

Creep properties of clastic soil in a reactivated slow-moving landslide in the Three Gorges Reservoir Region, China

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ABSTRACT

Most slow-moving landslides in the Three Gorges Reservoir (TGR) region of China are characterized by pre-existing shear surfaces. The large deformation within the shear zones usually gives rise to clastic soil formation. The creep properties have large influence on the kinematic feature of landslides. In this paper, we report an in-situ direct shear creep test carried out in the shear zone of a reactivated slow-moving landslide in the TGR region. Correspondingly, some laboratory ring shear creep tests are carried out to interpret the movement pattern of this landslide. The shear zone soil exhibits similar non-attenuating creep responses in both the in-situ direct shear and laboratory ring shear creep tests. At the same stress level, however, the in-situ direct shear creep test yields a larger rate of creep displacement due to shearing along the landslide direction. In the ring shear creep tests, at the prepeak stage, the critical creep stress that triggers creep failure is slightly lower than the peak shear strength but much larger than the residual strength; at the postfailure stage, the critical creep stress of the shear-zone soil is equal to the residual shear strength. The rate-dependent residual shear strength may account for the stepwise movement pattern of the landslide.

1. Introduction

Deep-seated and slow-moving landslides often involve large-scale mass movements with relatively small displacements (Pánek et al., 2011; Pánek and Klimeš, 2016). They are, however, often known to be the cause of severe damage to structures and infrastructure (Cascini et al., 2009; Di Maio et al., 2013; Fernández-Merodo et al., 2014; Li et al., 2019, among others:). According to reported statistics, there are some 1726 landslides, with a total volume of $1.3339 \times 10^{11} \text{ m}^3$ in the TGR region in China (He et al., 2008), and most of them can be classified as slow or very slow-moving landslides. For instance, (Miao et al., 2014) examined the movement rate of 21 landslides in the TGR area and found out that 6 landslides move at rates ranging from 6 cm per year to 1.5 m per year, while the other 15 display creeping displacement with an average sliding rate of less than 6 cm per year. These characteristics usually make slow-moving landslides difficult to detect, and as a consequence, many roads, buildings, and tourist resorts were constructed in areas prone to slow-moving landslides in the TGR region.

A typical case is the Huangtupo landslide in Badong District, a small county along the Yangtze River in China. This area is well known for its landslide-prone geological conditions and large-scale landslides that

have led to the relocation of thousands of residents (see Fig. 1). The Badong County seat has experienced relocation twice in 3 decades. The old county seat was moved to the Huangtupo area due to the rising water level following the impoundments of the Gezhou Dam in 1982. After the relocation, however, the Huangtupo area was identified as a large-scale landslide in 1992 (Deng et al., 2000). The slow movement of this landslide has destroyed construction and roads on this slope. As a consequence, the town was moved to an area approximately 10 km west of Huangtupo after 2008. Though the displacement rate of the sliding mass is not large enough to cause a sudden slide into the adjacent Yangtze river, the continuous creep displacement has posed a great threat to the safety of surrounding residents and the Yangtze river waterway. The large size of Huangtupo landslide makes it an ideal study object to better understand the mechanisms of similar slides in this area. In 2012 the Badong field test site was established by the Three Gorges Research Center for Geohazards (TGRC). The test site includes a large-scale tunnel with auxiliary adits and an extensive in-situ monitoring system.

Since the completion of the Badong field test site, research on the internal structure and kinematic feature of the Huangtupo landslide has been intensified. Through U–Th dating, Tang et al. (2015a) reported

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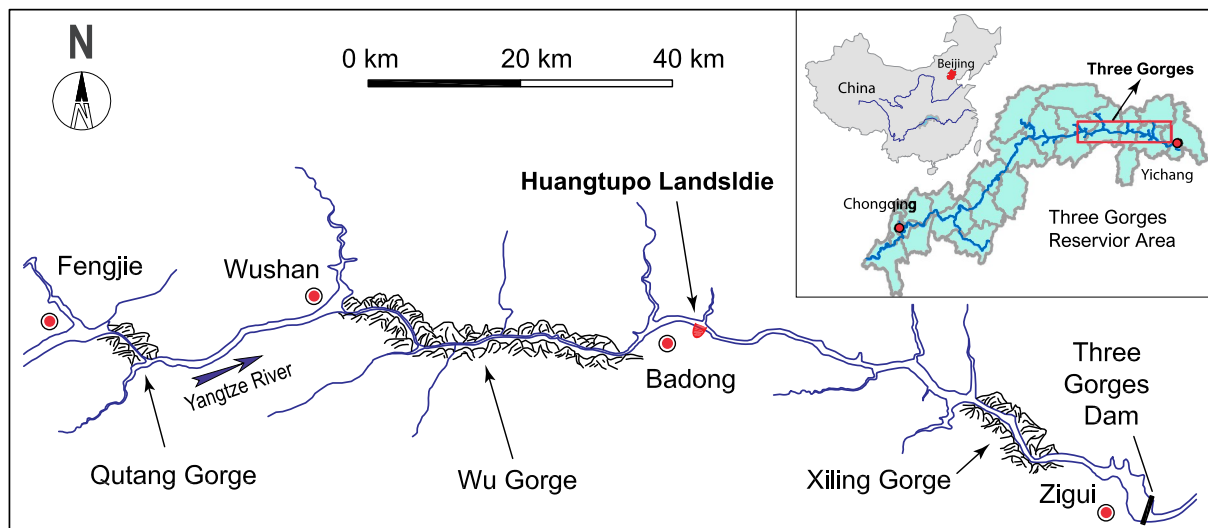


Fig. 1. The TGR region and location of the Huangtupo landslide in China, base map from Wang et al. (2018a).

that the Huangtupo riverside sliding masses might undergo at least two periods of movement. Subsequently, Wang et al. (2018a) proposed a double shear surface model for the Huangtupo No. 1 Riverside sliding mass based on the geological logs obtained from the excavation of the tunnel group and subsequent borehole logs. On the other hand, Liu et al. (2013) and Tomás et al. (2014) analyzed the deformation of the Huangtupo landslide based on InSAR data. The upper part of the landslide is affected by seasonal displacements caused by rainfall, while the lower part is affected by periodic water level fluctuation in the reservoir. Furthermore, Wang et al. (2014) proposed a concept model for analyzing the relation between the episodic landslide movement and the variation of the hydrological conditions of the landslide. Tang et al. (2015b) investigated the movement pattern of the Huangtupo landslide during groundwater variation and seasonal rainfall. Wang et al. (2016) compared the superficial and deep deformation of the Huangtupo landslide and identified that the water level variation and seasonal rainfall have different effects on the movement of the Huangtupo landslide. The aforementioned studies give rise to an improved understanding about the deformation mechanisms: the Huangtupo landslide moves stepwise with episodic accelerating and decelerating phases. However, influenced by hydrological boundary conditions, such as rainfall and the groundwater table, the creep deformation is driven predominately by the weight of the large sliding mass.

It is well known that the displacement rate of a landslide is the result of a dynamic equilibrium between destabilizing and stabilizing forces, which also depends on the viscous behavior of the involved soil (Van Asch, 1984; Di Maio et al., 2013; Tan et al., 2018). Hence, in addition to the hydrological conditions, it is believed that the creep property of the shear-zone soil plays an important role in the interpretation of the mechanism behind the movement pattern of the landslide. To date, there has been insufficient research on the creep properties of the shear-zone soil. Therefore, a series of laboratory creep tests over various stress ranges have been performed by the colleagues at TGR. Li et al. (2017) reported the creep properties of intact shear-zone soil in the triaxial stress state. Later, Wang et al. (2018b) investigated the residual state creep properties of undisturbed shear-zone soil using a ring shear apparatus. In most cases, however, there still remain difficulties in the laboratory tests for dealing with the coarse particles in the shear-zone soil. In-situ tests carried out in the shear zone of giant landslides are rare because such tests are rather time-consuming and the access to the soil at large depth is difficult. Fortunately, the excavation of the tunnels in the Badong field test site exposed several shear zones, which provide ideal conditions for in-situ tests,

e.g., the in-situ triaxial compression test (Tan et al., 2018). For further investigation of the viscous behavior of the shear-zone soil subjected to various stress paths, more creep tests both in the field and laboratory are required.

In this paper, we report an in-situ creep test performed at the Huangtupo landslide to investigate the creep properties of shear-zone soil by means of direct shear creep tests. Particularly, the direct shear creep test is performed in the shear zone of this landslide with the shearing direction parallel to the inclined shear surface. The viscous behavior of the shear-zone soil is examined over various shear stress ranges. Additionally, two ring shear creep tests, namely a prepeak ring shear creep test and a postfailure ring shear creep test, are carried out in the laboratory with the same soil. The test results are compared with those obtained by triaxial creep tests reported in the literature. Based on the test results, the factors that influence the measurement of the critical creep stress and the relation between the critical creep stress and landslide movement are discussed.

2. Description of the landslide and in-situ creep test site

This section is concerned with describing the position and creep movement of shear zone where the in-situ creep test was carried out. In addition, the environment of the in-situ test site is introduced.

2.1. Shear surfaces distribution

The Huangtupo landslide developed at the south bank of the valley of the Yangtze River with the bedrock dipping toward the valley. The crown elevation of the landslide is approximately 600 m.a.s.l., while its toe varies from 50 to 90 m.a.s.l. The toe is submerged in the Yangtze River, with water levels varying from 145 to 175 m, as regulated by the Three Gorges Dam. The composite landslide covers a total area of $1.35 \times 10^6 \text{ m}^2$, with a mobilized volume of nearly $6.93 \times 10^7 \text{ m}^3$, according to the investigation in 2001; therefore, it is one of the largest reservoir landslides in the TGR region. More details concerning the geological structure, lithology, and hydrological conditions of this landslide can be found in the literature (Deng et al., 2000; Wang et al., 2018b; Wang et al., 2016; Tang et al., 2015a; Tang et al., 2015a, 2015b; Tang et al., 2019; Jian and Yang, 2013).

According to the previous investigation in 2001, the Huangtupo landslide is a complex mass formed by multiple sliding masses that occurred over a period of at least 100,000 years (Tang et al., 2015a). The plan view of the Huangtupo landslide is shown in Fig. 2. On the surface, the landslide is divided by the Sandaogou Valley into two

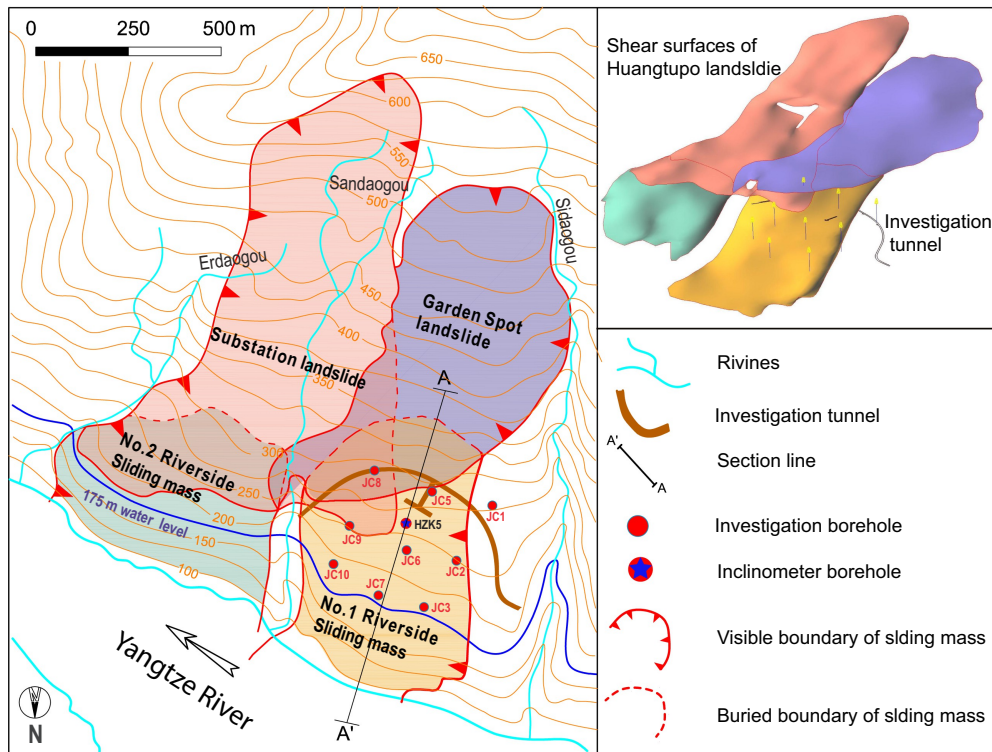


Fig. 2. The plan view of the Huangtupo landslide with the 3D model of the shear surfaces based on the data from the 2001 investigation.

groups of sliding masses. Each group is composed of two sliding masses, one on top of the other, with additional recent sliding masses developing adjacent to the edges of the landslide. The foreside sliding masses, the No. 1 and No. 2 Riverside sliding masses, are adjacent to the Yangtze River with their toes being submerged in the river.

The spatial distribution among the shear surfaces shown in Fig. 2 indicates that the four sliding masses are sliding along separate shear surfaces. The geological profile of the No. 1 Riverside sliding mass shown in Fig. 3 indicates that it is a chaotic mass consisting of either loose or dense soil and rock debris (Wang et al., 2014). The inclinometer profile of borehole HZK5 shows that significant horizontal displacement has occurred at depth of 54 m and 76 m. This implies a double shear-surface structure of this landslide.

To investigate the shear surface distribution in this landslide, a

investigation tunnel was constructed crossing the No. 1 Riverside sliding mass. As shown in Fig. 4(a), the investigation tunnel in the Badong field test site includes a main tunnel and five branch tunnels. The arc-shaped main tunnel is 908 m long. Five branch tunnels named BT1–BT5 are connected to the main tunnel. The excavation of the tunnel group exposed weak interlayers and shear zones located at both the main tunnel and the branch tunnels BT3 and BT5. More details can be found in the literature (Wang et al., 2018a; Tang et al., 2015a).

According to the U-th dating results by Tang et al. (2015a), the ages of the shear-zone soil in BT3 and BT5 are 40 ka and 100 ka, respectively. Thus, the shear zones in BT3 and BT5 are not the same layer; therefore, the Huangtupo No. 1 Riverside sliding mass can be further divided into secondary sliding masses (No. 1–1 sliding mass and No. 1–2 sliding mass). Based on this knowledge, Wang et al. (2018a)

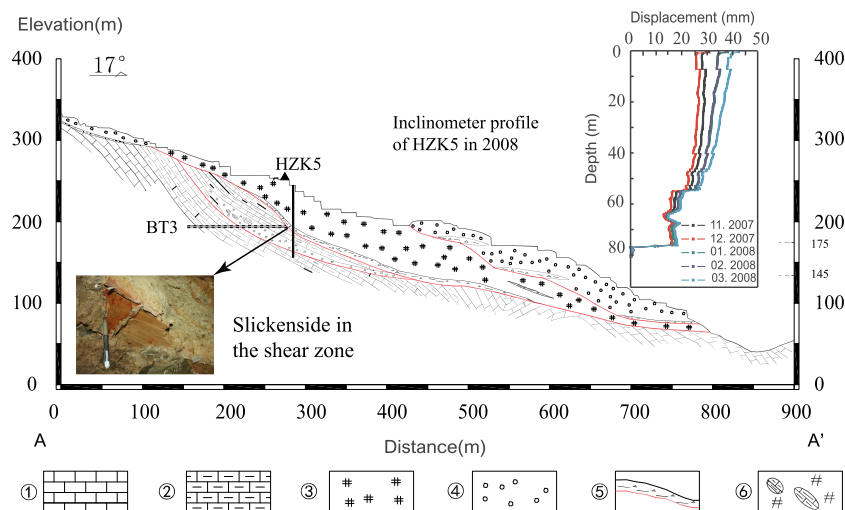


Fig. 3. Section A-A' in No. 1 Riverside sliding mass: ① limestone; ② pelitic limestone; ③ dense soil and rock debris; ④ loose soil and rock debris; ⑤ sliding zones with possible slip surfaces; ⑥ Mudstone fraction zone, and the inclinometer profile of borehole HZK5, inclinometer data from Jian and Yang (2013).

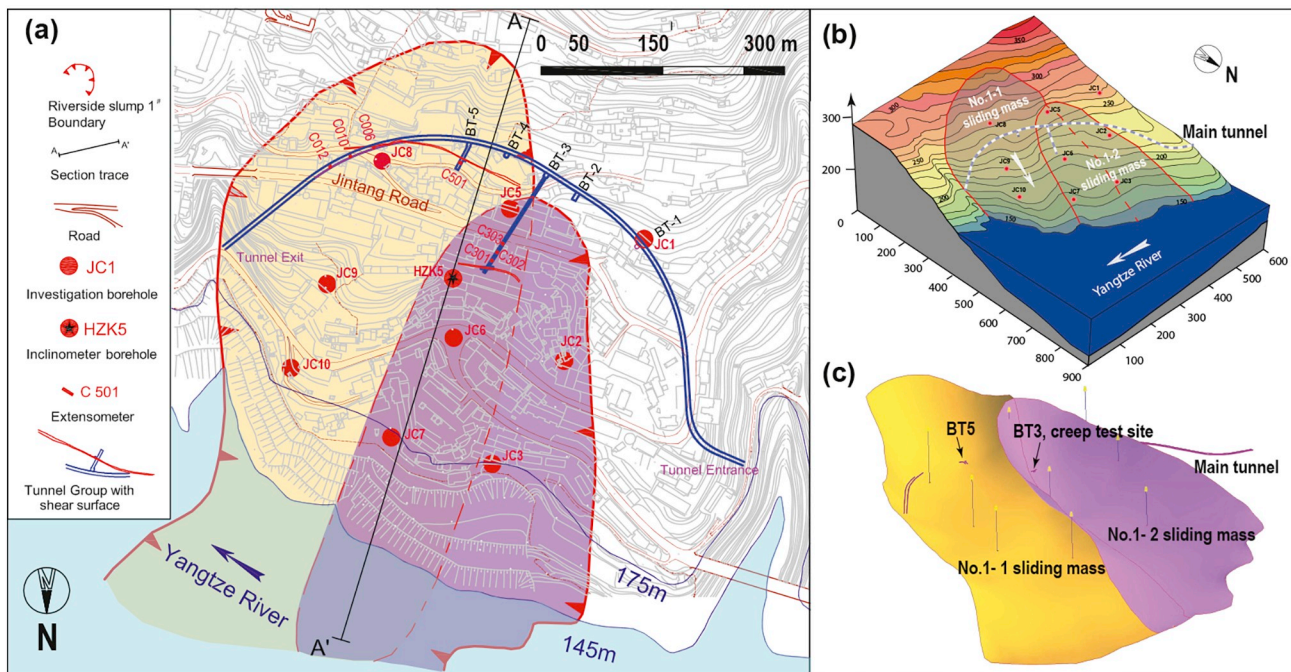


Fig. 4. The Huangtupo No. 1 Riverside sliding mass: (a) plan view, (b) 3D topographic model. Modified after Tang et al. (2015a), and (c) 3D view of the double shear-surface model, after Wang et al. (2018a).

proposed a double shear surface model of the Huangtupo No. 1 Riverside sliding mass using the geological data obtained from the excavation of the tunnels and the recent investigation boreholes. The 3D topographic model, and 3D view of this double shear surface model are shown in Fig. 4(b,c). From this model, it is easy to show the location where the in-situ creep test was carried out.

2.2. Movement of the shear zone

In-situ GPS monitoring indicates that the No. 1 Riverside sliding mass exhibits steady creeping, with a movement rate range from very slow (17.2 mm/year) to extremely slow (12.8 mm/year) in the direction of 20° northeast (Tomás et al., 2014; Tang et al., 2015a). Since the completion of the tunnel group in December 2012, this movement has resulted in many cracks in the lining and on the pavement of the tunnel. Several extensometers have been, therefore, installed in the tunnel to measure the propagation of these cracks since 2013. The distribution of the extensometers is shown in Figs. 6. The extensometers were installed in the tunnel in locations where the shear zones were exposed and vertical to the cracks to obtain the width variation. Hence, the development of cracks provides direct insight into the movement pattern of the Huangtupo landslide.

Fig. 5 shows the development of partial cracks in the tunnel group from August 2013 to December 2016, together with variation in the groundwater table and seasonal rainfall. The monitoring results of eight cracks, including 3 cracks in the main tunnel (C006, C010, and C012), 4 cracks in BT3 (C301, C302, C303, and C306), and 1 crack in BT5 (C501), are presented. During the monitoring period, the crack C501 obtained a deformation of 15 mm in the last four years, yielding the largest rate of displacement among these cracks. The other cracks attained relatively smaller displacement. The cracks C301, C302, C303 and C306 grew 1.65, 3.16, 5.84 and 3.60 mm, respectively, while C006, C010, and C012 grew 4.37, 2.47, and 5.80 mm, respectively. The development of the cracks indicates that the movement rate of the shear zone in the No. 1–2 sliding mass is larger than that in the No. 1–1 sliding mass.

On the other hand, the groundwater table and seasonal rainfall can influence the development of the cracks. The propagation of the cracks

coincides with the periods of intensive rainfall and significant declines in the TGR water level. The development of the cracks shows intermittent phases of acceleration and deceleration in the apparent dip of the shear surface and exhibits a stepwise creep tendency (Wang et al., 2018b). This result is in excellent agreement with the superficial monitoring data measured from GPS and InSAR (Liu et al., 2013; Tomás et al., 2014; Tang et al., 2015b; Wang et al., 2016), that is, the landslide has experienced alternating accelerating and decelerating movements (Wang et al., 2018b). The main conclusion drawn from this analysis is that the creeping movement of the Huangtupo landslide is predominantly governed by its self-weight. Affected by some external factors, the soil within the shear zone is undergoing a stepwise creep displacement. This movement pattern leads to the necessity of performing in-situ creep tests to investigate the viscous behavior of the shear zone material.

2.3. In-situ creep test site

After the construction of the investigation tunnel, two testing tunnels were excavated and advanced in the BT3 and BT5 at the 138-m and 20.1-m sections, respectively, for further in-situ tests and intact soil sampling. The creep tests were supposed to be carried out in the testing tunnel either in BT3 or BT5; see Fig. 6(a).

The shear zone in BT3 is usually 0.5- to 1.2-m thick, which is much thicker than that in BT5. Moreover, the hydrological condition in BT3 is more suitable for the in-situ test, while groundwater has passed through the shear zone in BT5, and water bursting has caused severe collapse on the tunnel face. Therefore, the testing tunnel in BT3 was selected as the in-situ creep test site, as shown in Fig. 6(b). Several samples were prepared in the east direction of the testing tunnel in BT3. To this end, the left side of the testing tunnel was excavated, leaving a platform with shear-zone soil above the shear surface. The shear-zone soil was then prepared to block samples for the in-situ creep tests. To mimic the landslide movement as realistically as possible, the in-situ creep shearing was supposed to be carried out parallel to the shear surface, as shown in Fig. 6(c).

It is worth noting that, based on this testing platform, the TGRC has proposed a comprehensive test plan including both in-situ and

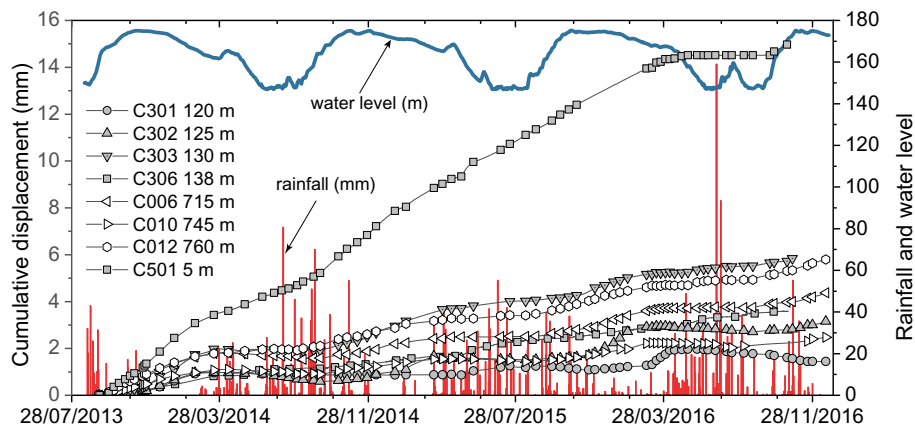


Fig. 5. Crack development in the tunnel group together with variation in TGR water level and rainfall.

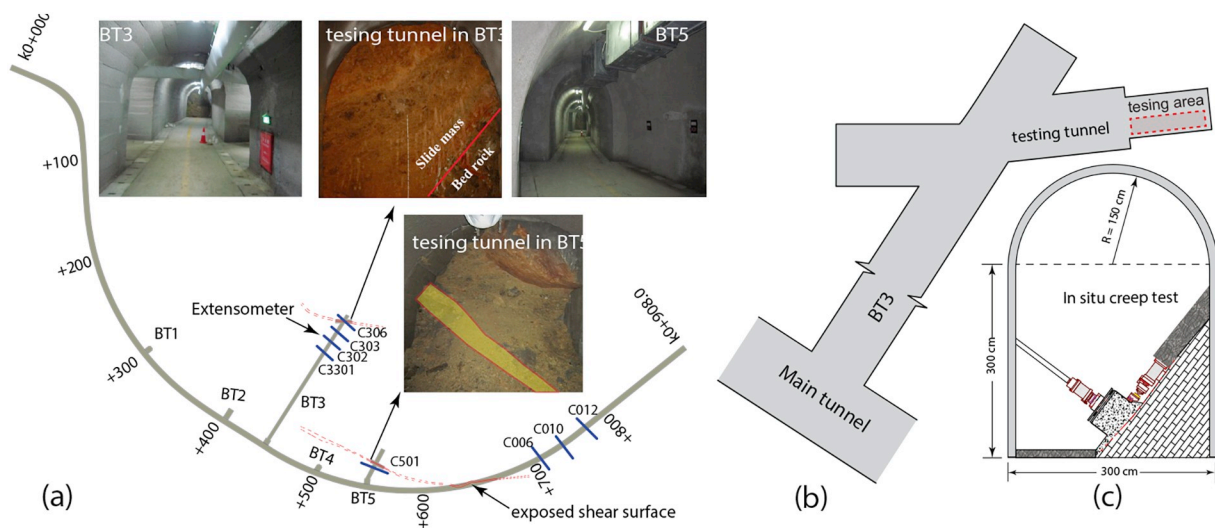


Fig. 6. (a) The tunnels in the Badong field test site, (b) creep test area in the testing tunnel in BT3, and (c) the direct shear device fixed on the shear surface.

laboratory creep tests using various testing approaches, e.g., in-situ and laboratory triaxial compression creep tests and in-situ and laboratory shear (direct shear and ring shear) creep tests. The in-situ test site for sampling and in-situ tests are shown in Fig. 7. The laboratory and in-situ triaxial compression creep tests were reported by Li et al. (2017) and Tan et al. (2018), respectively. In the following, therefore, we

report the experimental works of the in-situ and laboratory shear creep tests.

3. Experimental works

3.1. Test material

Some intact samples were taken from the shear zone exposed in the testing tunnel. Laboratory tests were then carried out to determine their basic physical properties. As presented in Fig. 8(a), the shear-zone soil typically comprises fine-grained soil (silt + clay) with a significant fraction of coarse-grained particles, constituting approximately 20 to 50 wt% of the material. Fig. 8(b) shows an X-ray CT (computed tomography) section of an intact sample, where the outline of the gravel clasts are clearly distinguished from the fine grained portion. Visual observations in the testing tunnel (see Fig. 7) and from the CT image reveal that the gravel is typically subrounded to angular and irregularly shape, with diameters varying from 0.5 to 2.5 cm. Previous experimental studies evidenced that the interaction of particles in coarse-grained soil, with 25 and 50 wt% coarse particles, remains complex (Jiang et al., 2016; Xu et al., 2007; Xu et al., 2008), and thus, the viscous behavior can be significantly influenced. For example, the creep threshold stress and the creep displacement leading to accelerated failure of shear-zone soil in the residual state increases with coarse particle content (Wen and Jiang, 2017).

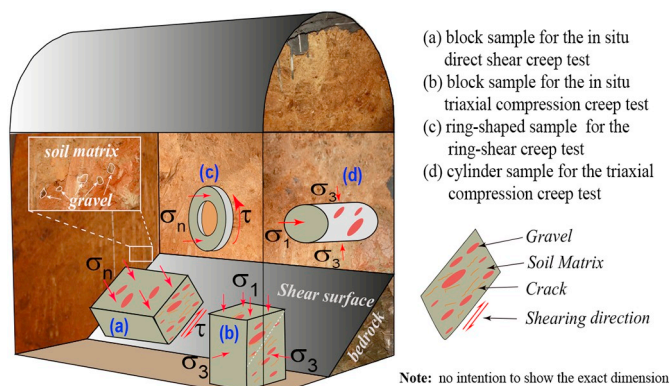


Fig. 7. The in-situ test site in the testing tunnel: the relative location of (a) in-situ direct shear creep test, and (b) in-situ triaxial creep test; the sampling position of the (c) ring shear sample, and (d) laboratory triaxial creep sample.

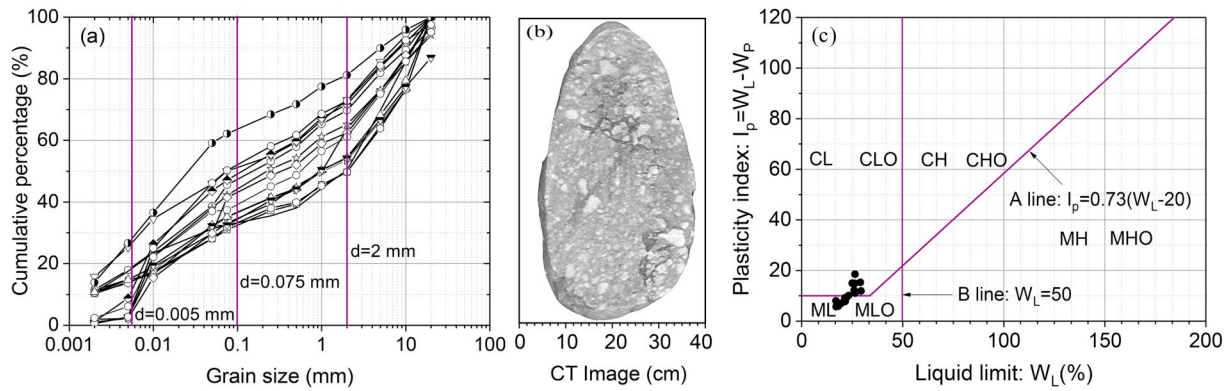


Fig. 8. Indices of physical properties of soil samples from Huangtupo landslide: (a) grain size distribution, (b) CT image of the shear-zone soil, and (c) plasticity of the finer soil.

On the other hand, the shear-zone soil contains between 50 and 80 wt% of fine material. It can be classified as silt-clay with low to medium plasticity and silt-clay mixtures of low plasticity according to the Casagrande chart shown in Fig. 8(c). It has been reported that there exists a relationship between the potential of creep in clays and the plasticity index; the coefficient of secondary compression C_α can be described by I_p using a linear function (Mesri et al., 1973; Nakase et al., 1989; Jin et al., 2019). This implies that the shear-zone soil may exhibit high creep potential under certain loading conditions.

4. In-situ creep apparatus and instrumentation

A reliable shearing device is required for the long-term in-situ creep test in the field site. For this purpose, a direct shear apparatus was designed by the TGRC. It was made of steel for reliability and comprises a reaction frame, two servo-controlled, oil-actuated hydraulic jacks, a circulation cooling device and the shear box. The shear box consists of two parts, i.e., an upper and a lower box. The upper box has the dimension $0.5 \times 0.5 \times 0.25$ m, while the lower box was $0.5 \times 0.5 \times 0.10$ m. The hydraulic jacks can produce a maximal shear force of 300 kN and a normal force of 500 kN. Both hydraulic jacks have a 100-mm stroke. This implies that the maximal shear and normal displacement is 100 mm. The physical dimensions and the arrangement of the shear box, transducer valves, reaction frame and the loading cylinders are shown in Fig. 9.

The in-situ creep test involved the assembly of the shear box on the shear surface and shearing at controlled forces. To mimic the shear movement of the landslide as realistically as possible, a block of sample, with near-vertical sides to the shear surface, was prepared according to dimensions of $0.5 \text{ m} \times 0.5 \text{ m} \times 0.35 \text{ m}$ on the inclined shear surface. Then, the exposed interface was flatly grouted before the bedframe was fixed. The bedframe had a height of 0.10 m and was used as the lower shear box, which accommodated the lower part of the soil sample. The upper shear box was centered over the upper part of the block sample and stabilized by two adjustable support legs connected to the bedframe. The gap between the box and the block was filled with wet coarse sand. Afterward, the loading cylinders, transducers, and data acquisition system were instrumented.

The application of normal and shear forces were of crucial importance for the creep test. To apply the normal and shear forces, two reaction walls in the shear direction and the normal direction were built on the top of the inclined shear surface and the left side of the tunnel lining, respectively (see Fig. 6). The shear force was applied directly to the upper box, at an offset to the plane of shear, to facilitate the alignment and operation of the device at the test site. The normal load was applied through a rigid top plate, supported by the tunnel lining, to impose a normal stress on the block sample. The plate is independent of the upper box and, therefore, free to move in the shearing direction. During testing, the shear and normal forces were maintained through feedback in the servo-control system; the shear displacement of the

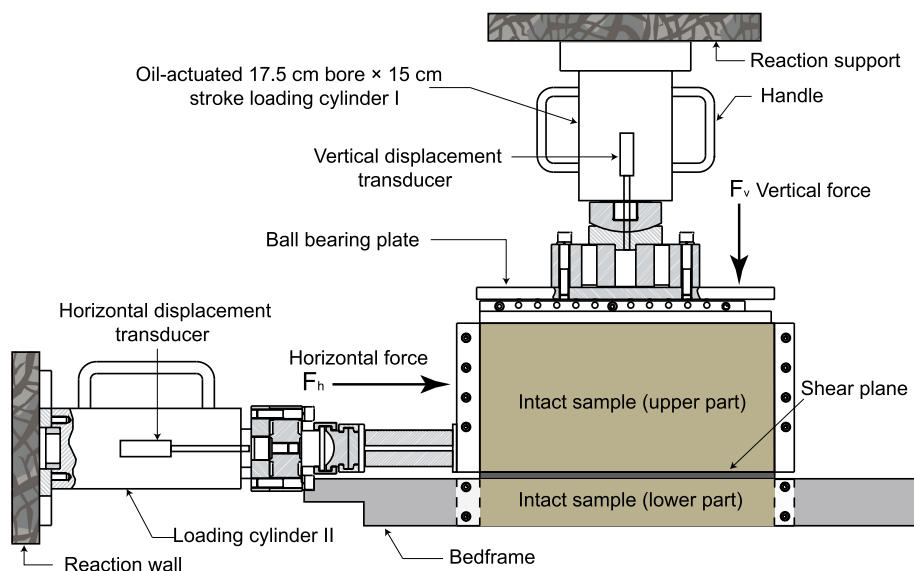


Fig. 9. Configuration of large direct shear creep box.

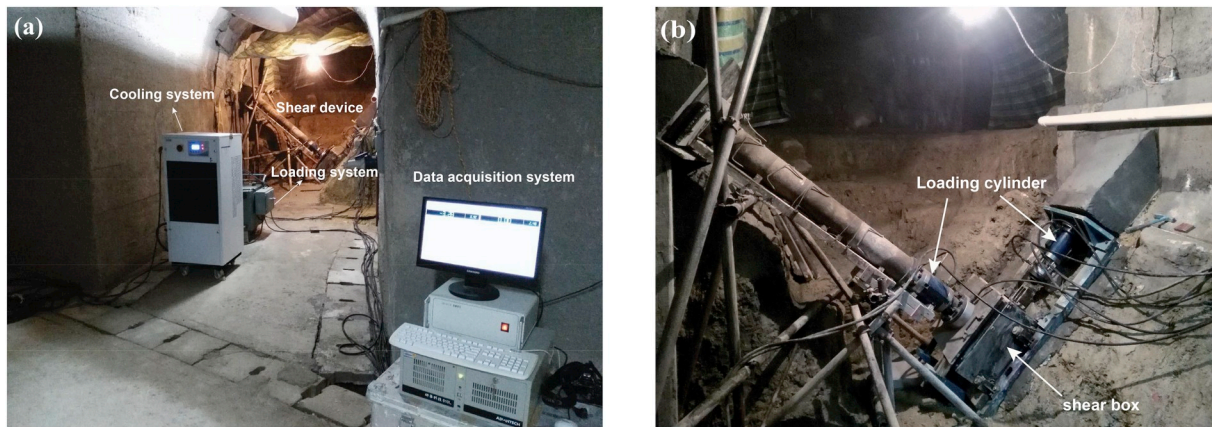


Fig. 10. (a) Field set up of the test and (b) the inclined fixed shear box.

upper shear box and vertical displacement of the loading plate were monitored by high-precision transducers. The test was terminated at a horizontal displacement of approximately 100 mm or when the failure occurred in the block sample. The field set up of the shear box on the shear surface is shown in Fig. 10.

4.1. Test program and procedure

The in-situ direct shear creep test is the so-called multistage creep test that is commonly used in creep tests (Ladanyi and Johnston, 1973). In the test process, the sample was consolidated until the rate of normal displacement approaches the null under constant effective normal stress of $\sigma_n' = 255$ kPa, which is one-third of the normal component of the overburden stress on the shear plane. The shear force was then brought rapidly up to an initial creep level. It was then increased to the limit in several equal stress increments under constant effective normal stress, each kept constant for several days. Five stages of shear stress were performed with different durations. Specifically, the tests of the first two states were maintained for approximately 34 days, which was referred to as the long-term creep test. The remainder of the creep tests took place for only two days for each stage, and they were referred to as the short-term creep tests.

A pair of ring shear creep tests, namely, a prepeak ring shear creep test and a postfailure ring shear creep test, were conducted in the laboratory. These tests were designed to demonstrate the creep properties of shear-zone soil under both prepeak and postfailure conditions (refer to Wang et al. (2018b)). The results can be used as a supplement to the in-situ direct creep shear test to interpret the movement pattern of the Huangtupo landslide. The prepeak ring shear creep test used an intact sample (sample RS2), while the postfailure ring shear creep involved a sample presheared to the residual state after a large displacement of shear (sample RS1). To this end, two intact ring-shaped samples, with a diameter of 200 mm and height of 50 mm, were obtained from the in-situ creep test site; see the sampling position in Fig. 7. By using a ring-shaped mold, the samples were made to a size of 100 mm inner diameter, 150 mm outer diameter, and 30 mm height. Compared with the height of the sample, some gravel in the soil is too large, so we used a steel needle to dig some large gravel out, leaving relatively fine-grained material in the “intact” samples. During the test, the sample was sheared in a container with wet cotton covered on the top of the container to mimic the high relative humidity environment in the testing tunnel. The configuration of the used ring shear apparatus is detailed in the references (Wang et al., 2012; Wang et al., 2018b).

The aforementioned test program is presented in Table 1, and all tests were carried out in a constant temperature environment to eliminate the influence of temperature on the test results. All measurements were done electronically, and test data were automatically recorded in the acquisition system.

5. Test results

5.1. Shear strength behavior

Prior to the in-situ and laboratory creep tests, two displacement-controlled tests, namely, an in-situ direct shear box test and a standard ring shear test, were carried out, thereby obtaining both the basic shear behavior and the general strength range of creep. The direct shear test was conducted on the intact sample in the testing tunnel. The ring shear test was performed using the intact soil taken from the testing tunnel (sample RS1), as shown in Fig. 7. For both the in-situ and the laboratory tests, samples were consolidated for 24 h under effective normal stress of $\sigma_n' = 255$ kPa. The samples were then sheared under a shear-rate controlled model with a shearing rate of 0.05 mm/min until failure was attained.

The shear stress variation with respect to the shear displacement is presented in Fig. 11(a). The in-situ direct test sample reaches peak shear strength ($\tau_{p1} = 127.5$ kPa) at a shear displacement of approximately 7 mm and then exhibits a pronounced postpeak drop in shear strength. However, the ring shear sample withstands less shear displacements before attaining its peak shear strengths ($\tau_{p2} = 168.3$ kPa), with a noticeable drop from the peak to the residual strength ($\tau_r = 143.5$ kPa). Note that the peak shear strength yields a poor agreement between the in-situ direct shear test and the laboratory ring shear test. One possible reason is the involved scale effect, that is, the shear strength may show a reduction the increase of the sample size (Zhang et al., 2015). More details regarding to the size effect will be given in the Sec. 5. Obviously, the anisotropic properties of the shear-zone soil may be another explanation for these results. As shown in Fig. 7, the shearing direction is parallel and orthogonal to the landslide direction in the in-situ direct shear test and the ring shear test, respectively. The landslide has experienced many years of deformation, and the cracks existing in the shear-zone soil may have an orientation toward the sliding direction. This can be evidenced by the slickenside observed in the shear-zone soil, as shown in Fig. 3. Since pre-existing cracks in the in-situ sample provide the easiest mechanism for the localization, to some extent, they have weakened the soil strength in the shearing direction.

The shear strengths of shear-zone soil are obtained from various tests on both intact soil and reconstituted soil with pure fine-graded particles (diameter $d < 2$ mm). The results shown in Fig. 11(b) are the relation between shear stress (or deviatoric stress in triaxial tests) and effective normal stress (or confining pressure in triaxial tests). Generally, the shear strengths obtained from various tests are concentrated in a certain range, giving rise to a friction angle of approximately $\phi' = 19.5^\circ$ and $\phi' = 26.4^\circ$ for shear tests and triaxial compression tests, respectively. Soil samples at relatively low effective normal stress attain a similar shear strength despite the intact coarse-graded and

Table 1
The program of all creep tests.

Creep test name, $\sigma'_n = 255$ kPa	Shear stress (kPa)	Time (hour)
In-situ direct shear creep test (long term)	96.5, 101.2	760
In-situ direct shear creep test (short term)	109.2, 112.8, 116.4, 120.5	210
Prepeak ring shear creep test (RS2)	155.4, 158.2, 161.5, 164.1	450
Postfailure ring shear creep test (RS1)	164.1, 146.3	65

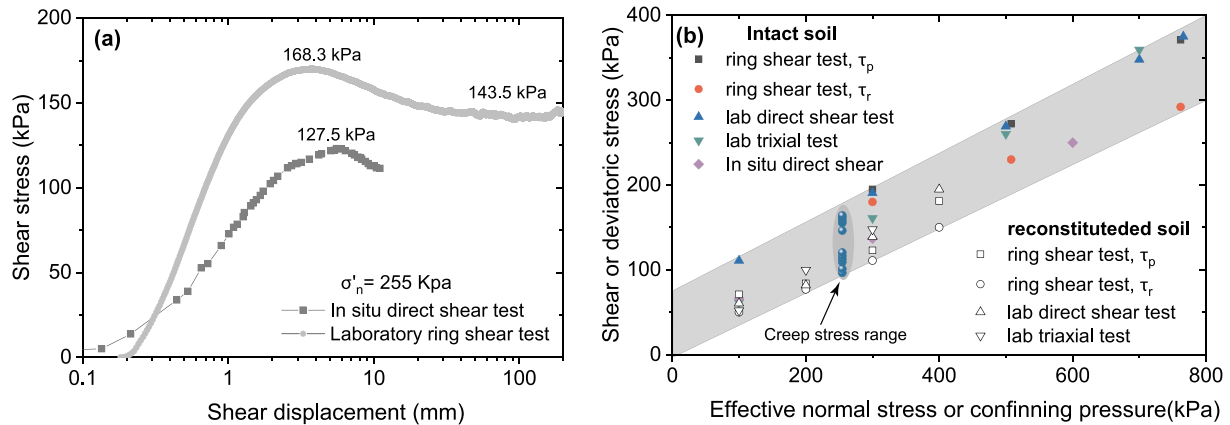


Fig. 11. (a) Shear displacement and shear stress curves in the in-situ direct shear test and ring shear test (sample RS1) with $\sigma'_n = 255$ kPa, (b) shear strength obtained from the ring shear test, triaxial compression test, laboratory and in-situ direct shear test; and τ_p and τ_r denote peak and residual strength, respectively.

reconstituted fine-graded soils being involved. The inspection of the shear strength suggests that intact samples tend to yield a greater variation in shear strength. The pronounced difference is assumed to be attributed to the influence of the coarse particles, overconsolidation or particle bonding in the soil (Wen and Jiang, 2017; Di Maio et al., 2013; Yin et al., 2016; Yin et al., 2017; Jiang et al., 2018).

5.2. In-situ direct shear creep test

As interpreted in subsection 3.3, the in-situ direct shear creep test was divided into two parts: a long-term creep test and a short-term creep test. The curves of creep displacement over time are shown in Fig. 12. The inspection of the creep curves reveals that each stage of the creep test starts with a rapid primary creep phase, which is followed by

a long-term secondary creep phase with nearly constant horizontal rate of displacement. The block sample experienced more than 12-mm shear displacement in the long-term creep test. In the short-term creep test, the shear stress increment was very small ($\Delta\tau = 3\text{--}4$ kPa). Each stage of the creep test might lead to more than 1.0-mm shear displacement. The block sample failed after 4-mm shear displacement at a shear stress of 120.5 kPa, which is very close to the assumed peak strength (127.5 kPa). The corresponding rate of creep displacement is presented in Fig. 13. The long-term creep test shows only attenuating creep with a relatively low creep rate. In the short-term creep test, the creep rate increases with the increase of the sustained creep stress. An increase in the shear stress to 120.5 kPa results in the activation of nonattenuating creep, during which three distinct stages of creep are observed.

The variation in the normal displacement over creep time is shown in Fig. 14. The initial normal displacement reached nearly 32 mm before starting the creep test, while the consequent 760-h long-term creep test and 450-h short-term creep test resulted in only approximately 1.2-mm and 1.8-mm normal displacement, respectively. A slight dilation was observed at the end of each stage of creep. This low compressibility is attributed to the overconsolidation of the shear-zone soil and the high content of coarse particles. The exposed shear-zone soil experienced an unloading process due to the excavation. Then, the voids and open cracks that were orientated in the shear direction were closed due to the normal stress and thus gave rise to relatively large initial normal displacement (32 mm). During the creep process, the crack may slightly open again due to the primary creep that occurred after the application of the shear force. This phenomenon is also reported in the in-situ triaxial compression test at the same testing site (Tan et al., 2018).

5.3. Prepeak ring shear creep test

The ring shear creep test was carried out by torque-controlled shear after 24 h of consolidation under the same normal stress as that employed in the in-situ creep test. The torque was applied step-by-step until creep failure occurred. According to the ring shear test detailed in subsection 4.1, the peak strength was assumed to be $\tau_{p2} = 168.3$ kPa. Multistage shear stresses with ten increments from $\tau = 0.60\tau_{p2}$ to

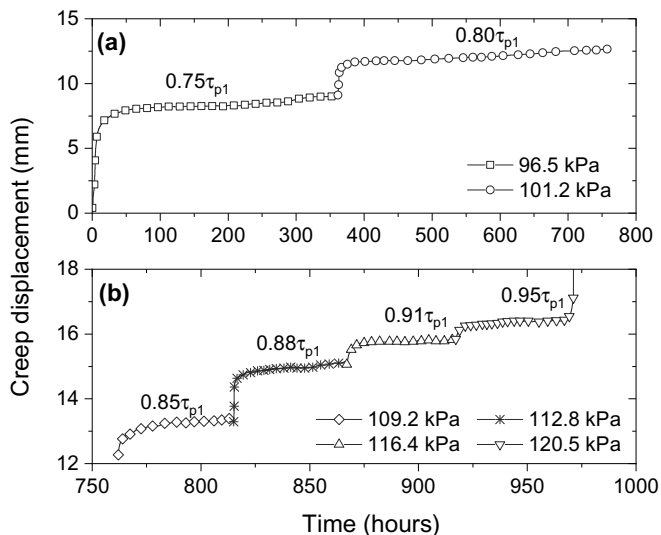


Fig. 12. Creep displacement against creep time in (a) long-term creep test and (b) short-term creep test ($\tau_{p1} = 127.5$ kPa).

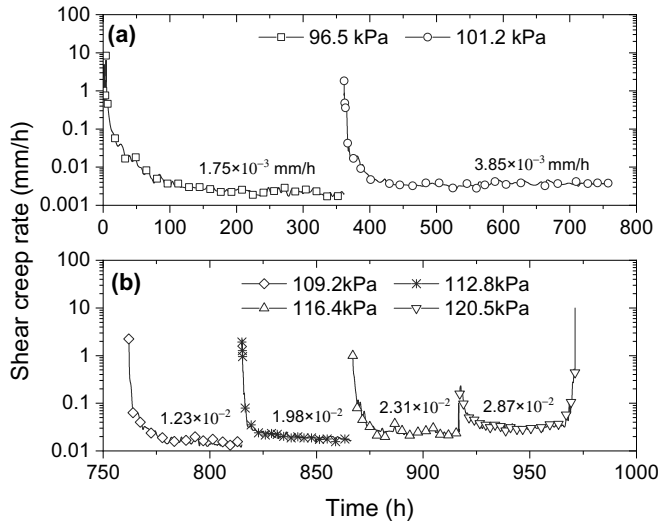


Fig. 13. Creep rate against creep time in (a) long-term creep test and (b) short-term creep test.

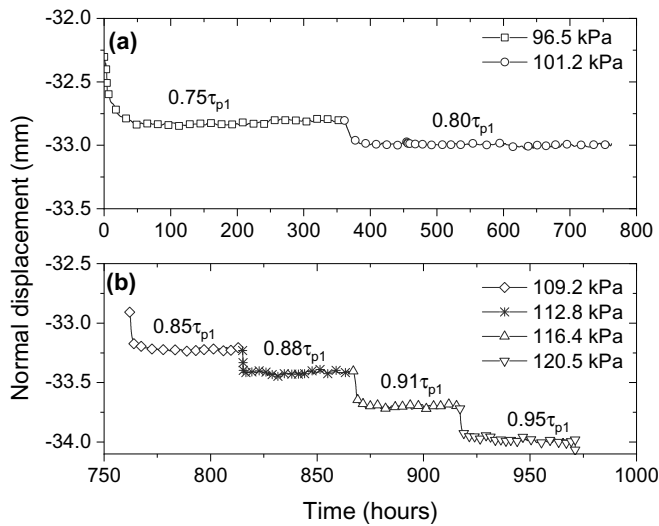


Fig. 14. Normal displacement against creep time in (a) long-term creep test and (b) short-term creep test ($\tau_{p1} = 127.5$ kPa).

$\tau = 0.98\tau_{p2}$ were applied on this sample to perform the ring shear creep test.

Fig. 15(a) shows the last four stages of creep curves at $\sigma'_n = 255$ kPa. With the increase in the shear stress, the secondary creep becomes increasingly more obvious, while it is not until the last stage of the shear stress of $\tau = 164.1$ kPa that the accelerated failure is observed. The applied shear stress leading to failure is slightly lower than the assumed peak strength, exhibiting an agreement with the observation in the in-situ creep test. As noted previously, the latter test yields a failure stress of approximately $0.95\tau_{p1}$. The relation of creep displacement with creep time appears to confirm the trend of the shear displacement at each stage of shear stress, as the secondary creep rate increased with the increase in shear stress, as shown in Fig. 15(b).

Fig. 15(c) compares the curves of accumulated creep displacement at various shear stresses and the ring shear displacement – shear stress curve extracted from Fig. 11(a). The dashed red line AB denotes the direction of the continuous creep displacement at the shear stress of $0.96\tau_{p2}$, at which the average rate of displacement at the secondary creep stage is evaluated to be $v = 1.47 \times 10^{-3}$ mm/h, constant for 100 h. Under the hypothesis of an indefinitely constant rate, creep will

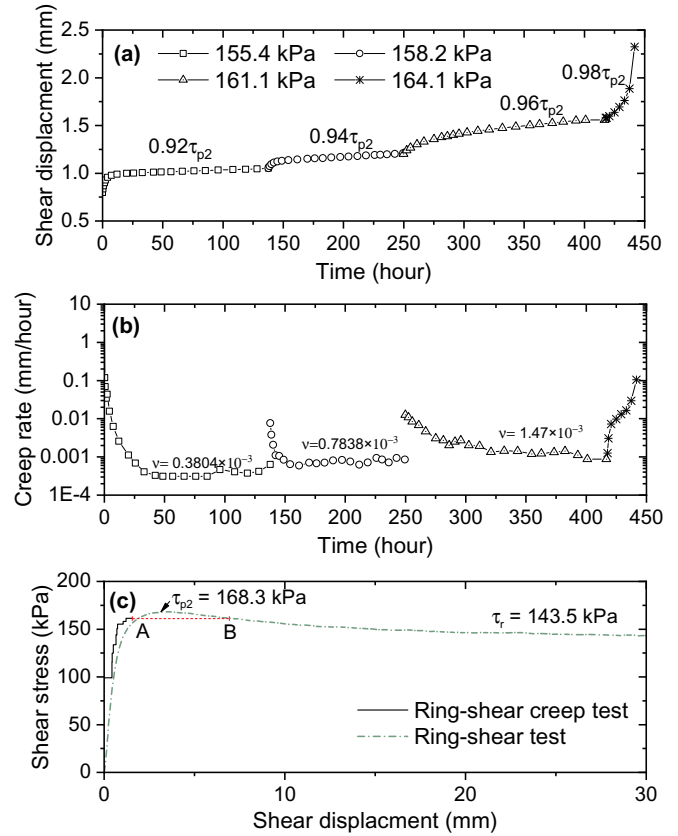


Fig. 15. Prepeak ring shear creep test on sample RS2 (a) creep displacement against time, (b) creep rate under various shear stresses against time, and (c) comparison between cumulative creep displacement at various shear stresses and shear displacements—shear stress curve obtained from ring shear test ($\tau_{p2} = 168.3$ kPa).

allow the sample to reach point A and B in 41 days and 154 days, respectively. In reality, however, the rate of creep displacement will increase with creep displacement. Jiang et al. (2018) reported that the creep stress leading to accelerated creep is between 69% and 95% of the peak shear stress; this implies that accelerated failure may occur at a displacement between A and B.

5.4. Postfailure ring shear creep test

The creep properties of shear-zone soil under the postfailure condition after large shear displacement are unclear due to the limitation of shear displacement in the in-situ direct shear test. This shortcoming, however, does not exist in the ring shear creep test. According to the results from the ring-shear test presented in Fig. 11(a), the soil sample may fail at approximately 4–5 mm shear displacement. The sample RS2 experiences only 1.6 mm of creep displacement during the prefailure ring shear test, so it is considered suitable for further testing the postfailure behavior under different shear stresses. In addition, the sample RS1, which is sheared to the residual state (see subsection 4.1) can be used to study the postfailure behavior as well. Thus, the sample RS1 is subjected to 146.3 kPa, which is slightly larger than its residual strength ($1.02\tau_r$), to perform the postfailure ring shear creep test.

The results of the postfailure ring shear creep test are shown in Fig. 16. The inspection of the curves suggests that the intact and pre-sheared samples exhibit different postfailure behavior. For the pre-sheared sample RS1, at $1.02\tau_r$, the shear displacement gradually increases with approximately 10.0 mm/h rate of displacement until 1.0 h of the test. Then, the displacement experiences approximately 3 h of increase with fluctuation of the rate of displacement. Subsequently, a

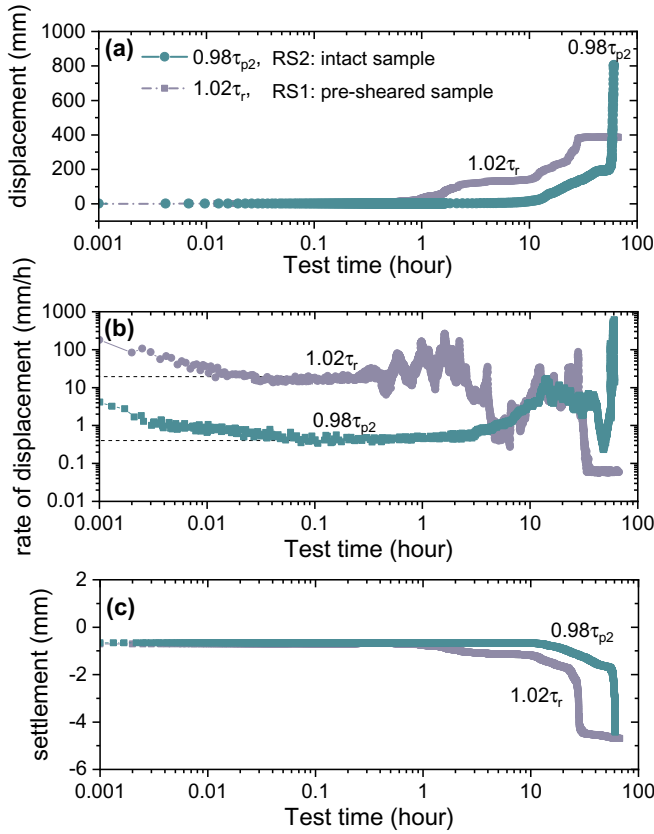


Fig. 16. Postfailure ring shear creep test (a) shear displacement over time, (b) creep rate of displacement over time, and (c) settlement over test time, $\tau_{p2} = 168.3$ kPa, $\tau_r = 143.5$ kPa.

stable displacement curve at approximately 130 mm is followed, corresponding to a significant decrease in the rate of displacement. This “static stage” is maintained for a few hours with a constant decrease in the rate of displacement. Then, the shear displacement rises steeply from 130 mm to 380 mm within 20 h, whereas it then ceased with a relatively low rate of displacement; however, this may not be the maximum attainable shear displacement. The shearing may be re-activated after a few hours of a dormant period. Correspondingly, the vertical displacement ceased after a rapid settlement. Similar to the preshear sample, the intact sample RS2, sheared at $0.98\tau_{p2}$, experienced an increase of displacement from 10 to 45 h before a short “static stage”

was attained, followed by a vertical rise of displacement, which brought the displacement to 800 mm in a few hours, indicating an accelerated failure in a manner of viscous flow. A similar vertical response is observed in sample RS2.

The comparison between the rate of displacement at various shear stresses shows that the sample RS1, sheared at $1.02\tau_r$, yields a much higher secondary creep rate of displacement (see Fig. 16(b)). This result may be attributed to the previous shear process, which has brought the sample to residual state and formed a complete shear surface in which the orientation of particles occurred. On the other hand, without the pre-existing shear surface in the RS2 sample, it withdraws to a relatively low rate of displacement. According to the study of Wang et al. (2018b), a shear stress that is slightly larger than the residual strength could lead to an accelerated failure. Sample RS2 is subjected to a stress of 164.1 kPa, which is equivalent to $1.15\tau_r$. This suggests that once a complete shear surface is formed in sample RS2, the rate of displacement rises rapidly and exceeds the rate of displacement in sample RS1.

6. Discussion

Different testing approaches and shearing directions can lead to significantly different viscous responses. The ring shear test in the laboratory yields a greater peak and residual strength than that tested from the in-situ direct shear test (see Fig. 11(a)). Similar creep curves can be observed in the ring shear and direct shear creep tests. However, a much larger rate of creep displacement can be observed in the direct shear creep test because the sample is sheared toward the landslide moving direction (see Fig. 7). Once a complete shear surface is formed in the sample, a significant increase in creep rate can be observed in the ring shear creep test (see Fig. 16(a)). Thus, the rate of creep displacement depends on the applied shear stress, shear direction and other time-dependent state variables, e.g., residual strength.

6.1. Comparison with triaxial compression creep tests

The curves describing the time-dependent deformation of intact shear-zone soil under various stress states in the laboratory and in-situ triaxial compression creep tests are reproduced in Fig. 17. In contrast to the in-situ and laboratory shear tests, no accelerated failure was observed in either the laboratory or the in-situ triaxial creep tests. For example, the laboratory triaxial compression test yielded a peak deviatoric shear strength σ_{pd} (260.2 kPa) under 500 kPa confining pressure. The sample, with $0.95\sigma_{pd}$ loading deviatoric stress, gave rise to a primary and a secondary creep. On the other hand, the in-situ triaxial creep test with loading deviatoric stress, e.g., 600 kPa, which is much

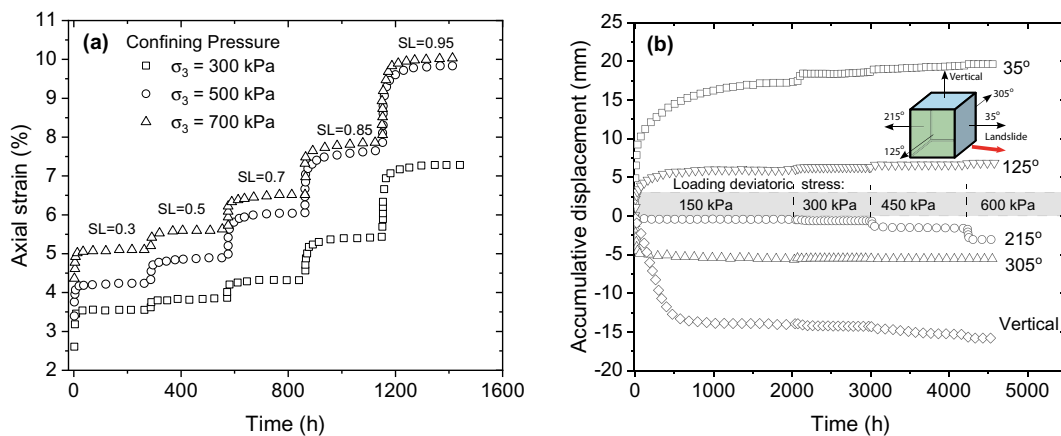


Fig. 17. (a) Creep strain - time curve in the triaxial creep test in the laboratory, where SL denotes the ratio of loading deviatoric stress to the peak deviatoric stress (σ_{pd}); data from Li et al. (2017); (b) displacement - time curve in the in-situ triaxial creep test. Negative values indicate shrinking deformation, with confining pressure $\sigma_3 = 500$ kPa; data from Tan et al. (2018).

larger than the ultimate peak deviatoric stress σ_{pd} attained in the laboratory, exhibits a response that is similar to the laboratory test. The differences in creep patterns observed in triaxial creep tests and shear creep tests may be attributed to the boundary conditions. Specifically, the former test does not prescribe a failure plane, while the latter test assumes a failure surface between the upper and lower shearing box. Normally, the failure surface is represented by a 2- to 3-mm shearing gap, where no lateral constraints are applied. In addition, the initial stress state may also influence the creep response, since the samples were consolidated at isotropic stress states and at K_0 states in the triaxial tests and the shear tests, respectively.

The phenomenon of creep failure has long been ascribed to the accumulation of structural defeat and the consequent formation of failure surfaces in the soil under the sustained loading (Vyalov, 1986, and Suklje and Šuklje, 1969; Yin et al., 2012; Wang et al., 2018c). For the in-situ direct shear creep test, as shown in Fig. 7, the shearing direction is parallel to the shear surface, where the weakness has already formed. Thus, failure is bound to occur if the sustained creep stress is larger than a creep failure threshold. On the other hand, for the in-situ triaxial creep test, as shown in Fig. 17(b), the lateral surface (35° direction) with the similar direction to the landslide gained the largest deformation. A closer inspection of the creep deformation reveals that a significant lateral displacement took place in the 35° direction when 300-kPa deviatoric stress was applied. It is therefore reasonable to presume that creep failure might have already occurred at the second stage of stress.

A comparison of all the creep curves obtained from different testing approaches suggests that the shear-zone soil exhibits a similar secondary creep deformation pattern, while the rates of creep displacement are rather different. Overall, the rate of creep displacement is greater in the in-situ tests than that obtained from laboratory tests. The largest rate of creep displacement is obtained from the in-situ shear creep test. This is attributed to the fact that, for the in-situ direct shear creep test, the shearing direction is the same as the landslide moving direction. Although the in-situ triaxial creep test was not carried out in the same manner, a dominating weak plane in the 35° direction was accommodated in the sample. In contrast to the in-situ tests, there was not a dominating weak plane in the sample for the laboratory test, as shown in Fig. 7. During the shear creep process, orientation rearrangement of the fine particles may occur with time and thus results in the accumulation of structural defeat in the soil, leading to an increase in creep displacement in the direction of orientation. This degradation process, however, is much slower than that occurred in the in-situ creep tests.

The size of the samples may account for another factor that influences the viscous response. Especially, in the laboratory tests with relatively small samples, the presence of coarse particles in the shear zone may play an important role in affecting the stress field and the failure mechanism in the soil (Xu et al., 2007; Xu et al., 2008). In the laboratory triaxial creep tests, the position of coarse particles in the samples may not change significantly; instead, some particles may rotate resistant to the sustained creep stress in the sample, thereby leading to a relatively low rate of creep displacement. This effect can be visualized by CT scanning the distribution of coarse particles before and after the tests, as reported by Li et al (2017). In the ring shear creep tests, localization can only occur at the prescribed shear surface. Increasing the content of coarse particles in the shear zone will significant increase the sustained stress leading to creep failure (Wen and Jiang, 2017). In the in-situ creep tests, however, the effect of coarse particles may be negligible, since the size of samples is much larger than the contained coarse particles. The samples may behave in a similar mechanism as the gravel-free soil. Jiang et al. (2016) reported a group of large-size stacked ring shear tests on reconstituted shear-zone soil. The size of the samples were $0.6 \times 0.6 \times 0.6$ m. The results show similar strength response with conventional direct shear tests on gravel-free shear-zone soil.

6.2. Link creep failure to landslide movement

The experimental results in Fig. 16 may correspond to the catastrophic movement of two types of landslide: one is the new slow-moving landslide, and the other is a reactivated slow-moving landslide. As previously discussed by many authors (Deng et al., 2000; Wang et al., 2018b; Li et al., 2017; Tan et al., 2018), the Huangtupo landslide is a typical reactivated slow-moving landslide. Its stepwise movement pattern is similar to the observed creep behavior in the postfailure ring shear creep test for a presheared sample under a shear stress of $1.02\tau_r$. It is, therefore, believed that the stress state of the Huangtupo landslide is apparently coincident with the presheared sample, from the perspective of the displacement pattern characteristics.

Some authors (Hu et al., 2018; Leroueil, 2001; Wang et al., 2010; Ter-Stepanian, 1975; Lucas et al., 2014) suggest that the rate effect may be attributed to this stepwise displacement pattern. A soil may show slight rate-weakening behavior at low rates and a rate-strengthening behavior at comparatively higher rates (Scaringi et al., 2018). In the postfailure ring shear tests, the sustained stress $1.02\tau_r$ rendered the sample RS1 a creep rate approximately 20 mm/h at the secondary creep stage, and the rate increased with time. In this rate range, the shear-zone soil may exhibit rate-strengthening behavior. As shown in Fig. 18, the residual strength increases with the rate of displacement ranging from 12 to 2400 mm/h (Hu, 2012). This implies that the residual strength is rate-dependent and can be significantly higher than the residual shear strength obtained at the rate of displacement of 0.05 mm/min. While the sustained stress is $1.02\tau_r$ during the whole test, the applied stress level, i.e., the ratio between the driving shear stress and the available shear strength, can vary. At the beginning of the test, the rate of displacement is relatively low, and the residual shear strength attains its minimum value. After a certain creep displacement, the rate of displacement increases and, as a feedback, the rate-dependent residual strength, $\tau_r(v)$, increases. This gives rise to a “static stage” in the displacement with a relatively low rate of displacement. Thus, the rate-dependent threshold, $\tau_r(v)$, attains a relatively low value, and accelerated failure may occur again. In a reactivated slow-moving landslide, the rate-dependent residual strength largely affects its movement pattern. The critical creep stress is equal to or slightly larger than the residual strength but lower than the peak strength, as also reported in the literature (Bhat et al., 2011; Bhat et al., 2013; Hu et al., 2018); therefore, a rate-strengthening behaviour can prevent dramatic acceleration and long runout (Leroueil, 2001; Wang et al., 2010; Hu et al., 2018).

On the other hand, the intact sample RS2, although sheared at a stress less than the peak shear strength, experienced a large creep displacement followed by a catastrophic failure. At the beginning of the test, the sample creeps at low rate of displacement. During the creep,

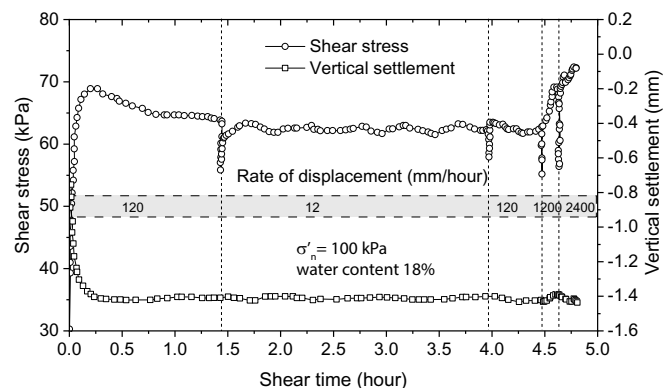


Fig. 18. Ring-shear test on a remould, consolidated under an effective normal stress of 100 kPa, presheared to the residual condition with 120 mm/h and then sheared under increasing displacement rates, from 12 to 2400 mm/h, data from Hu (2012).

the shear stress gives rise to interlocking loss and crack propagation in the sample, leading sliding between particles accumulate in the shear zone (Kuhn and Mitchell, 1993; Yin and Chang, 2009). Such a process can result in excessive viscous displacement, and eventually leads to accelerated deformation in the sample. Then, the resistant stress rapidly drops to the rate-dependent residual strength. Whilst the $\tau_r(v)$ may increase with the increase in the rate of displacement, it is still much lower than the sustained shear stress. Consequently, an acceleration failure will definitely occur. In a new slow-moving landslide, the accumulation of the viscous displacement may be the dominating factor that leads to the failure. The critical creep stress can be slightly lower than the peak shear strength but much larger than the residual strength. This results was also experimentally observed by Jiang et al. (2016). In this case, catastrophic failure with extremely fast velocity may occur if the soil experiences a sudden reduction in the peak shear strength due to factors such as heavy rainfall events (Xu et al., 2015; Xu et al., 2016).

7. Conclusions

This paper reports on the results of an in-situ direct shear creep test carried out at the Badong field test site. The creep properties of the clastic soil within the shear zone are examined over various shear stress ranges. Additionally, some laboratory ring shear creep tests are carried out with the same soil. The test results are compared with those obtained by triaxial creep tests reported in the literature. The following conclusions can be drawn:

- The in-situ direct shear creep test yields a failure stress of approximately 0.95 of the peak shear strength. Compared to the ring shear creep test, a much larger rate of creep displacement is obtained due to shearing along the landslide direction.
- The tested sample in the in-situ direct shear creep test exhibits very low compressibility during the creep due to the high content of coarse particles (25 and 50 wt%) and overconsolidation.
- In the ring shear creep tests, the critical creep stress is approximately 0.98 of the peak shear strength at the prepeak state. However, at the residual state, the critical creep stress is equal to the residual strength of the soil.
- The shear-zone soil is prone to failure under shear condition, rather than triaxial compression condition. Nonattenuating creep responses were obtained in both the in-situ direct shear and the laboratory ring-shear creep tests, while no accelerated failure was observed in either the laboratory or the in-situ triaxial creep tests.

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.

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