

Chapter 8

Mechanism for the Rapid Motion of the Reactivated Qianjiangping Landslide in Three Gorges Dam Reservoir, China

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Abstract The first impoundment of the Three Gorges Dam reservoir in China started from a water-surface elevation of 95 m on June 1, 2003 and reached 135 m on June 15, 2003. Shortly after the water level reached 135 m, many slopes began to deform and some landslides occurred. The Qianjiangping landslide is the largest one; it occurred on the early morning of July 14, 2003, and caused great loss of lives and property. Field investigation revealed that although failure occurred after the reservoir reached 135 m, the stability of the slope was already reduced by pre-existing, sheared bedding planes. To study the mechanism of the rapid motion of this reactivated landslide, two soil samples were taken from a yellow clay layer and a black silt layer in the sliding zone and a series of ring shear tests were conducted on the samples. One series of ring shear tests simulates the creep deformation behavior, while the other series simulates different shear rates. Conclusions drawn from analysis of the ring shear tests indicate that the mechanism of the rapid motion of the reactivated landslide was caused by the rate effect of the black silt layer during the motion phase after the creep failure. The yellow clay layer did not play any important role in the rapid motion in the 2003 event.

Keywords Landslide · Rapid motion · Ring shear tests · Clay · Silt · Water–soil interaction · Rate effect

Introduction

The Three Gorges Dam construction on the Yangtze River in China is the largest hydroelectricity project in the world. The dam site is located at Sandouping village near Maoping, the capital of Zigui County, Hubei Province. The designed final dam height is 185 m, the final length 2309.5 m, and the designed final highest water

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level 175 m. When dam construction is finished, the Three Gorges Reservoir will reach Chongqing city, about 660 km upstream from the dam. The first impoundment started from 95 m on June 1, 2003, and reached 135 m on June 15, 2003. Shortly after the water reached 135 m, many slopes began to deform and some landslides occurred.

In the early morning, at 00:20 on July 14, 2003, the Qianjiangping landslide occurred at Shazhenxi town (Fig. 8.1) beside Qinggan-he River, a tributary of the Yangtze. The Qianjiangping landslide was located on the western side of Qinggan-he River. On the opposite side of the river is the main street of Shazhenxi. The distance from the landslide to the junction of the Qinggan-he River with the Yangtze is about 3 km, and the distance along the Yangtze River from the junction to the Three Gorges Dam is about 50 km (the direct distance is about 40 km).

There was some loss of life and serious economic damage caused by the Qianjiangping landslide. It destroyed 346 houses and 70 ha of fields and rice paddy. Four factories on the lower part of the slope near Qinggan-he River (a brick factory, a metal and silica factory, a chemical fertilizer factory, and a food factory) were seriously damaged. Direct economic losses were about 7 million USD, and it reduced the asset value at Qianjiangping by 40%. Most of the workers became unemployed. In addition, 3 km of provincial roads and 20.5 km of electricity lines were cut. Twenty-two boats and ships were damaged and sunk in Qinggan-he River and the Yangtze River. Although a warning was given by the local government based on precursory deformation of the slope 2 h before the final failure, 13 people on the slope and 11 fishermen on boats in the nearby area were killed. The main reason for the deaths on the slope was that the people did not imagine that the landslide area would be so large and believed their houses would be safe, because there was no ground deformation around them before the final failure of the slope. For the deaths on the river, the reason was just that it was not predicted that the landslide could



Fig. 8.1 Location map of the Qianjiangping landslide in the Three Gorges Reservoir area, Hubei Province, China

Fig. 8.2 View of the Qianjiangping landslide from the upstream side of Qinggan-he River (taken by F.W. Wang, March 15, 2004)



move so rapidly. It was the wave caused by the rapid sliding that killed the fishermen on their boats. A water trace left on the red bridge (shown in Fig. 8.2) detected after the landslide indicated that the highest level of the wave was about 30 m above the water level of 135 m.

Features of the Landslide

Figures 8.2 and 8.3 are photographs of the landslide taken from the upstream side and front of the landslide, respectively. The landslide had a tongue-shaped plan, with a length of 1,200 m and a width of 1,000 m. It moved about 250 m in the main sliding direction of S45°E. The average thickness of the sliding mass was about 20 m, thinner in the upper part and thicker at the lower part. The total volume was estimated to be more than 20 million m³. The elevation of the main scarp was 450 m, and the elevation of the Qinggan-he River water level was 135 m when the



Fig. 8.3 Front-on view of the Qianjiangping landslide (taken by Y.M. Zhang, July 15, 2003)

Fig. 8.4 View of the unrotated trees on the sliding mass in the middle of the Qianjiangping landslide



landslide occurred. The landslide release surface was along a bedding plane in the bedrock (Fig. 8.2). Factory buildings on the sliding mass still remained standing after sliding for about 250 m (Fig. 8.3). However, because serious cracks developed in the buildings, they could not be used again and were demolished and the building materials were recycled (compare with Fig. 8.2). Standing trees on the sliding mass in the middle of the landslide (Fig. 8.4) indicate that the angle of the sliding surface remained constant and no rotation occurred. The exposed sliding surface at the upper part was very planar and subparallel to the sandstone bedrock strata (Fig. 8.5). All of these phenomena show that the sliding mass slid along a planar sliding surface. When the sliding mass entered Qinggan-he River, the dip direction of the strata was changed to N45°W, which is opposite to the original dip direction of S45°E. The dip angle is about 5° in the bed of Qinggan-he River (Fig. 8.6). The deposits at

Fig. 8.5 The exposed planar sliding surface, which is also a sandstone bedding plane (there is a damaged rice paddy in front of the person)



Fig. 8.6 View of the reversed dip direction of the sandstone beside Qinggan-he River (taken by F.W. Wang, March 16, 2004)



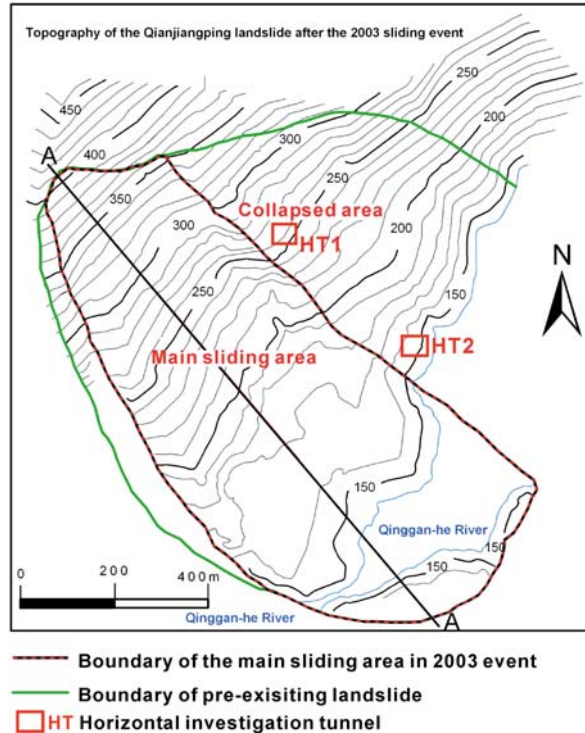
the distal landslide margin contain white gravel with clasts up to 100 mm or so in diameter. The dip angle of the sandstone bedding at the distal margin is steeper than 30° (Fig. 8.7).

Figure 8.8 is a topographic map of the Qianjiangping landslide prepared after the 2003 event (modified from Wu et al. 2006). The water level of the Qinggan-he River changed from 95 to 135 m when the first impoundment of the Three Gorges Dam reservoir was conducted. The Qinggan-he River made a big curve in front of the Qianjiangping slope, and erosion by the river at the toe of the slope probably had a negative effect on the stability of the slope. The highest elevation of the main scarp is about 420 m. The landslide has a tongue-like shape in plan view, with a length of 1,200 m and a width of 1,000 m. It moved about 250 m in the main sliding direction



Fig. 8.7 Gravels displaced from the bed of Qinggan-he River and deposited in the toe of the landslide deposit (circled in photograph). Note that the sandstone dip direction is reversed and the dip angle has become steeper compared with Figs. 8.4 and 8.5)

Fig. 8.8 Topography of the Qianjiangping landslide after the 2003 sliding event (modified from Wu et al. 2006)



of S45°E. The average thickness of the sliding mass was about 26 m, thinner in the upper part and thicker at the lower part. The total volume was estimated to be more than 20 million m³.

Based on the sliding distance of the 2003 event, the Qianjiangping landslide was divided into two areas: the main sliding area and the collapsed area. There was no obvious difference between the two areas before the 2003 event. However, they became quite different after the 2003 event. In the main sliding area, the sliding plane was exposed at the main scarp, and the toe slid into the Qinggan-he River, damming the river flow. The collapsed area was just drawn down by the movement of the main sliding area. Many cracks were found in the collapsed area, but its sliding distance was very limited in the 2003 event. According to an analysis of the topographic features of the two areas, it is believed that the two areas moved together for a very long period before the 2003 event and that their sliding planes should be the same surface or should have exhibited very similar behavior.

Generally, reactivated landslides cannot move at high velocities, because the material in the sliding zone was previously crushed to clay and little excess pore pressure can be generated during the later landslide motion. However, the Qianjiangping landslide showed a behavior that was different from this scenario. To clarify the mechanism of this high-velocity reactivated landslide, more detailed analyses

were conducted on the materials taken from the sliding zone of the Qianjiangping landslide, including grain size distribution, clay mineral components, and mechanical properties. As a typical dip-slope landslide, it attracts investigations by many landslide researchers (Wang and Yang 2005; Cao et al. 2007; Yin and Peng 2007).

In order to investigate the sliding zone of the 2003 landslide in more detail, two horizontal investigation tunnels (HT1 and HT2) were excavated by China Three Gorges University in the collapsed area to check the properties of the sliding zone as they existed before the 2003 event. The locations of the two tunnels are plotted in Fig. 8.8.

Figure 8.9 is a longitudinal section of the main sliding block of the Qianjiangping landslide. The bedrock in the landslide area is feldspathic quartz sandstone, fine sandstone with carbonaceous siltstone, siltstone with mudstone, and silty mudstone of the Lower Jurassic Niejiashan formation (J_{1-2n}). The dip direction of the strata is reversed near the Qinggan-he River. The average slope angle of the slide plane in the upper part is 24° . An energy line connecting the top and toe of the sliding mass is inclined at 10° , which is the apparent friction angle of the landslide. The 10° value indicates a highly mobile landslide.

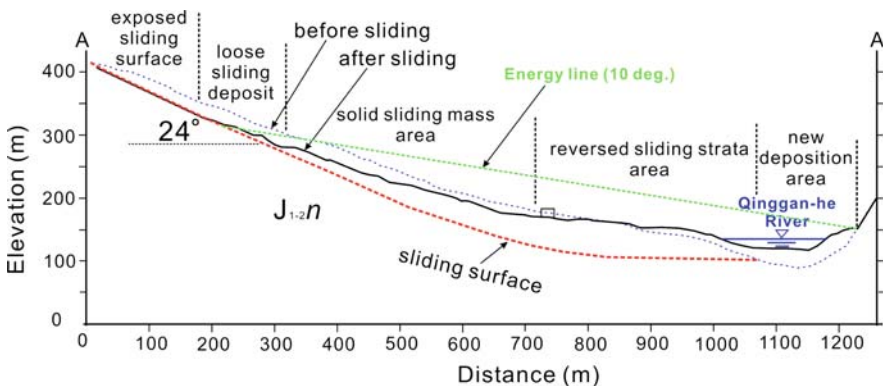


Fig. 8.9 Central longitudinal section of the Qianjiangping landslide

Scratches on the Failure Surface

Figure 8.10 shows a sequence of two photographs of scratches on the sliding surface at the upper part of the landslide. Figure 8.10a shows the original situation, while in Fig. 8.10b a slice of sandstone has been stripped away from the plane to reveal more scratches. This shows that the scratches were present beneath the landslide sliding surface before the landslide occurred. The scratch strike direction is $S15^\circ W$ (red arrow). The sliding direction ($S45^\circ E$) of the July 2003 event (blue arrow) is also shown by the water dribble trace in Fig. 8.10b. The angle between the sliding direction and the scratches is about 60° . Comparison of the two photographs shows that the scratches were not formed by the event of July 2003. They must have

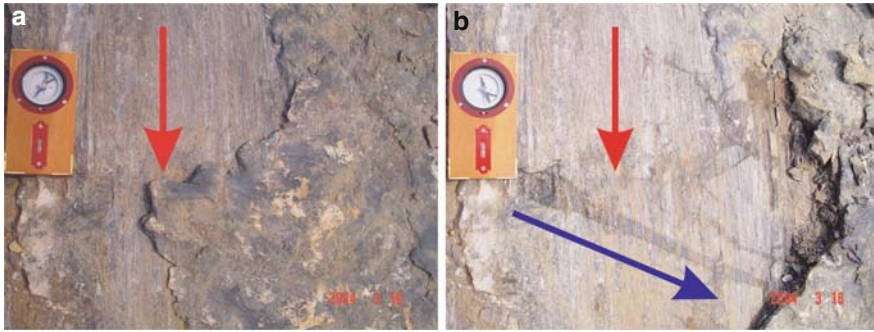


Fig. 8.10 A pair of photographs at the same site showing the pre-existing scratches that underlie the sliding surface of the July 2003 event (*blue arrow* shows the sliding direction, *red arrows* show the strike direction of the pre-existing scratches). The scratches are inferred to be slickensides formed by bedding-plane shear during folding of the rocks in the Cretaceous period

formed earlier, probably in a much older geological event, because calcite veinstone is widely distributed along them. This site lies between the Zigui syncline and Baifulai-Liulaiguan anticline that were folded during the Cretaceous period (Wang et al. 2002) and so the scratches can be interpreted as slickensides along bedding-plane shears.

Possible Triggering Factors and Sliding Mechanism

The high water level within the landslide toe caused by impounding of the Three Gorges Reservoir was naturally considered to be the trigger. Impoundment started from June 1, 2003, and the reservoir water level reached 135 m on June 15, 2003. The first cracks due to the slope deformation, however, were observed on October 22, 2002, near the present main scarp. This means that the slope was in a critical state even before impoundment of the reservoir. With the slope in such a critical state, failure probably was triggered by the direct reduction in normal load within the toe of the slope caused by the rising water level.

According to local people, sandstone and mudstone were exposed in the Qinggan-he River bank before the reservoir was impounded. The dip angle of the sliding surface, which is also a bedding plane, was measured as 32° at the upper part. Zhang et al. (2004) observed two sets of large transversal cracks crossing the upper and middle parts of the slope. Although some rice paddies were located on the upper part of the landslide (Fig. 8.3), the associated high water table from irrigation should have been kept perched by the impervious weathered mudstone. The rice paddy was started 10 years ago, and a pond for water supply was built near the paddy. So, for landslide stability analysis, the boundary conditions for the landslide are very clear. There was little volume of landslide in the uppermost part. The right lateral boundary was open and so had no friction to resist sliding. As shown in Fig. 8.10, the sliding surface was along a pre-existing structural shear plane.

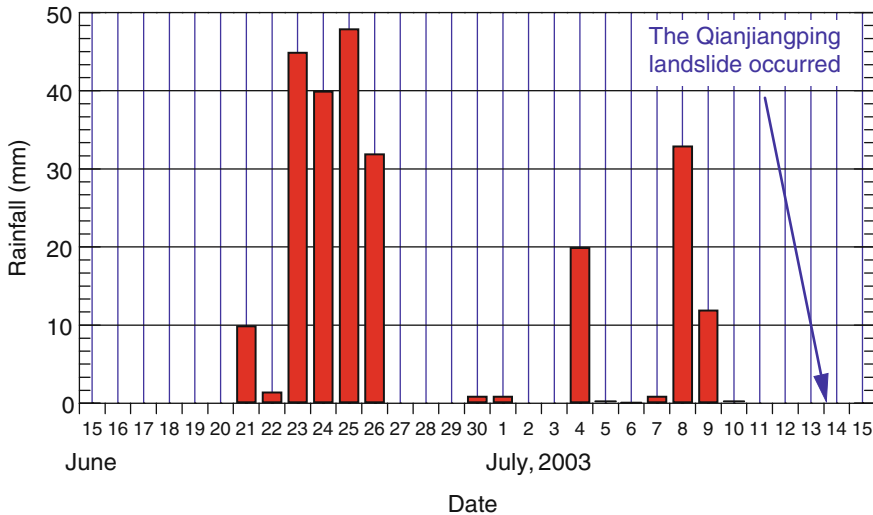


Fig. 8.11 Precipitation data monitored in Yichang city, 70 km from the landslide

Precipitation is monitored in Yichang city (Fig. 8.1), about 70 km from the landslide site. Intense rain from June 21 to June 26 and rain from July 4 to July 9, 2003 (Fig. 8.11) may have saturated the sliding mass and increased its unit weight. However, considering the high permeability of the slickensided sandstone and the transverse cracks crossing the sliding mass, it is not likely that high pore pressure would have resulted.

The landslide toe had been eroded by the Qinggan-he River long ago and offered little resistance to sliding. Only the left lateral boundary offered side friction and tension resistance from the neighboring rock mass. For such a huge landslide, the mechanical model can be simplified as a two-dimensional longitudinal section as shown in Fig. 8.12a. The situation of the landslide after failure is sketched in Fig. 8.12b.

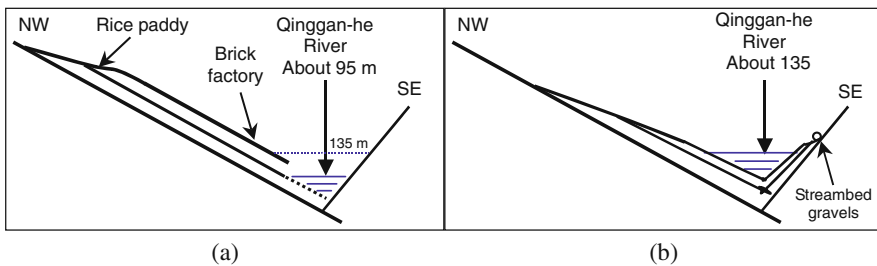


Fig. 8.12 (a) Structural model of the Qianjiangping slope and (b) a sketch of the landslide after failure

With the slope in a critical state (Fig. 8.12a), an increase in water level in the river would decrease the effective normal stress in the toe of the slope, and the shear resistance would decrease at the same time. These changes in the mechanical balance resulted in the slope failure.

For additional factors contributing to the failure, some attention should be paid to the factories on the lower slope before the landslide occurred. The brick factory had been quarrying mudstone in the lower part of the slope as raw material for brick making from 1997. Some 2–3 million bricks (each about $250 \times 120 \times 50 \text{ mm}^3$) had been produced during the 6 years. Such removal of mass from the toe of a landslide is a very dangerous action that leads to slope instability (but the presence of the landslide was unrecognized by experts until the cracks appeared in 2002).

When the existence of the widely distributed scratches on the sliding surface is considered, it is easy to assume that the shear strength between the sliding mass and the sliding surface was at residual strength. If this assumption were true, the rapid, long-runout sliding would be difficult to explain, because a rapid loss of shear strength is necessary to achieve the high rate of acceleration. As mentioned previously, the scratches probably were formed in the Cretaceous period, and the beds on either side of the scratched surface were bonded together with calcite cement. After the occurrence of the landslide, Zhang et al. (2004) observed crushed crystalline calcite, some 20–30 mm thick, widely distributed on the exposed sliding surface on the upper part of the landslide. Figure 8.13 shows a sample of the calcite. Generally, failure of crystalline calcite is characterized by brittle fracture. After a certain small distance of shearing, the cement bonding would have been destroyed. The quick loss of adhesion of the crystalline calcite could be the main reason for the rapid landslide acceleration after the initial failure of the slope.

We have to mention that the above estimation was made after the first investigation on the Qianjiangping landslide (Wang et al. 2004). Later, after many times field investigation, especially after horizontal tunnels were excavated in the landslide site, we got some new findings presented in the following section (Wang et al. 2008).



Fig. 8.13 Photograph of a crystalline calcite sample taken from the sliding surface at the upper part (The white color is crystalline calcite)

Experimental Study on the Rapid Sliding Mechanism of the Reactivated Landslide

Sampling the Sliding Planes

Observation and sampling were conducted in tunnel HT1. Figure 8.14 shows two photographs taken in the tunnel that face the sliding direction and show the sliding zone. In the investigation tunnel, two layers were observed in the sliding zone. The upper layer is a black clay layer with calcite blocks, while the lower layer is yellow clay. Both layers were water saturated with a thickness of 200 mm and were in a plastic state.

In situ observations reveal that the yellow clay layer may be the old sliding plane because the percentage of fine-grained particles was higher in the yellow layer than in the black layer. It is likely that the black layer formed in the 2003 event. Numerous

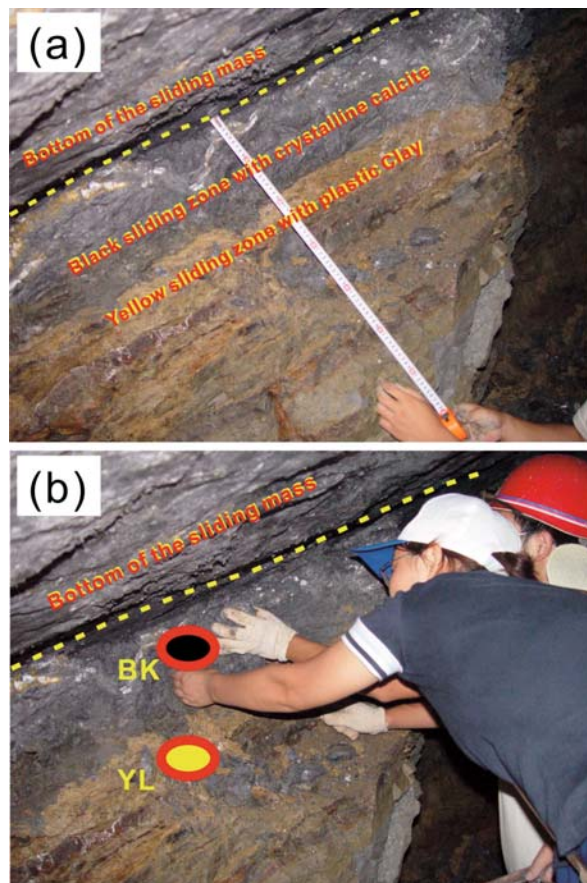


Fig. 8.14 Photograph of the sliding zone in a horizontal investigation tunnel. (a) Under the bottom of the sliding mass, the sliding zone can be observed to consist of two layers. The upper layer is black and includes crystallized calcite, and the lower layer is yellow plastic clay with a high water content. (b) Sampling from the black silt (BK sample) and yellow clay (YL sample) layers of the sliding zone

small blocks of crushed calcite were found in the black layer. It is reasonable to consider that the black clay layer was derived from the original black mudstone during the recent sliding. For laboratory tests, soil samples were taken from the two layers and named as the black sample (BK sample) and the yellow sample (YL sample).

Soil Properties

To clarify the basic properties of the two samples, grain size analyses, Atterberg limit measurements, and X-ray diffraction analyses were conducted on the two samples. Figure 8.15 shows the grain size distributions (Fig. 8.15a) and soil classification (Fig. 8.15b) of the two samples. It is clear that the YL sample has more fine particles than the BK sample. In the I_p - W_L chart, the YL sample is classified as CL (clay with a low liquid limit), while the BK sample BK is classified as ML (silt with a low liquid limit).

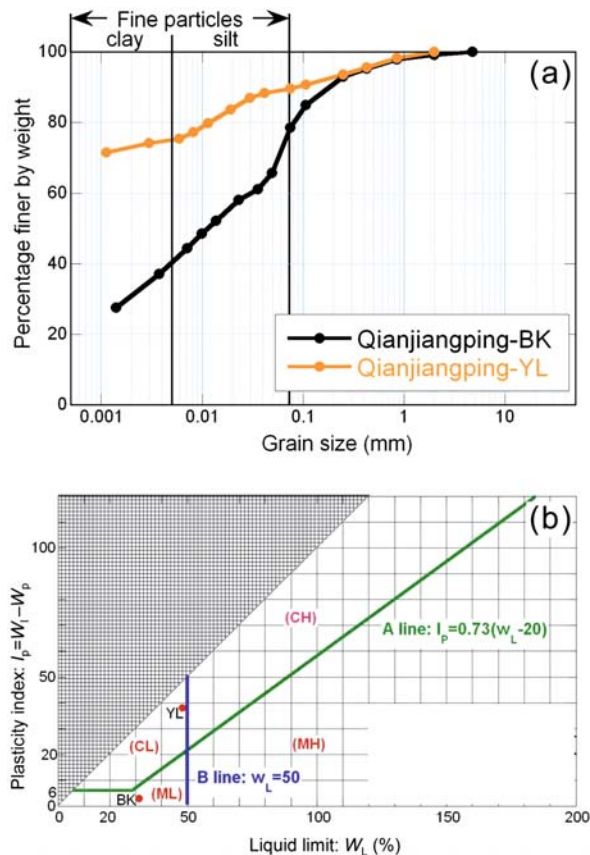


Fig. 8.15 Properties of the samples taken from the slide zone. (a) Grain size distribution of the samples. (b) Soil classification in the I_p - W_L chart. The black silt sample is labeled BK, and the yellow clay sample is labeled YL

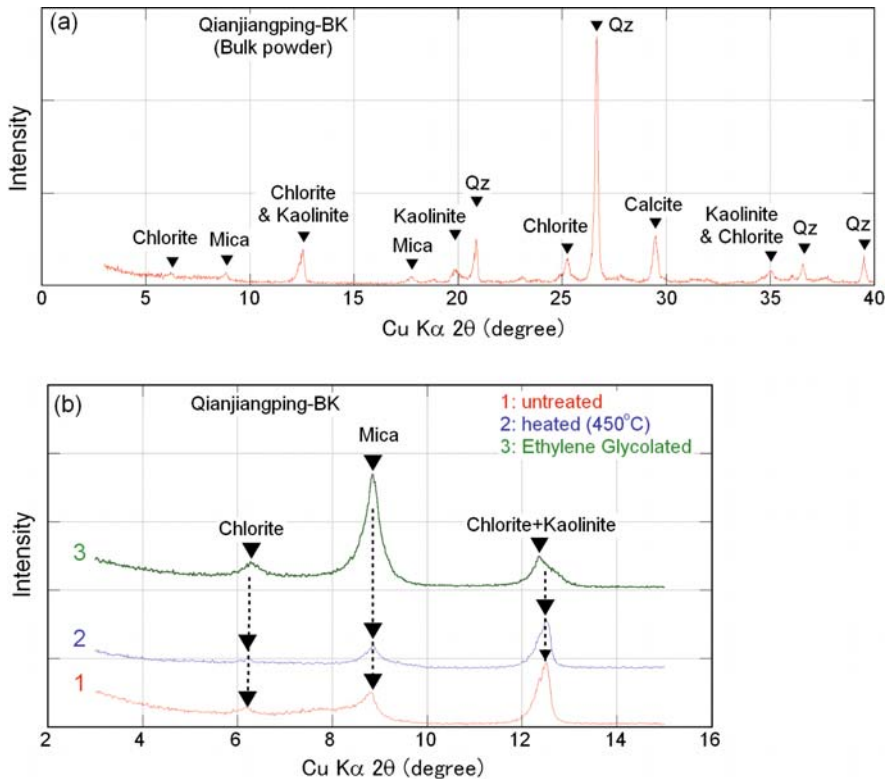


Fig. 8.16 X-ray diffraction pattern for the black silt sample (BK sample)

Figures 8.16 and 8.17 show the X-ray diffraction patterns of the BK sample and the YL sample, respectively. Figures 8.16a and 8.17a show the results for bulk powders of the two samples. Figures 8.16b and 8.17b show the results in the range from 3 to 15° for heating treatment to 450°C and treatment with ethylene glycol (Yoshimura 2001). Comparison of the BK sample (Fig. 8.16) with the YL sample (Fig. 8.17) indicates the following tendencies:

- (1) There is no chlorite in the YL sample.
- (2) There is no calcite in the YL sample.
- (3) Chlorite–vermiculite appears as a weathering product in the YL sample.

From these results, it can be determined that the BK sample and the YL sample have the same rock source and that the YL sample is more maturely weathered than the BK sample. Through long-period shearing, grain crushing could occur in the sliding zone. As the result, the silt would become too clay. Moreover, oxidation due to groundwater flow by the water–soil interaction through the sliding zone changed the soil color from black to yellow.

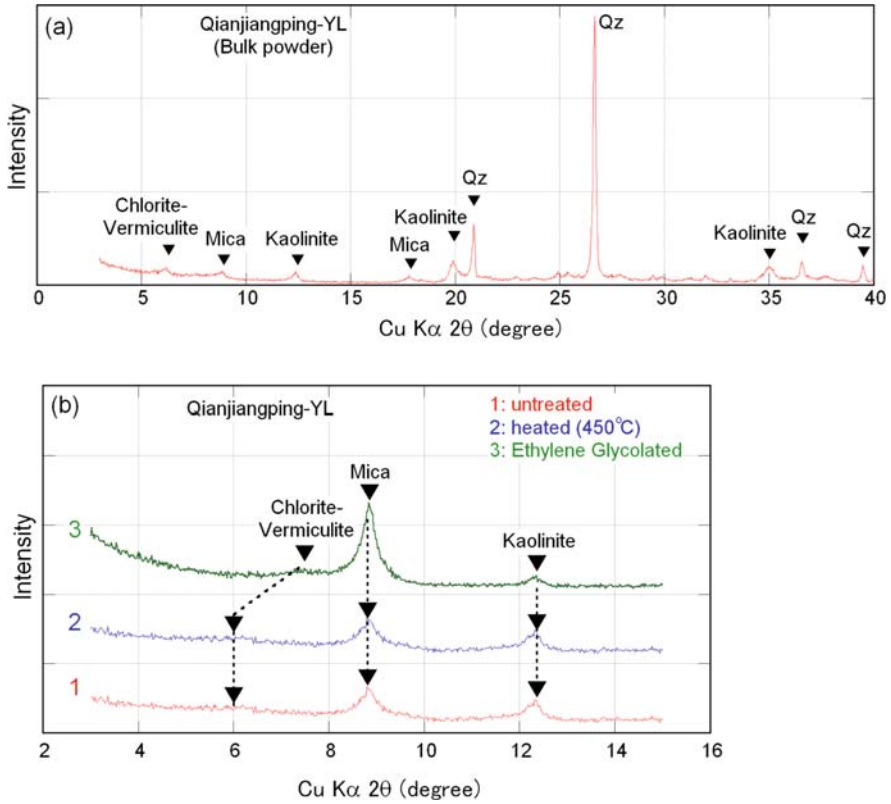


Fig. 8.17 X-ray diffraction pattern for the yellow clay sample (YL sample)

Ring Shear Tests to Simulate the Sliding Process

Ring shear apparatus DPRI-5 was employed to simulate the sliding process of the Qianjiangping landslide. In the 1980s, a series of ring shear apparatus was developed in the Disaster Prevention Research Institute (DPRI), Kyoto University, Japan (Sassa 1994; Sassa et al. 2003). The ring shear apparatus DPRI-5 is one of the series with an outer ring that is 180 mm in diameter and an inner ring that is 120 mm in diameter. Shear torque tests and shear velocity tests can be conducted with the DPRI-5. It is also possible to conduct undrained shearing tests to simulate rapid sliding in undrained (water-saturated) materials.

In this study, two series of ring shear tests were conducted. One series uses shear torque tests to simulate the landslide sliding process and the other uses shear velocity tests to examine the effect of shear velocity in mobilizing the shear resistance.

Shear Torque Test to Simulate the Sliding Process

The Qianjiangping landslide has a slope angle of 24° in the exposed sliding surface at main scarp and an average thickness of 26 m. These parameters were used to determine the initial normal stress acting on the soil element in the sliding zone (assuming the unit weight of the sliding mass is 18 kN/m^3). To simulate the sliding process in the sliding zone after failure, the test was designed as given below.

First, consolidate the sample normally with the initial normal stress and then apply part of the initial shear stress gradually under drained conditions. Thereafter, change the shear box to the undrained condition and increase the shear stress until failure occurs. Through this process, the after-failure processes of a rapid landslide can be simulated.

The tests were conducted on the BK sample and YL sample with the following procedure:

- (1) Set the oven-dried soil sample into the shear box using the air deposition method.
- (2) Saturate the sample with CO_2 gas and deaired water.
- (3) Consolidate the fully saturated sample with the initial normal stress of 388 kPa.
- (4) Apply the shear stress of 141 kPa at 1.2 kPa/min to avoid excess pore water pressure generation under natural drained conditions with the drainage valve open.
- (5) Increase the shear stress further to make the soil specimen fail under undrained conditions at the rate of 0.28 kPa/min and measure the shear resistance mobilized during the shearing process.

The test results for the BK sample and the YL sample are shown in Figs. 8.18 and 8.19; they indicate that the two samples behaved differently. For the BK sample, shear resistance decreased quickly when the sample failed at a low residual shear resistance. Meanwhile, for the YL sample, shear resistance showed only a very small decrease and recovered quickly to the post-failure value.

The mobilized friction angles of the BK sample and the YL sample, which are defined as the inverse tangential function of the ratio of mobilized shear resistance to the total normal stress, are shown in Fig. 8.20. For the YL sample, after failure, the mobilized friction angle stayed at about 20° . Using the 24° slope angle of the sliding plane, a factor of safety (FoS) of 0.82 is obtained, which indicates that the slope will move along the yellow layer. For the BK sample, the mobilized friction angle was 26.5° at first, showing a greater shear resistance to the sliding when the slope angle was 24° . Because the black layer is between the rigid sliding block and the soft yellow layer (see Fig. 8.14), shear displacement would be caused by the sliding that occurred along the yellow layer. After the shear displacement reached about 2,000 mm, the mobilized friction angle started to decrease; and when the shear displacement reached 2,600 mm, the mobilized friction angle decreased suddenly from 25 to 12° . Considering that the slope angle of the energy line of the Qianjiangping

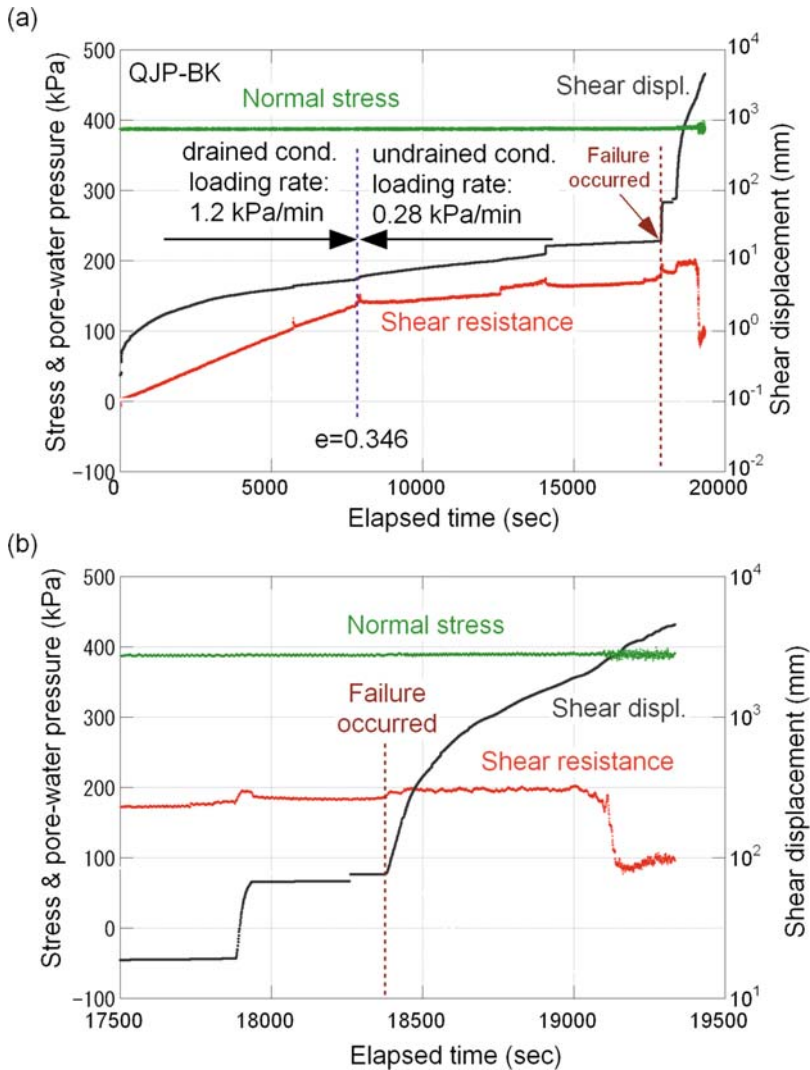


Fig. 8.18 Shear torque ring test results for the black silt sample (BK sample) from the Qianjiangping landslide. (a) Time series data for the whole test. (b) Close-up of the data near failure

landslide is about 10° , the mobilized 12° friction angle suggests the main sliding may have occurred along the black silt layer. The decrease of the mobilized friction angle may be due to excess pore pressure generation caused by grain crushing of the black silt layer during the shearing. This phenomenon was comprehensively studied by Sassa (Sassa et al. 1996, 2004) and his colleagues (Wang et al. 2000, 2002; Fukuoka et al. 2004) with undrained ring shear tests on sandy soils. Through this study of the Qianjiangping landslide, it is reasonable to estimate that excess

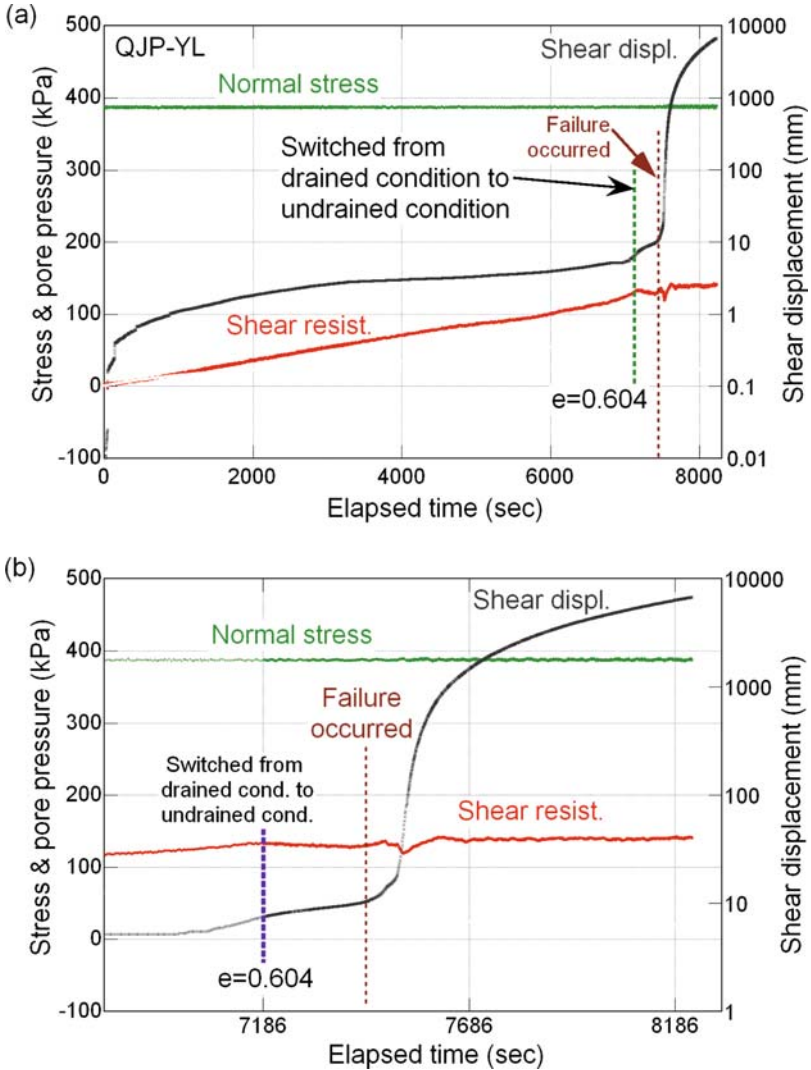


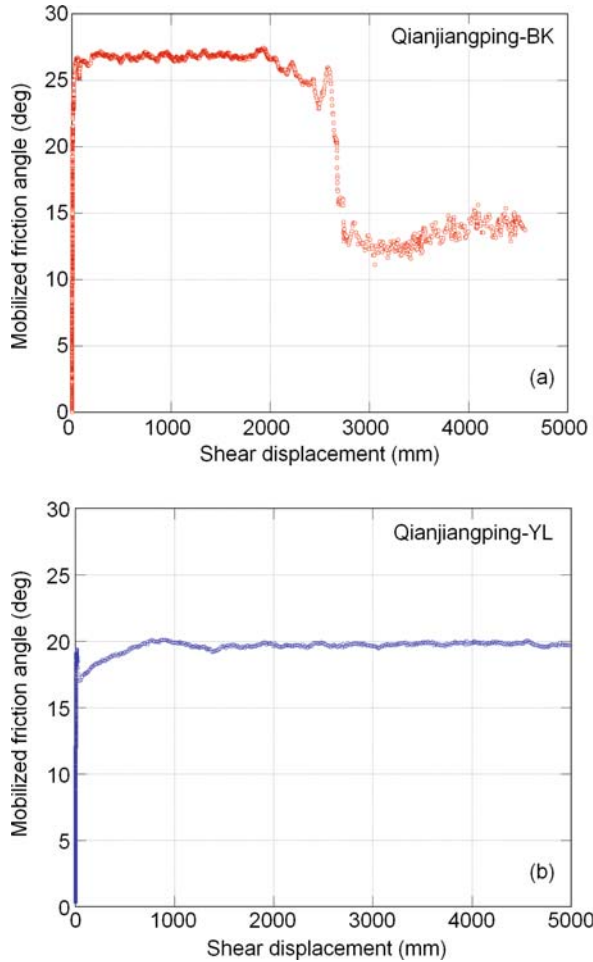
Fig. 8.19 Shear torque ring test results for the yellow clay sample (YL sample) from the Qianjiangping landslide. (a) Time series data for the whole test. (b) Close-up of the data near failure

pore pressure generation was caused by grain crushing of the black silt layer, which eventually will become a yellow clay layer.

Shear Velocity-Controlling Test to Simulate the Sliding Process

The purpose of this test series is to investigate the rate effect on the shear resistance. Rate effect is a phenomenon where the shear resistance of some soils becomes

Fig. 8.20 Mobilized friction angle from the creep simulation test for the black silt sample (a) and the yellow clay sample (b) from the Qianjiangping landslide

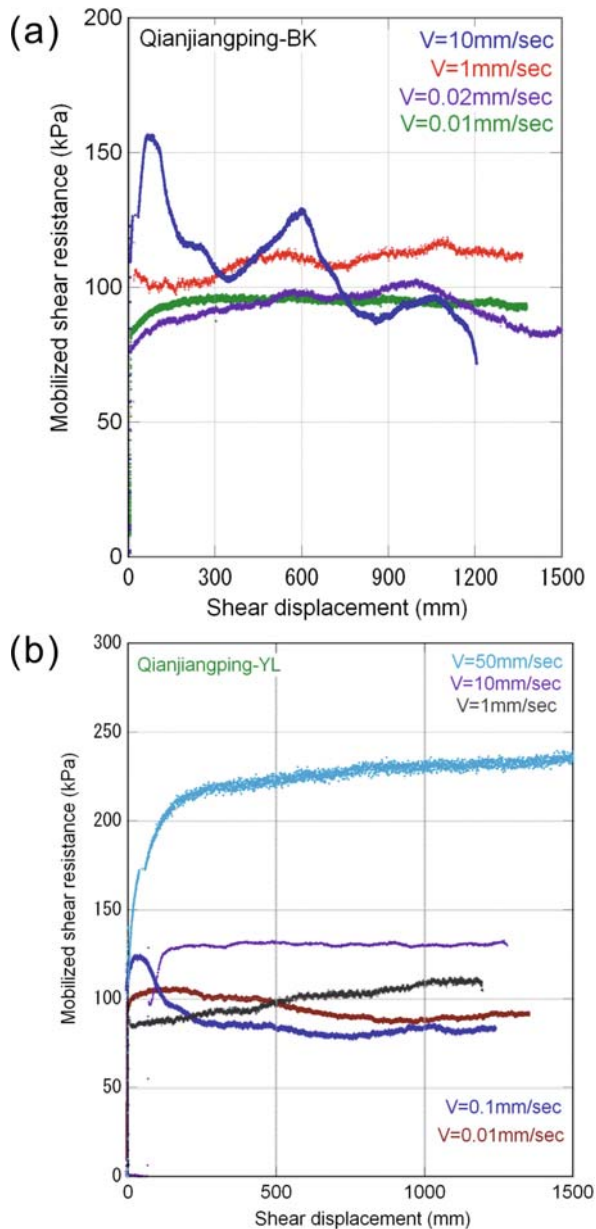


smaller when the shear velocity becomes higher; while for other soils, their shear resistance becomes larger or remains constant (Tika et al. 1996; Saito et al. 2007). In this series, tests were conducted using the following procedure:

- (1) Prepare the saturated sample with the same procedure as described previously.
- (2) Consolidate the sample at 200 kPa.
- (3) Shear the sample with different shear velocities under the undrained condition and measure the shear resistance.

Figure 8.21 shows the relationship of the mobilized shear resistance with shear displacement of the two samples at different shear velocities. It is clear that for the BK sample, the shear resistance does not depend on the shear velocity when the shearing velocity is small (Fig. 8.21a). At shear velocities of 0.01 and 1 mm/s, the shear resistance did not show any obvious change, whereas at a shear velocity of

Fig. 8.21 The relationship between mobilized shear resistance and shear displacement at different shear velocities in the shear velocity ring test. Results for the black silt sample (a) and the yellow clay sample (b) from the Qianjiangping landslide



10 mm/s, the shear resistance became smaller than those produced at lower velocities when the shear displacement exceeded 750 mm. This result would mean that for the BK sample, shear resistance became lower when movement occurred at high velocity, while, for the YL sample, the mobilized shear resistance became higher at higher shear velocity (Fig. 8.21b).

Fig. 8.22 The relationship between mobilized shear resistance and shear velocity for the black silt sample (a) and the yellow clay sample (b) from the Qianjiangping landslide

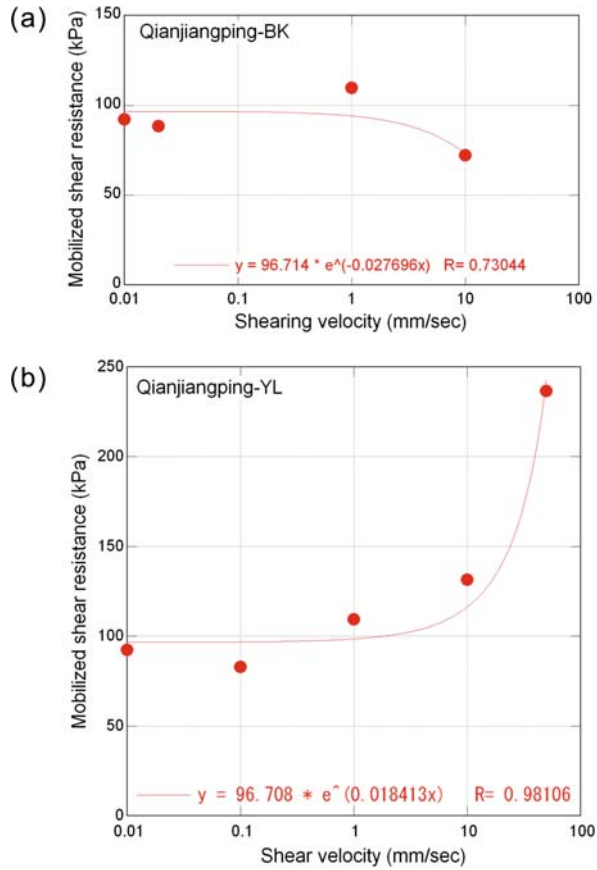


Figure 8.22 was obtained by plotting the mobilized shear resistance with shearing velocity for the two samples. Through comparison of the results, differences in the change of mobilized shear resistance with shear velocity can be examined. For the BK sample, the shear resistance became slightly smaller when the shear velocity increased, while for the YL sample, the shear resistance became much greater when the shear velocity increased. For the YL sample, the shear resistance shows a positive relationship with the shear velocity. When the shear velocity becomes higher, the shear resistance becomes higher.

Discussion and Conclusions

A detailed field investigation was made of the Qianjiangping landslide that occurred after the first impoundment of the Three Gorges Reservoir. The mechanisms of the landslide, especially the factors affecting slope failure, were studied, and reasons for the rapid and long distance movement of the landslide were considered. Based on fieldwork and analyses, the following conclusions were reached:

- (1) The Qianjiangping landslide is a reactivated landslide with dip structure. The sliding surface was along a pre-existing structural plane of weakness (a bedding-plane shear).
- (2) Quarrying of mudstone for brick manufacture from the toe of the slope and intense rainfall before the landslide put the slope in a critical state. This is not to assign blame for the landslide on the operators of the brick factory. Although quarrying of rock from the toe of a landslide is not good practice, there was no information to indicate that it was bad practice until the landslide was recognized. Prior to the cracks appearing, there appears to have been no reason to expect that the slope was unstable and hence little reason to exercise caution.
- (3) The high water level in the Qinggan-he River through impoundment of the reservoir was the trigger for the landslide occurrence.
- (4) The mechanism of the Qianjiangping landslide can be interpreted from the two series of ring shear tests. The motion of this landslide could be controlled by the black layer and the yellow layer. During the first phase of landsliding, the sliding mass should move along the yellow layer. Because the black layer lies on the yellow layer, shearing deformation should also occur in the black layer. After shearing proceeded for a large distance and the shear velocity reached a certain value, the main sliding plane shifted to the black layer because the shear resistance in the yellow layer became greater than that in the black layer. With the sudden decrease of the shear resistance in the black layer, rapid motion occurred in this reactivated landslide.
- (5) Because the landslide has a tectonic background, and there are many similar slopes in the nearby area, detailed evaluations for those slopes are very important for the future stability of the Three Gorges Reservoir when it is raised to its final operating level.

This research has very important significance for landslide disaster prevention and mitigation. Although reactivated landslides generally move with low sliding velocity, they can shift to rapid motion when the soil near the existing sliding plane has the tendency toward a negative rate effect. When landslides have moved for a certain distance, the decrease in shear resistance of the newly developed sliding plane could make the landslide move rapidly.

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