed in Table 2. (a) Dimensionless velocity v_d versus dimensionless time t_d , and (b) velocity v versus time t .

Fig. 17. Predicted velocity and cumulative displacement of the Tangjiao landslide under stress perturbations in the period from September 1, 2014, to August 31, 2015, by using the proposed SRF model. (a) Imposed dimensionless stress perturbation U_d according to the daily h_w , as shown in Fig. 15a; (b) predicted velocity of the Tangjiao landslide with the parameters listed in Table 2; (c) comparison of the predicted cumulative displacement and recorded displacement of GPS6.

Table 1. Basic physical properties of the specimens.

Properties index	G_s	CF (%)	W_L (%)	W_P (%)	PI
S111	2.70	18.1	43.78	23.84	19.94
S139	2.71	21.4	47.56	25.70	21.86

Note: G_s : specific gravity of the soil particles; CF: clay-size fraction by weight; W_L : liquid limit; W_P : plastic limit; PI: plasticity index.

Table 2. Parameters of the Tangjiao landslide in the prediction model.

α (°)	H (m)	γ^* (kN/m ³)	h_w (m)	z_m (m)	z_s (m)	v_c (m/s)	$\eta_{0(\text{field})}$ (kPa·s)	$\eta_{0(\text{lab})}$ (kPa·s)
15.0	15.0	20.0	Fig. 15a	0.30	0.01	5.4×10^{-4}	5.06×10^6	3.91×10^4

Note: α and H are determined by the 9-9' geological cross section shown in Fig. 2b. The value of daily h_w is given in Fig. 15a. The value of z_m is well-defined by two boreholes (see Fig. 5), and the value of z_s is estimated by the ring shear test. The values of $\eta_{0(\text{field})}$ and $\eta_{0(\text{lab})}$ were determined by back analysis and ring shear testing, respectively.

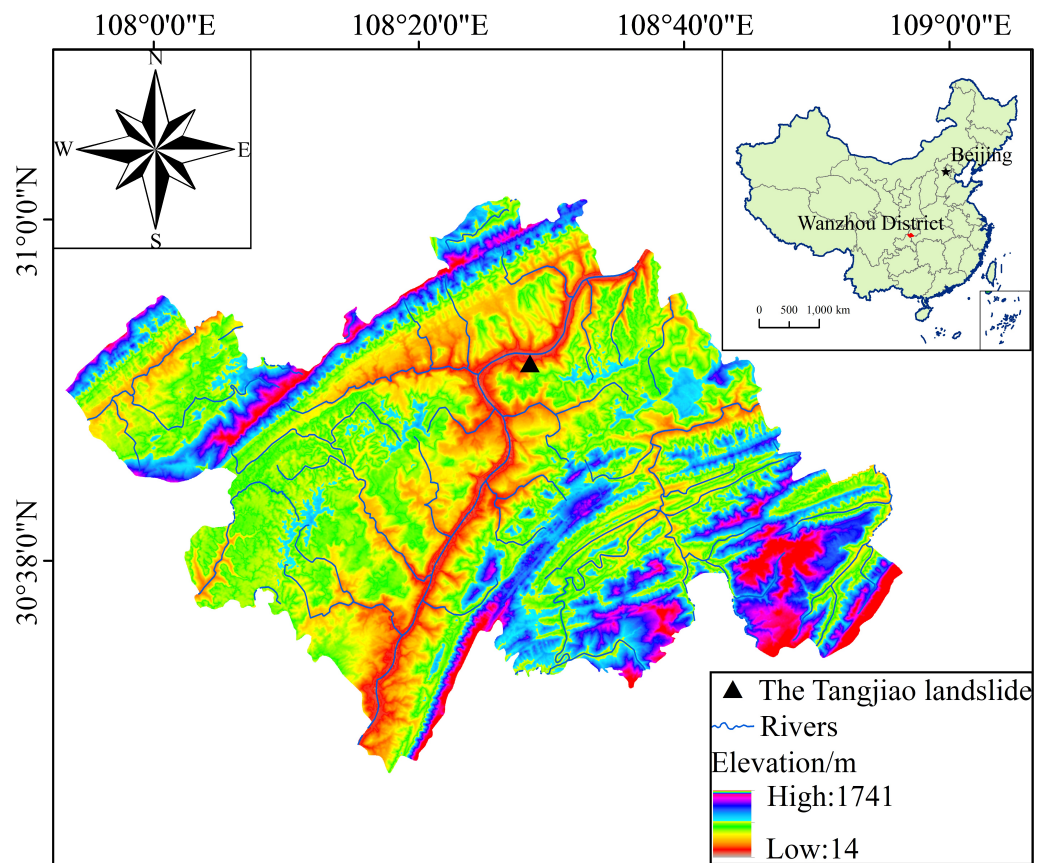


Fig. 1. Location of the Tangjiao landslide.

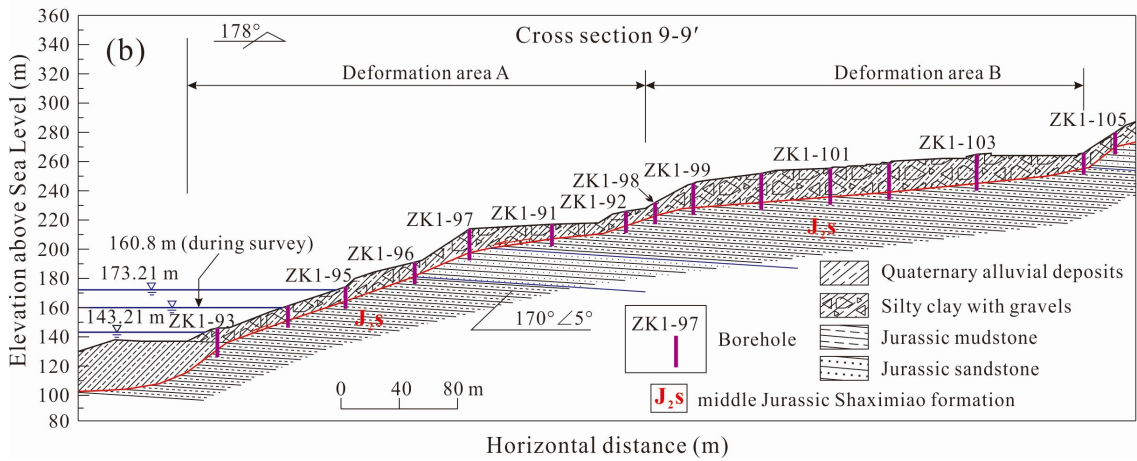
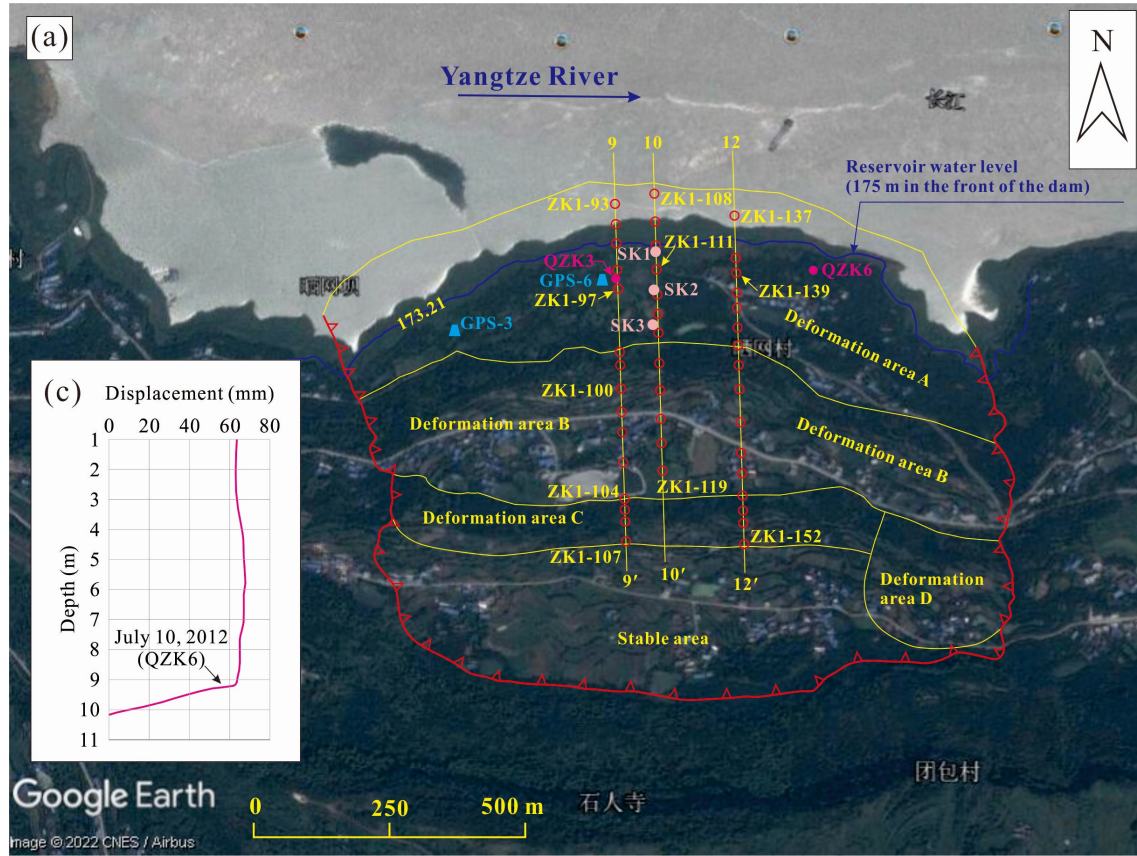


Fig. 2. Geological setting, monitoring and surveying of the Tangjiao landslide. (a) Arrangement of monitoring and surveying and deformation area division on the image from Google Earth; (b) 9-9' cross section covering deformation areas A and B with the boreholes; (c) displacement versus depth measured by inclinometer in QZK6 (from Yang, 2019), indicating a depth of approximately 9.3 m of the located sliding surface. Note: CREEC Chongqing Survey, Design and Research Institute Co., Ltd., designed 26 borehole profiles with 243 boreholes during geotechnical surveying in August 2011. Fig. 2a shows only three borehole profiles. The magenta dots represent the locations of two inclinometer boreholes (QZK3 and QZK6). The pink dots represent the locations of three hydrological boreholes installed by the China University of Geosciences (Wuhan) (from Yang, 2019).

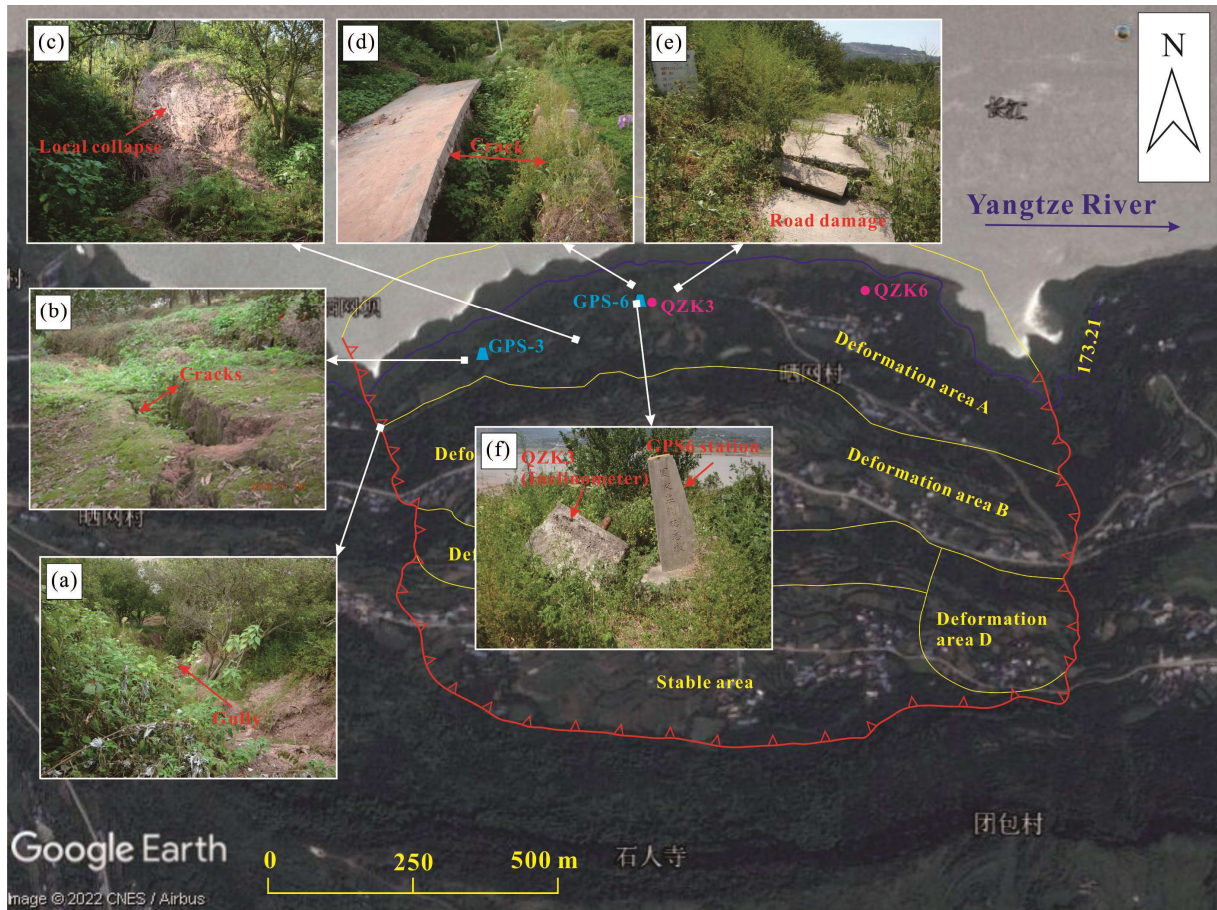


Fig. 3. Signs of deformation in deformation area A of the Tangjiao landslide: (a) left boundary (a gully); (b) ground surface cracks; (c) small-scale local collapse; (d) and (e) road damage; and (f) sliding of GPS6 station and QZK3 (inclinometer).

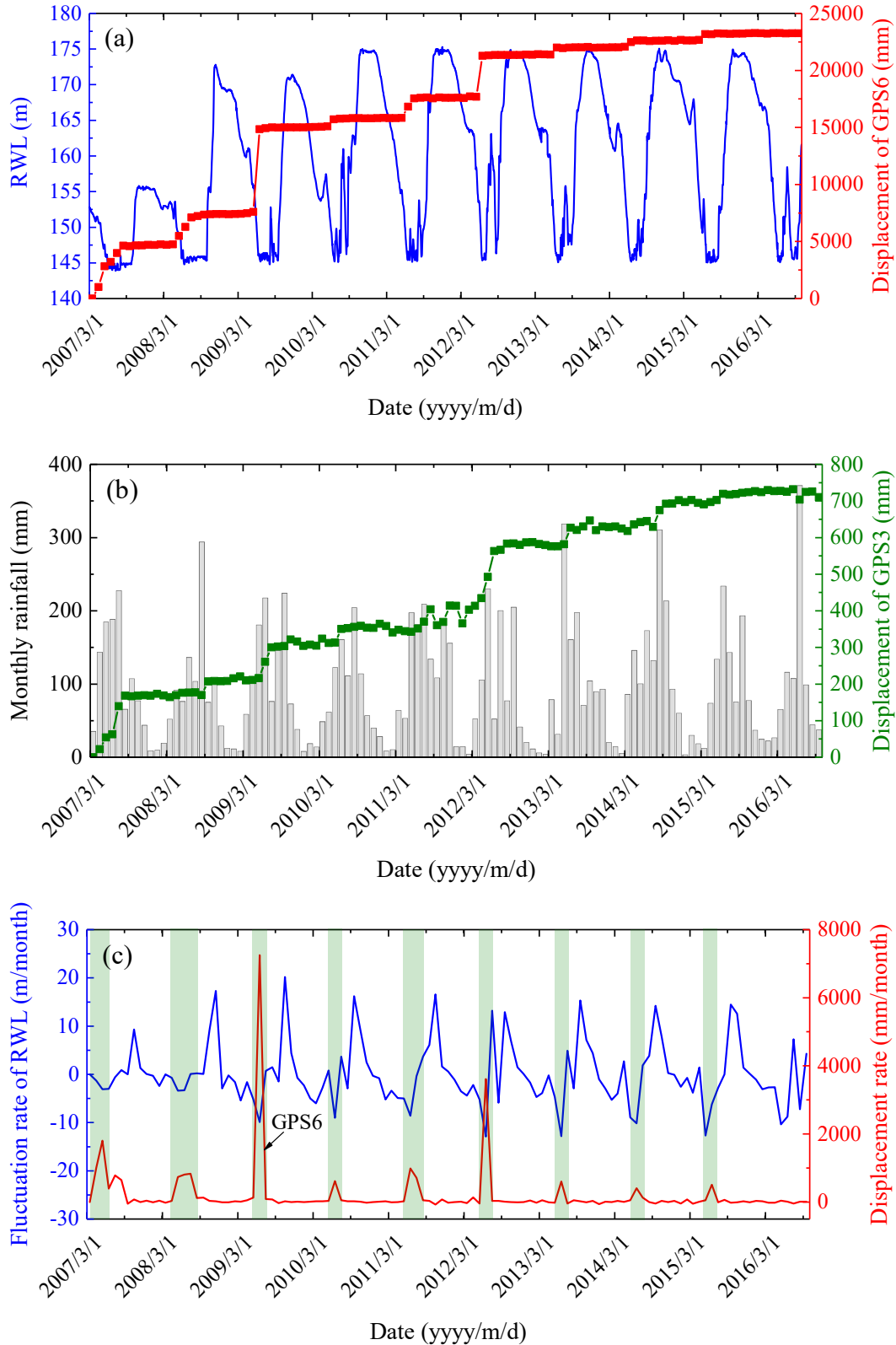


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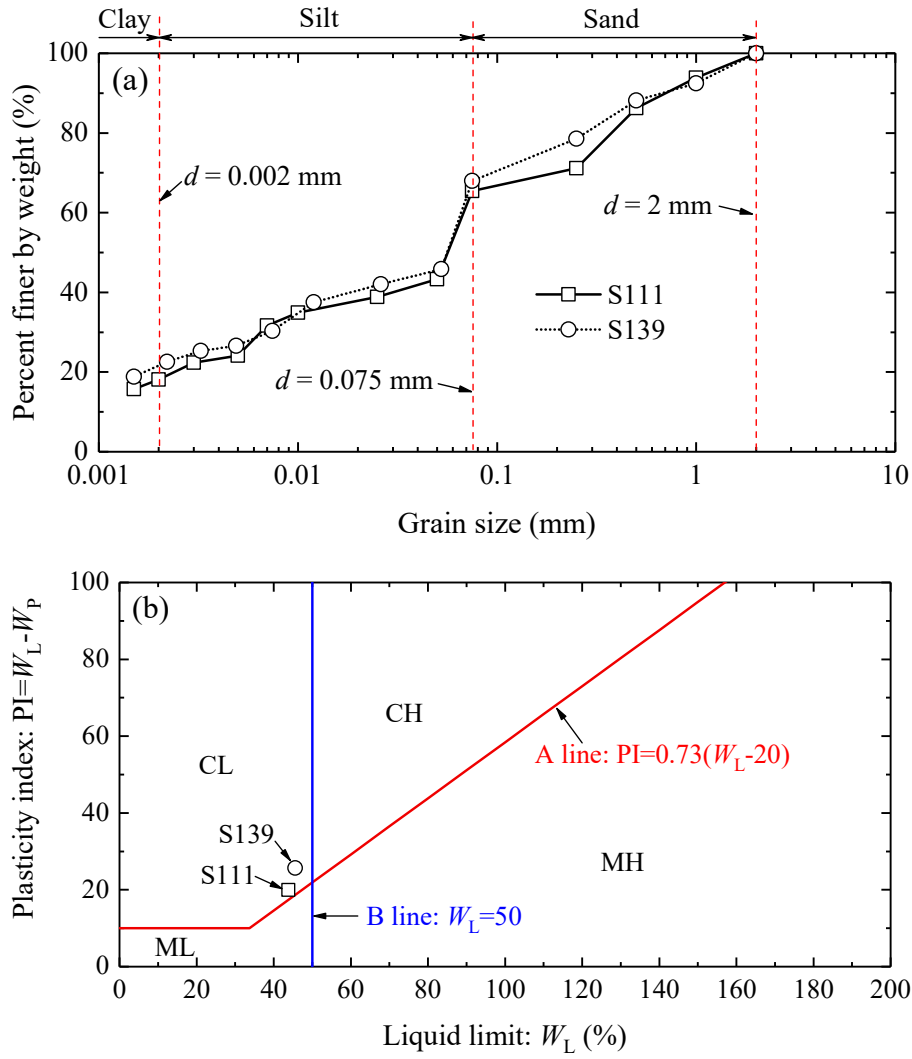


Fig. 6. (a) Grain size distributions of specimens S111 and S139 and (b) plasticity charts of specimens S111 and S139.

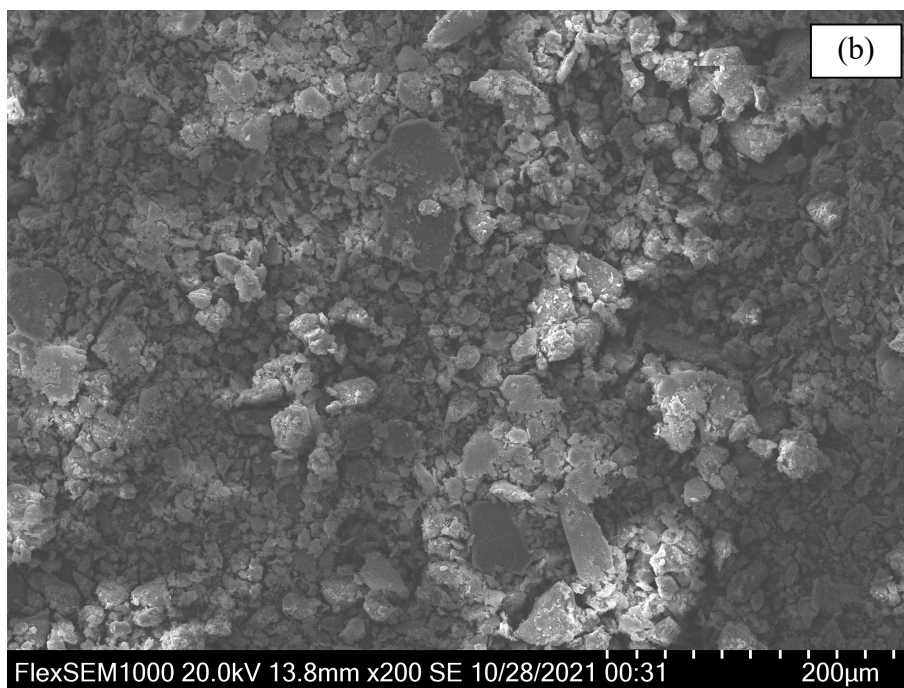
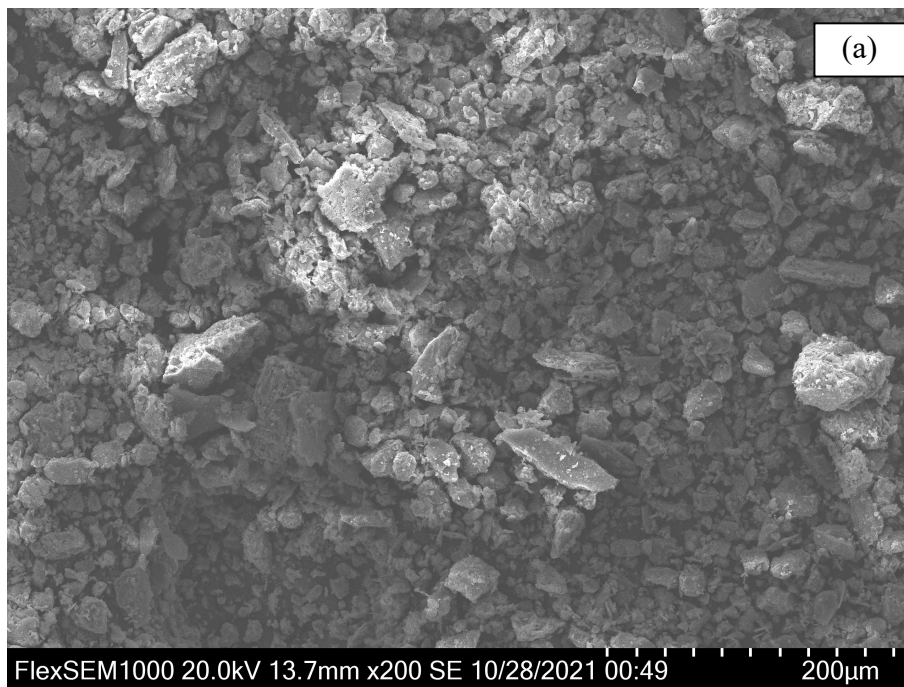


Fig. 7. SEM micro-images of the soils with particle sizes less than 0.075 mm: (a) sample S111 and (b) sample S139.

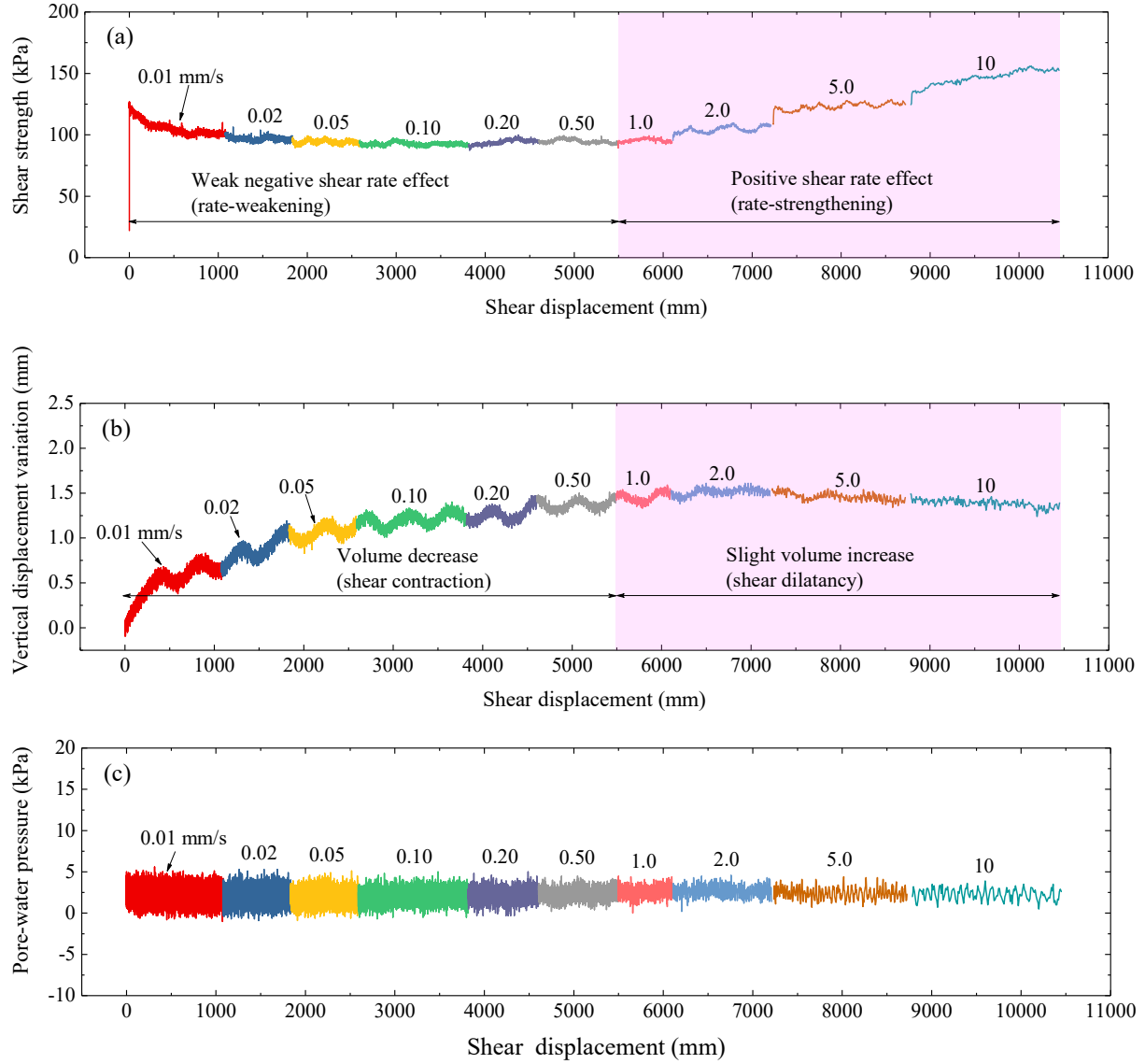


Fig. 8. Drained ring shear test on the saturated specimen S111 with increasing shear displacement rate at the same total normal stress of 310 kPa. (a) Shear strength versus shear displacement during the entire shearing; (b) vertical displacement variation versus shear displacement; and (c) pore-water pressure near the shear surface versus shear displacement.

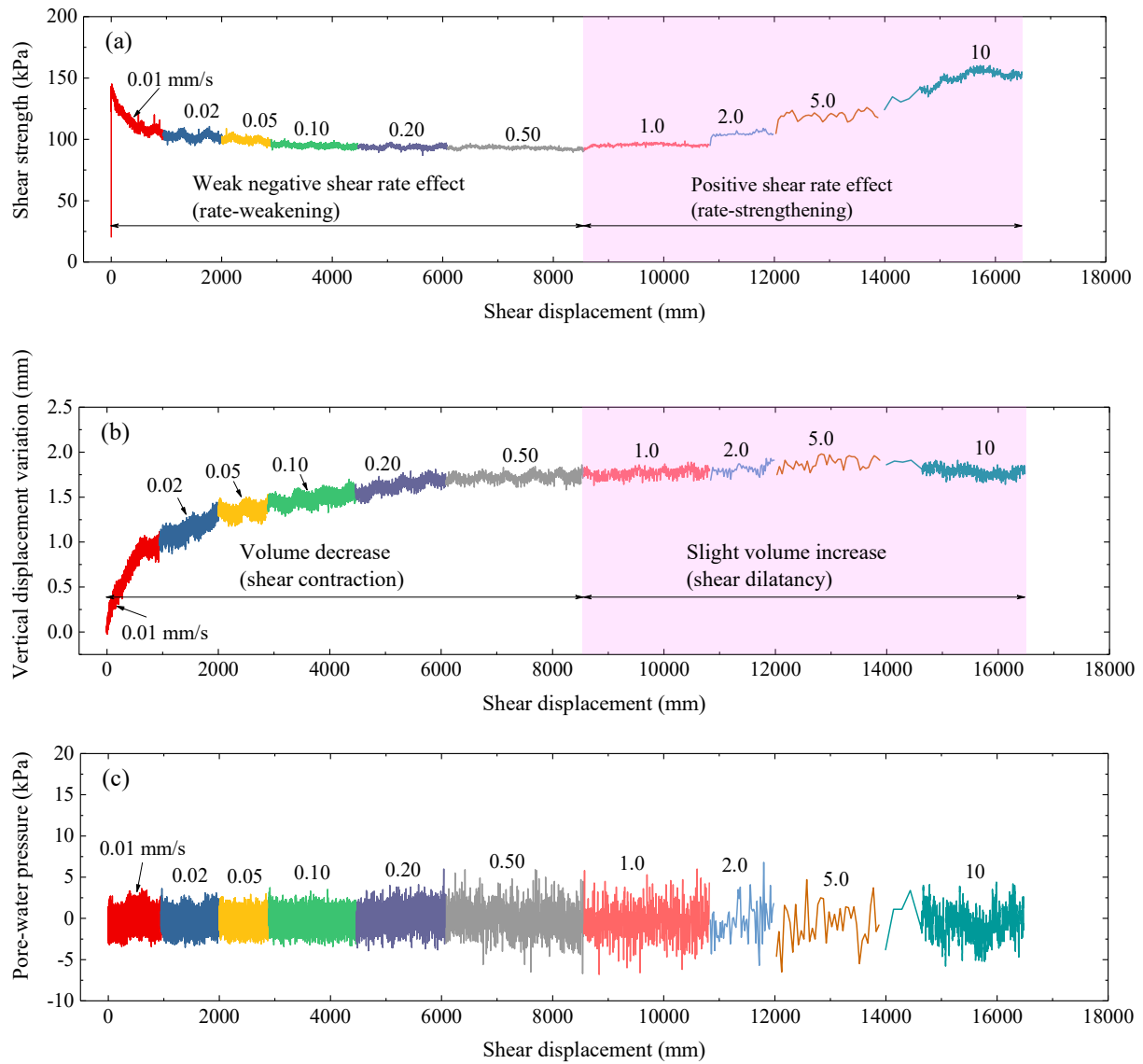


Fig. 9. Drained ring shear test on the saturated specimen S139 with increasing shear displacement rate at the same total normal stress of 310 kPa. (a) Shear strength versus shear displacement during the entire shearing; (b) vertical displacement variation versus shear displacement; and (c) pore-water pressure near the shear surface versus shear displacement.

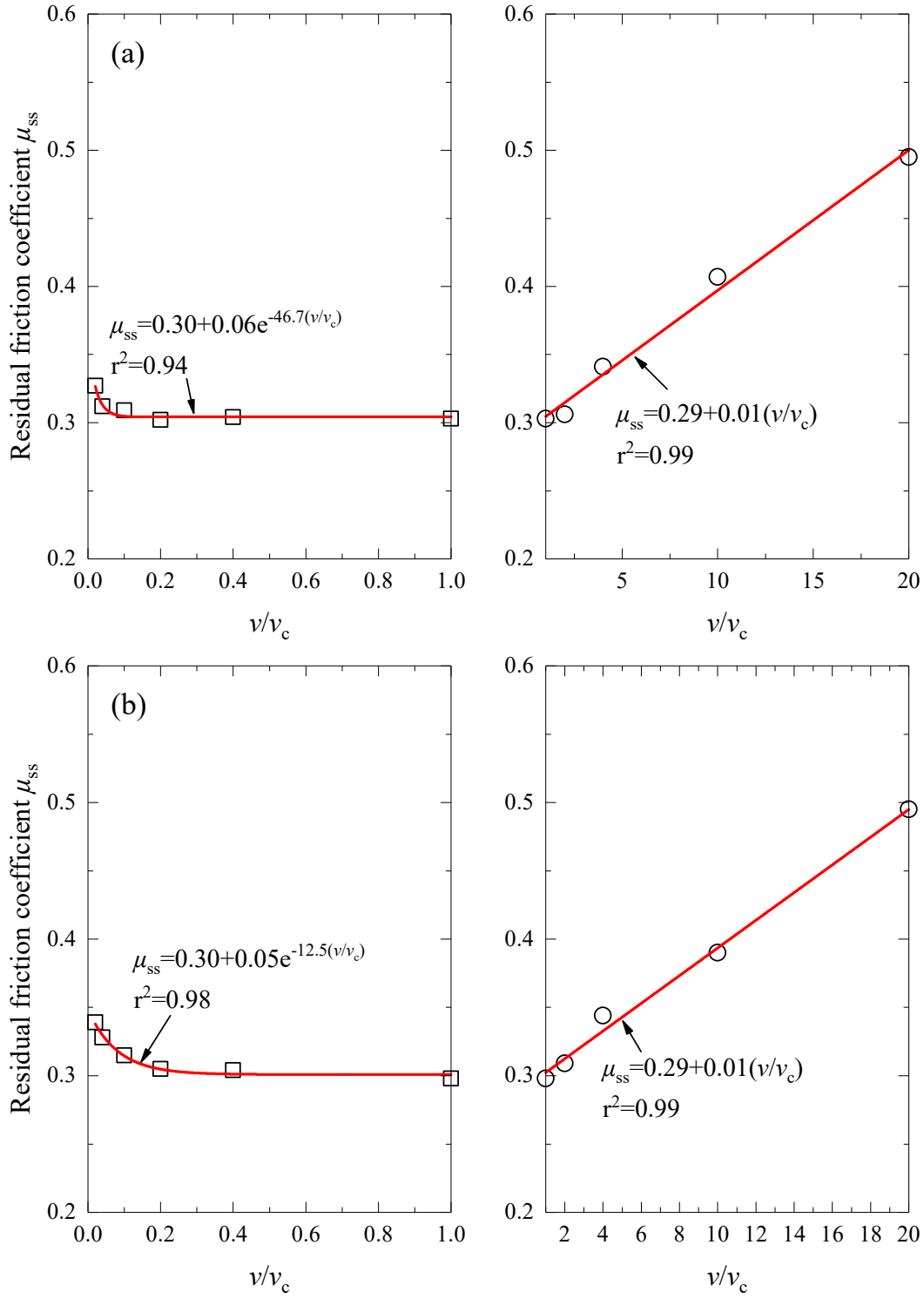


Fig. 10. Relationship between the friction coefficient in the steady state (i.e., residual state in this study) (μ_{ss}) and the shear displacement rate. (a) Specimen S111 and (b) specimen S139. v_c is defined as the critical shear rate at which a negative shear rate dependency of the residual strength transits into a positive shear rate dependency. In other words, v_c is the reference shear rate that distinguishes these two different shear rate dependencies.

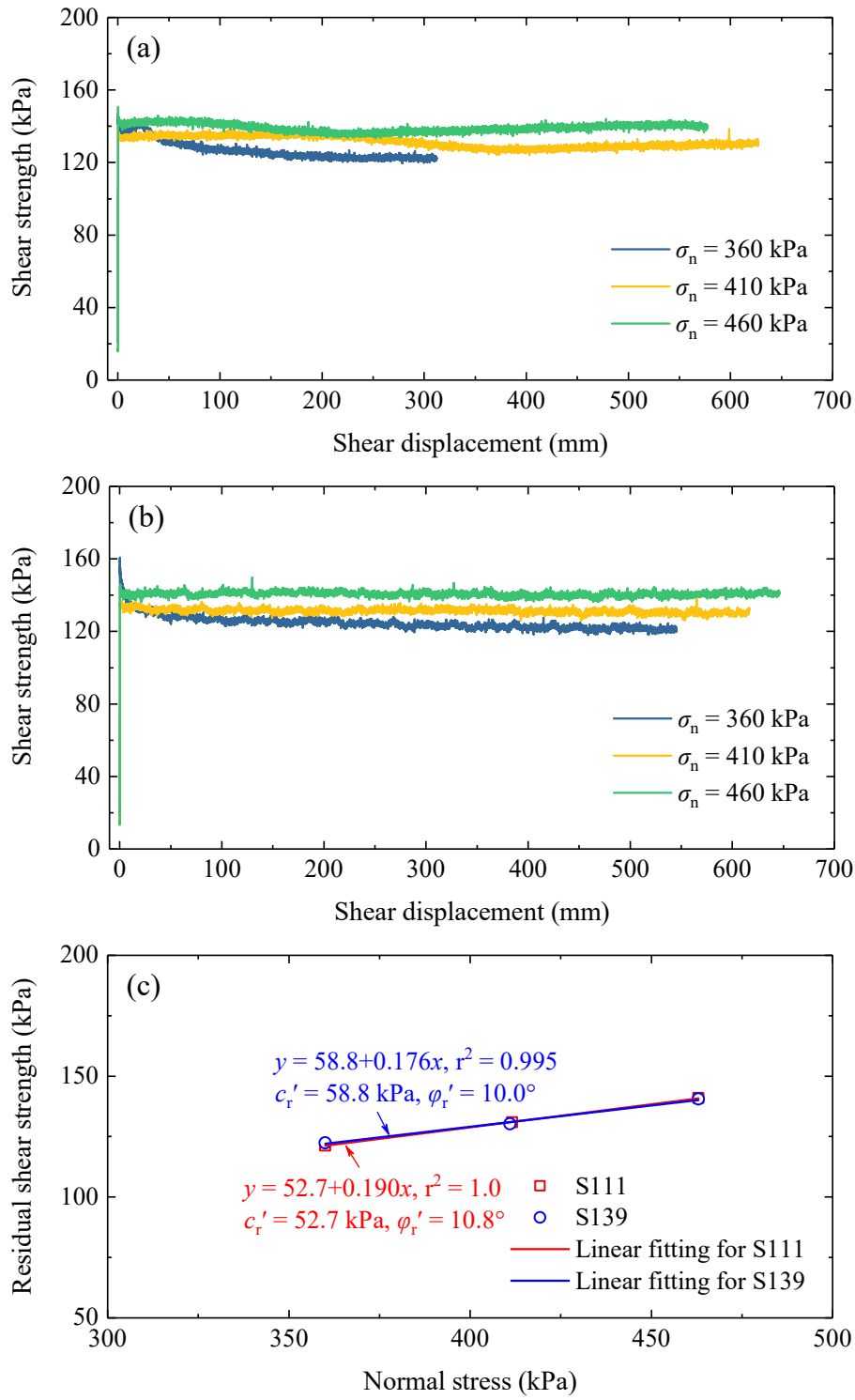


Fig. 11. Drained ring shear test on the two saturated specimens under different total normal stresses (σ_n). (a) Shear strength of specimen S111 versus shear displacement under different total normal stresses; (b) shear strength of specimen S139 versus shear displacement under different total normal stresses; and (c) residual strength envelopes of the two specimens.

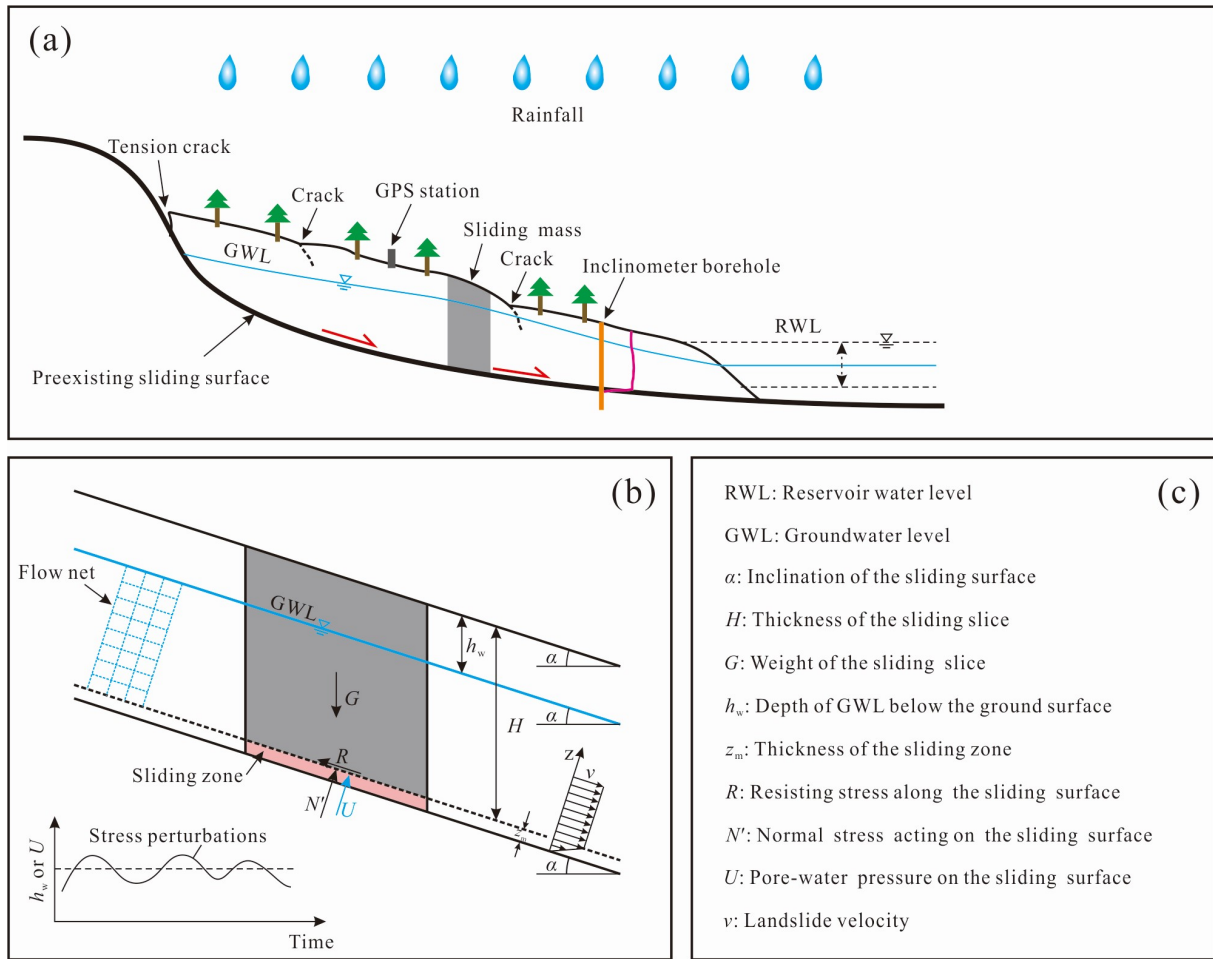


Fig. 12. Conceptual diagram for the reservoir ancient landslide in the stage of intermittent reactivation. (a) Reservoir ancient landslide with the preexisting sliding surface and the monitoring system such as GPS and inclinometer; (b) slope components under 1D infinite slope geometry and stress perturbations resulting from transient precipitation and reservoir water level fluctuation; and (c) interpretation of acting stresses on a sliding slice and relevant parameters. Note: as shown in Fig. 12c, a linear velocity profile over the depth is assumed within the range of the sliding zone.

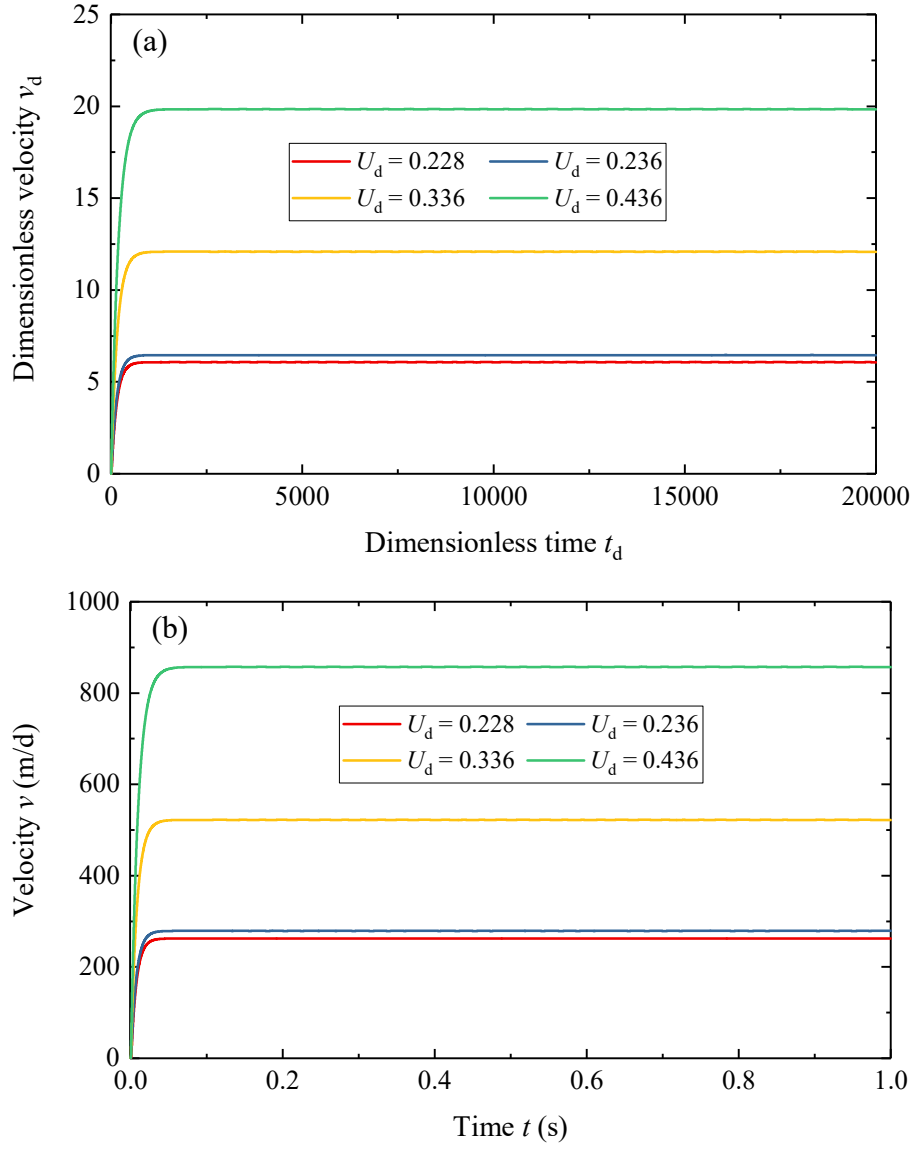


Fig. 13. Calculated dimensionless velocity v_d and velocity v of the Tangjiao landslide corresponding to the imposed dimensionless stress perturbations U_d by using Eq. (8) with parameters $\alpha = 15^\circ$ and $H = 15$ m. (a) Dimensionless velocity v_d versus dimensionless time t_d , and (b) velocity v versus time t .

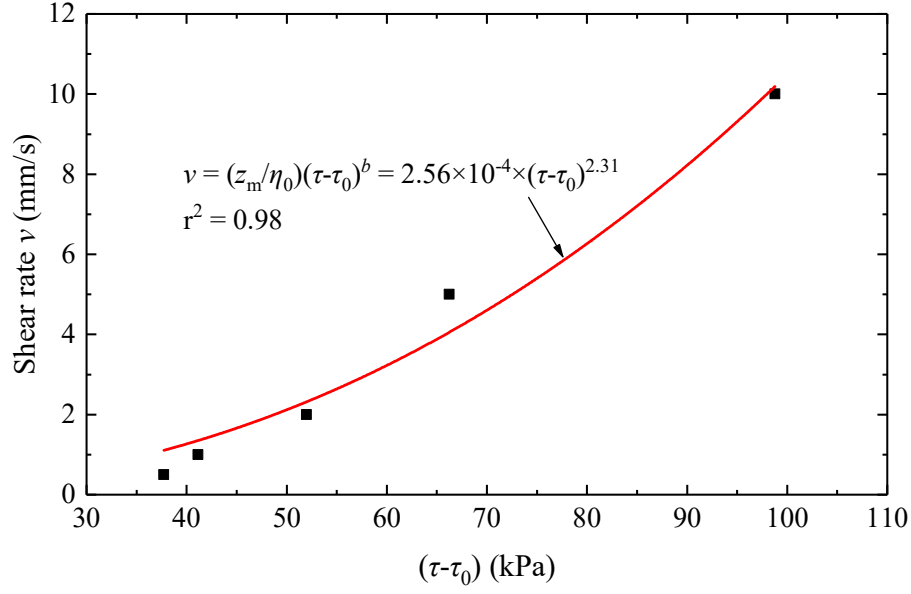


Fig. 14. The fitting curve of the shear rate v with $(\tau - \tau_0)$ follows the form of Eq. (11). Note that the unit of the thickness of the shear zone (z_m) is mm, and τ_0 is calculated by using Eq. (9b) with a residual friction angle ϕ'_r of 10.0° , as shown in Fig. 11c. The data of τ and u are obtained from Fig. 9 with a total normal stress σ of 310 kPa.

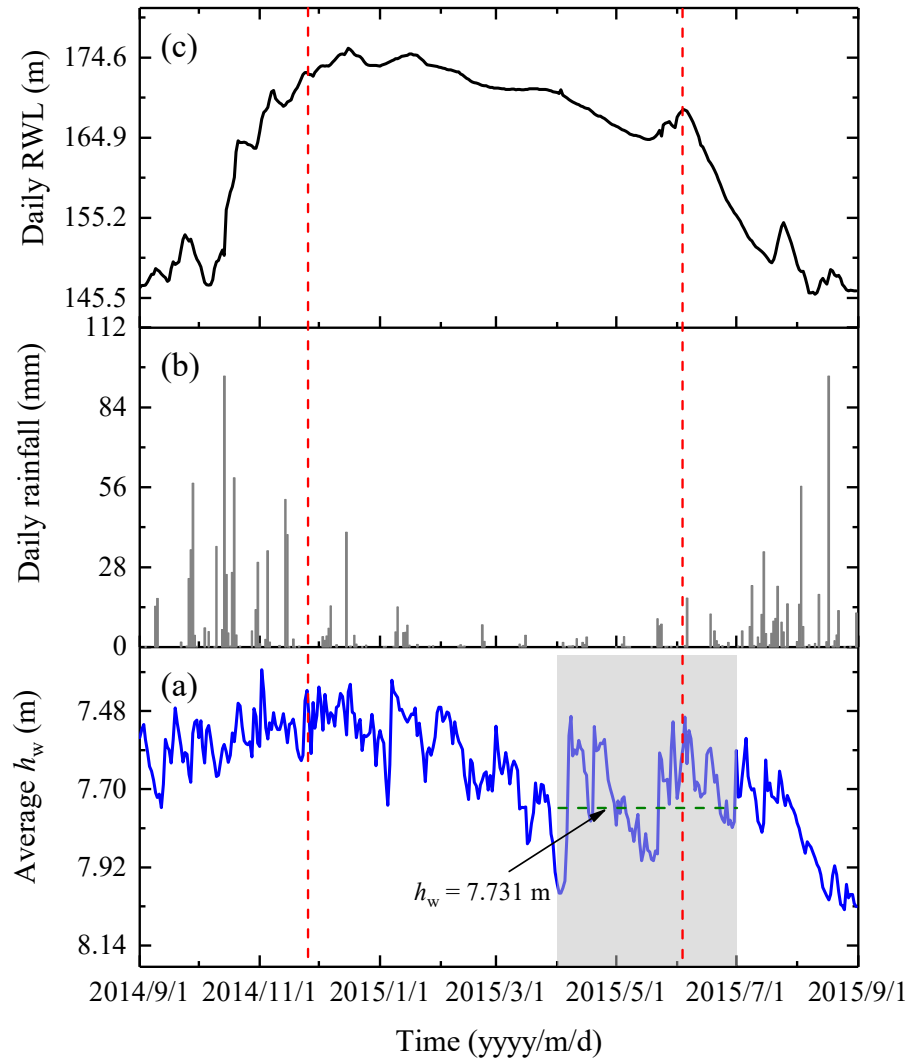


Fig. 15. (a) Daily groundwater level depth below the ground surface (h_w), (b) daily rainfall and (c) daily reservoir water level (RWL) in one hydrological year (September 1, 2014 to August 31, 2015). Note: h_w was determined as the average groundwater level depth of SK1, SK2 and SK3, whose locations are shown in Fig. 2a; the value of h_w of 7.731 m is the average value in the period from April 1 to June 30, 2015.

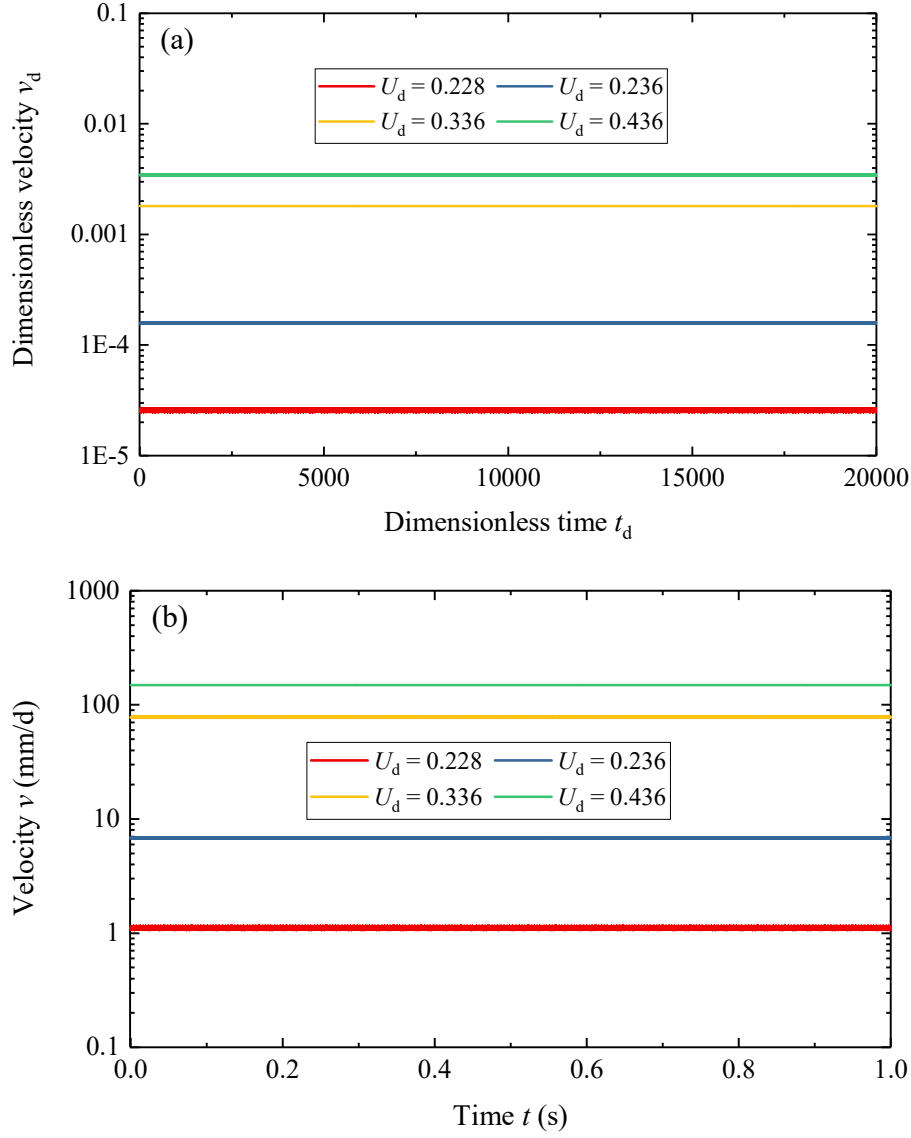


Fig. 16. Calculated dimensionless velocity v_d and velocity v of the Tangjiao landslide corresponding to the imposed dimensionless stress perturbations U_d by using Eq. (12) with the parameters listed in Table 2. (a) Dimensionless velocity v_d versus dimensionless time t_d , and (b) velocity v versus time t .

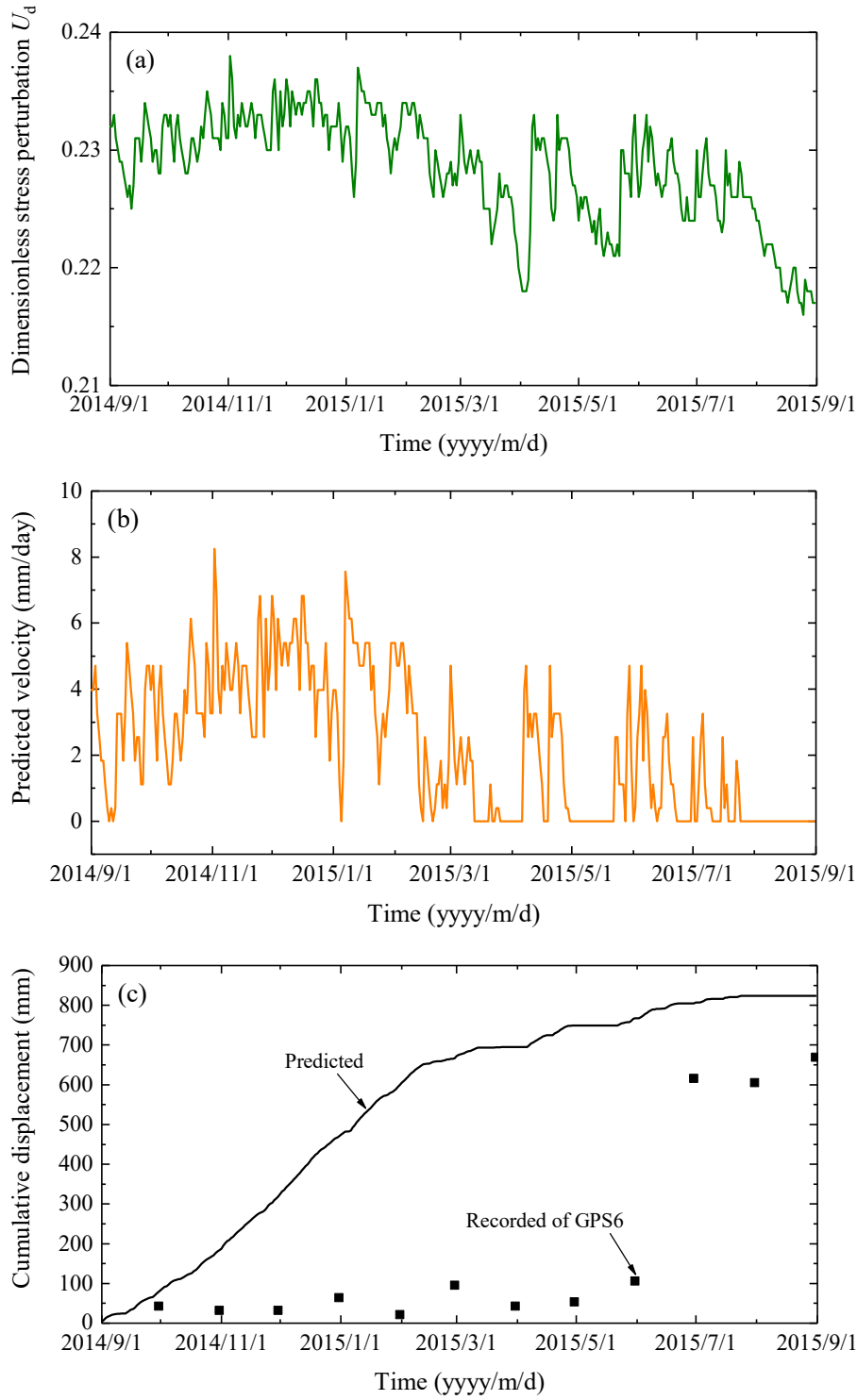


Fig. 17. Predicted velocity and cumulative displacement of the Tangjiao landslide under stress perturbations in the period from September 1, 2014, to August 31, 2015, by using the proposed SRF model. (a) Imposed dimensionless stress perturbation U_d according to the daily h_w , as shown in Fig. 15a; (b) predicted velocity of the Tangjiao landslide with the parameters listed in Table 2; (c) comparison of the predicted cumulative displacement and recorded displacement of GPS6.