ABSTRACT

On February 9 2010, the landslide dam formed in the Kashmir earthquake that occurred on Oct 8 2005 at Hattian Bala in Pakistan was breached after incessant rains. The authors had been involved in a research project to monitor the long-lasting change of the landslide mass at regular 6 monthly intervals since June 2008, and they noticed that air-exposed pieces of sandstones and mudstones of the landslide mass had disintegrated and crumbled due to slaking that dated back to the breach. The change in the landslide mass shape observed between June 2008 and November 2009, did not seem so significant except for a 300 m-long gulley that appeared all of a sudden at the toe of the mass during winter time from 2008 to 2009. Displacements from GPS-measurements conducted in June and November 2009 showed that the crest part subsided by about 10 cm while the toe part heaved slightly up where the overflown water fell into the eroded gully. A field survey was conducted over the breached landslide dam in April 2010, two months after the breaching event. A severely eroded breach channel was observed along the spillway, which was excavated immediately after the formation of the dam. Given the chronological change in precipitation of the catchment area of Hattian Bala obtained from the TRMM (Tropical Rainfall Measuring Mission) satellite data, the dam is considered to have been breached due to the overtopping of water over the landslide mass of slakable nature. The slakable nature of the material is discussed through both standard slaking tests and advanced unconventional direct shear tests on prepared specimens. Significant creep deformation and a reduction in their peak strength were observed as the slaking developed in the specimens, suggesting that the slakable nature of the mudstones might have been responsible for the breach of the landslide dam.

Key words: direct shear test, GPS, Hattian Bala, Kashmir Earthquake, landslide dam, slaking (IGC: B3)

INTRODUCTION

On October 8 2005 an earthquake of magnitude 7.6 Mw (USGS, 2005) hit Kashmir and other northern areas of Pakistan. Figure 1 illustrates seismo-tectonic map of Northern area of Pakistan. The epicenter marked in Fig. 1 was located at about 30 km north-west of Muzaffarabad, the capital of Azad Jammu and Kashmir State (AJK hereafter), and its focal depth was about 26 km. It is reportedly estimated that the earthquake caused respectively 87,350 and 10,300 fatalities in Pakistani- and Indian-controlled parts of the former princely state of Jammu and Kashmir (Hussain et al., 2006), and it is to be noted that the death toll associated with landslides reached about 26,500 (Petley et al., 2006). The biggest landslide mass in this earthquake, located in Hattian Bala, AJK, is shown in Fig. 1, buried more than 500 people. Figure 2 is a schematic map of Hattian Bala landslide dam and its vicinity, and Photo 1 is the satellite imagery from QUICKBIRD taken on Oct. 27 2005, a few days after the earthquake. The broken line and solid line show the perimeters of the landslide source area and the landslide debris deposit, respectively. The shooting locations for site photographs that will appear in this paper are also marked on Photo 1. The landslide debris formed a natural dam of about 85 million m³ volume and blocked the streams of both Karli and Tung valleys. These two lakes, created by the waters impounded by the natural damming, had been threatening downstream areas with a possible flood risk.

On February 9 2010, the Hattian Bala landslide dam was breached. The discharged water reportedly inundated low-lying areas along Karli and Jehlum rivers, destroying two dozen houses and killing one child (The News International, 2010). Sattar et al. (2010), who conducted a dam break analysis before this tragic event, reported...
that although the Karli river and the Jhelum river would have the sufficient capacity to convey the major flood wave, infrastructures on low terraces may be affected by the flood wave, especially at the confluence of the two rivers.

As a part of a MEXT project (No. 20254003), routine field surveys have been conducted since June 2008, twice a year with the monsoonal season in between, to observe landform changes occurring over the landslide dam. The dam body was mainly composed of mudstones and sandstones which originally formed the “Dana hill” slope. These sedimentary rocks, especially mudstones, are particularly susceptible to slaking disintegration. Broad distinction between drying and wetting may have accelerated the disintegration of the rocks of the landslide dam.

The first half of this paper describes the geological settings for the areas surrounding Hattian Bala landslide dam and the observed landform changes before and after the breaching, and the second half deals with the description of the results of experiments carried out to explain the triggering mechanism for the dam breaching. This experimental approach consists of: i) a series of accelerated slaking tests (JGS 2125) to observe the effects of slaking on the mechanical properties of mudstones and sandstones retrieved from the site; ii) a series of direct shear tests on mudstone samples retrieved from the site, and on Chiba gravel samples (gravely soil accounting for sandstone), a material was chosen to draw comparisons.

**GEOLOGICAL SETTING AND FEATURES OF THE LANDSLIDE DAM**

The Kashmir earthquake (Mw 7.6) of October 8 2005 was caused by the rupture of the Muzaffarabad fault (MF) and the Jhelum thrust Tanda fault (JT-TF) (see Fig. 1). Muzaffarabad, Bagh and Balakot, major cities located along the activated faults, which are called Balakot-Bagh fault as a whole, were severely damaged. The Hattian Bala landslide occurred at the southern end of the Tanda fault about 33 km south-east of Muzaffarabad. The source area of the landslide is composed of Miocene Murree Formation mudstones and sandstones including minor proportion of limestone on the hanging wall of the Tanda fault (Dunning et al., 2007).

The main features of the Hattian Bala landslide dam and of the two lakes that appeared behind it are given in Table 1. Figure 3 shows the relationship between volumes of landslide dams and their impounded lakes around the world. According to this figure, the Hattian Bala landslide dam is among the largest landslide dams in the world. Since the formation of the landslide dam, Water and Power Development Authority of Pakistan had been monitoring water levels of the two lakes and their discharges, and they conducted a stability analysis of the dam (WAPDA report, 2006). Based on their
Table 1. Summary of information about the Hattian-Bala landslide dam and lakes

<table>
<thead>
<tr>
<th>Characteristics of landslide dam and lake</th>
<th>Data (assessed and inferred)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Location and date of dam formation</td>
<td>Hattian-Bala, 8 Oct 2005 (34°08'N, 73°43'E)</td>
</tr>
<tr>
<td>2. Trigger or cause of landslide</td>
<td>8th Oct, 2005 M = 7.6, 44 km from epicenter</td>
</tr>
<tr>
<td>3. Type and characteristics of landslide forming dam</td>
<td>Rock and debris avalanche</td>
</tr>
<tr>
<td>a) Landslide volume</td>
<td>80.78 million m³</td>
</tr>
<tr>
<td>b) Landslide scar altitude</td>
<td>2040–1300 m</td>
</tr>
<tr>
<td>c) Max. debris thickness above valley bottom</td>
<td>250 m</td>
</tr>
<tr>
<td>d) Landslide surface area</td>
<td>1.02 km²</td>
</tr>
<tr>
<td>e) Source maximum length up to deposited surface</td>
<td>1750 m</td>
</tr>
<tr>
<td>f) Source maximum width</td>
<td>520 m</td>
</tr>
<tr>
<td>g) Morphology (Costa and Schuster, 1988)</td>
<td>Type III</td>
</tr>
<tr>
<td>4. Rock type</td>
<td>Miocene Murree formation Mudstones</td>
</tr>
<tr>
<td>5. Underlying causes of landslide</td>
<td>Seismically Reactivated landslide</td>
</tr>
<tr>
<td>6. Landslide dam</td>
<td></td>
</tr>
<tr>
<td>a) Height of landslide dam in front of spillway</td>
<td>130 m</td>
</tr>
<tr>
<td>b) Width of landslide dam at crest</td>
<td>700 m</td>
</tr>
<tr>
<td>c) Base length of landslide dam along spillway</td>
<td>1330 m</td>
</tr>
<tr>
<td>d) Slope of dam faces</td>
<td>Downstream 20–26°/ Flat near crest</td>
</tr>
<tr>
<td>7. Karli Lake (Large lake)</td>
<td></td>
</tr>
<tr>
<td>a) Volume</td>
<td>62 million m³</td>
</tr>
<tr>
<td>b) Catchment area</td>
<td>44.17 km²</td>
</tr>
<tr>
<td>c) Maximum altitude</td>
<td>2497 m</td>
</tr>
<tr>
<td>d) Minimum altitude</td>
<td>1237 m</td>
</tr>
<tr>
<td>8. Tang Lake (Small lake)</td>
<td></td>
</tr>
<tr>
<td>a) Volume</td>
<td>5 million m³</td>
</tr>
<tr>
<td>b) Catchment area</td>
<td>30.10 km²</td>
</tr>
<tr>
<td>c) Maximum altitude</td>
<td>2884 m</td>
</tr>
<tr>
<td>d) Minimum altitude</td>
<td>1149 m</td>
</tr>
</tbody>
</table>

Fig. 3. Location of Hattian Bala landslide dam on bivariate plot of impounded lake volume versus landslide dam volume (after Korup, 2004)

BREACHING FAILURE OF LANDSLIDE DAM results, Pakistan Army excavated a spillway across the crest of the dam in order to lower the maximum water level of Karli Lake (large lake) and thus to reduce the risk of flood in case of dam breaching (see Photo 1). The water level of Karli Lake reached the spillway at the end of March 2007, and from the first days of April, the water started flowing gently along the spillway (Schneider, 2009).

Ermini and Casagli (2003) proposed a Dimensionless Blockage Index, DBI, to estimate the stability of a landslide dam, as follows;

\[
DBI = \log \left( A \times \frac{H}{V} \right)
\]

where \( A \) is the catchment area upstream the blockage (in km²), \( H \) is the maximum crest height of landslide dam (in m) and \( V \) is the volume of landslide dam (in million m³). It is considered that landslide dams with DBIs smaller than 2.92 hardly collapse, while those with DBIs greater than 3.25 are susceptible to breaching. From Table 1, the DBI for Hattian Bala landslide dam is estimated to be 1.85, allowing experts to have optimistic views that the dam would not pose an imminent threat to people living in its downstream reaches.

Pakistan has four distinct seasons; a dry cool winter (from December through February); a dry hot spring (from March through May); the early summer monsoon (from June through September); and the winter monsoon (October and November). Half of the annual rainfall is usually concentrated in July and August with an average monthly rainfall reaching 255 mm (Blood, 1994), suggesting that the risk of dam breaching can be high during this period of year. However, the dam was breached on February 9 2010 during the dry winter.

FIELD SURVEYS

Before Breaching

(1) Field observations

The greater part of Hattian Bala landslide dam is made up of both sandstones and mudstones of Murree formation which can be easily disintegrated because of their weak and slakable nature (see Photo 2). These geomaterials seemed to have the same distribution quantity on the surface of the dam. Figure 4 shows particle size distributions of the surface soils (mixtures of sandstones and mudstones) which were measured by grid by number analysis (Casagli et al., 2003). The particle size of the surface soils seems to have a wide distribution (\( D_{max} = 525 \sim 4,000 \) mm, \( D_{50} = 4.4 \sim 430 \) mm with fines content of 0 ~ 25%).

The pairs of Photos 3 to 7 compare views of the landslide dam at different times (June 2008 and November 2009). The photo-shooting locations are marked on Photo 1. Photo 3 shows the water flowing out from the Karli Lake (large lake) along the spillway excavated across the crest. Photos 4 and 5 show the middle portions of the landslide dam. The spillway and the dam surface did not show any clear sign of change during this one- and-half year period. Photo 6 shows the debris source area and the spillway at the toe part of this source area. Since there were few changes in the slope of the source
area, it can be assumed that there was limited discharge of debris to the dam body.

No significant change in the shape of the dam was observed between June 2008 and November 2009, except for the erosion that appeared at the toe part of the dam (Photo 7). This erosion was first observed in the survey carried out in June 2009, although no sign of this erosion had been noticed in the previous survey (November 2008). According to an eye-witness, a rainfall event in winter caused this erosion (the exact date of this event is not known). Since the landslide dam was also breached after incessant rains in winter of the following year, the above erosion may have been an early sign of the breaching in 2010. The details about relationship between the breaching and precipitation will be discussed later.

(2) GPS measurements

This huge landslide dam was considered to be gradually deformed. Therefore, the exact locations were measured at points marked over the landslide dam in June and November 2009 using a Differential Global Positioning System (DGPS). Observation points were arranged along two lines, one along the spillway (longitudinal line) and the other in its transverse direction (transverse line). Figures 5 and 6 show respectively in-plane and vertical displacement vectors along longitudinal and transverse lines.

The perimeter of the gully erosion at the toe part of the dam shown on Photo 7 was also traversed using a GPS
Fig. 5. Deformation vectors obtained by comparing DGPS data for June 2009 and November 2009 surveys. Perimeter of erosion channel observed in June 2009 and November 2009 survey is marked near the toe of debris mass (after Sattar et al., 2010)

Fig. 6. Settlements obtained by comparing DGPS data for June 2009 and November 2009 surveys. Erosion and breach perimeters are marked on the map (after Sattar et al., 2010)

receiver in June and November 2009. Both perimeters traversed at different times of the year were nearly identical to each other (Figs. 5 and 6), and showed little sign of additional erosion during the five months.

In Fig. 5, the displacements are smaller than or at most equal to 5 cm, and do not show any clear orientation, indicating that they are all local and do not show any sign of mass movement. Figure 6 shows that the maximum settlement of about 10 cm is reached near the east end of the transverse line, where it is presumed that the dynamically placed debris mass is the thickest. On the other hand, a slight uplift was observed where the spillway water falls in the eroded gully near the toe of the dam.

After Breaching
(1) Landform changes and affected areas
A field survey was conducted from April 11 to 14 2010, roughly two months after the breaching occurred. Figure 7 shows a contour map of the landslide dam before and after breaching, and the severely breach channel can be seen along the spillway.

The pairs of Photos 8 to 15 compare views of the landslide dam before and after the breaching. The photo-shooting locations are shown on Photo 1. Points were
marked along the perimeter of the scraped gorge with handy GPS receivers and plotted on Figs. 6 and 7 (see open triangles and red lines, respectively).

In Photo 9, showing the view from the debris source area, there is a steep-sided gorge carved deeply by the breaching event, while the remaining debris deposit behind it shows little sign of deformation with its surface covered with thin grass. The side wall was the highest, reaching 80 m and dipping 50 degrees at Point A in Photo 9. Although the surface of the dam was covered with both sandstones and mudstones of large particles, the dam body was found to be comprised mainly of relatively small mudstones that can be seen along the sides of the exposed breach channel. Photo 10 shows the view from the depositional area. The lower end of the debris mass remaining on the naked slip surface has been deeply eroded. A deeply scoured gorge (breached channel) can also be clearly seen in Photo 11, showing the middle part of the debris mass. Looking down the toe of the debris mass (Photo 12), it is noted that the river bed has been raised by about several tens of meters, while the toe slope has been deeply scoured. Photo 13 shows the frontal view of the toe as far as about 1 km away from the downstream side. A rock outcrop, observed in the deeply scoured gorge, may have prevented the gorge from being scoured deeper.

Photo 14 shows a village on a gentle grassy slope on the right side of Karli lake (large lake). It was this village which suffered from the coherent landslide with its 300 houses destroyed. According to eye-witnesses, the mass movement was remarkable a few hours after the breaching. Water marks remaining on the left bank of Karli lake show that the water level for the full capacity of the lake was lowered by about 45 m due to breaching. On the other hand, the water level in the Tang Lake (small lake) was slightly higher than that before the breaching (Photo 15).

This outburst of landslide dam has caused a mudflow along Karli valley (Photos 16 and 17). Though the mudflow reached several tens of meters above the original riverbed, most houses resting on river terraces narrowly escaped from being swept away (Photo 17). The debris deposit along Karli valley was roughly estimated to be about 40 m thick immediately below the toe of the dam, and it became less as we went down the valley. At the con-
BREACHING FAILURE OF LANDSLIDE DAM

Photo 16. Downstream area inundated by debris flow and flood (photographed toward upstream, Latitude 34.156567°, Longitude 73.744833°).

Photo 17. Surviving settlement in the Karli valley (photographed toward downstream, Latitude 34.156617°, Longitude 73.744833°).

Photo 18. Junction of Karli river and Jhelum river inundated by flood (photographed toward upstream, Latitude 34.168423°, Longitude 73.739982°).

Photo 19. Muzafarabad city area inundated by flood (photographed toward downstream, Latitude 34.348894°, Longitude 73.470177°).

The landslide dam was breached at 3:00 PST, before dawn, and according to local witnesses, a large sound of rock movements was heard 30 min before the breaching, and the people of the Hattian Bala town evacuated on their own initiative because they had been informed of the possible risk of a breach. It is regrettable that one child was killed in this event.

(2) Precipitation record

The precipitation record of the catchment area of the Karli River, obtained from the Tropical Rainfall Measuring Mission (TRMM) satellite is shown in Fig. 8. The TRMM is a cooperative mission between JAXA and NASA, which estimates the rainfalls over tropical to subtropical regions with 0.25° × 0.25° resolution. Data are open to public via their website.

Significant precipitations of about 50–60 mm/day were recorded during the monsoon season every year since April 2007 when Karli Lake was filled up. However, the dam was breached in February 2010 during the dry winter. A peak precipitation of about 30 mm/day can be observed on February 9 2010, the day of the breaching. Although the water level of Karli Lake immediately before the breaching is not known, it is expected to be higher than that observed in November 2009 (see Photo 3 (right)). Figure 8 indicates that the accumulated precipitation between November 2009 and the day of the breaching was lower than that before the survey in June 2008. Therefore, it can be considered that the water level of the lake was not higher than that shown in Photo 3 (left).
In addition to the overflow surcharge, it is likely that this unseasonal rain has also accelerated the slaking of the dam materials, especially mudstones. Soil dunes formed over the dam body shown in Photo 2 suggest that the mudstones were highly susceptible to deterioration by slaking. Figure 8 also shows that precipitations were extremely low during the months before the breaching day. Since the slaking phenomenon is caused by drying and wetting cycles, sudden rain after this drought period might have caused the severe slaking of the mudstones.

LABORATORY TESTS

The performed laboratory tests comprise a series of accelerated slaking tests on sandstone and mudstone specimens to identify their standard slaking classes, and a series of unconventional direct shear tests with an advanced direct shear apparatus in order to examine stress-deformation characteristics of geomaterials which experienced the accelerated slaking process.

Accelerated Rock Slaking Test

(1) Test procedure and materials

Accelerated rock slaking tests on crushed sandstone and mudstone samples retrieved from Hattian Bala landslide dam were conducted, in accordance with guideline No. JGS 2125–2006 edited by the Japanese Geotechnical Society.

Each specimen, approximately 50 cm³ in volume, was first oven-dried at a temperature of 40°C for 24 hours, and then subsequently immersed in distilled water for 24 hours. In the immersion process, distilled water is to be poured into a 20 cm-diameter container until the specimen is fully immersed (approximately within 1 minute). This drying-wetting cycle was repeated three times.

To estimate the changes in shapes and colors of beddings and/or laminae, each specimen was photographed: before the immersion, within 1 minute (t = 0 min here-after), at 30 min., 1 hour, 2 hours, 6 hours and 24 hours after the start of pouring. The slaking class of the specimen was then identified based on the illustrated features of deterioration (Table 2).

(2) Test result

Figure 9 shows the identified slaking classes of the sandstone and the mudstone specimens with respect to time during the first drying-wetting cycle. Photos 20 and 21 show the overall changes of the sandstone and the mudstone specimens before and after completing the tests (i.e. after the three drying-wetting cycles).

Sandstone specimens showed little sign of slaking (estimated slaking class = 0), while small cracks and small

---

Table 2. Definition of slaking classes (JGS 2125–2006)

<table>
<thead>
<tr>
<th>Class</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>There is no change</td>
<td>Original shape remains with a few cracks</td>
<td>Many cracks appear, specimen is divided into some fragments, and original shape can be recognized.</td>
<td>Whole body has crumbled, however, not muddy. Original shape cannot be recognized.</td>
<td>Whole body is muddy.</td>
</tr>
<tr>
<td>B</td>
<td>There is no change</td>
<td>Original shape remains with a few cracks or with light stratigraphical disintegration.</td>
<td>Circumference has crumbled and it is difficult to recognize original shape.</td>
<td>Circumference has crumbled completely and separated into many particles. Original shape cannot be recognized.</td>
<td>Whole body is sandy.</td>
</tr>
</tbody>
</table>

A: Typical states for mudstone of fine-grained tuff
B: Typical states for siltstone, sandstone or coarse-grained tuff
Photo 20. Sandstone samples before and after accelerated slaking test

Photo 21. Mudstone samples before and after accelerated slaking test

bubbles appeared on surfaces of the mudstone specimens 30 minutes after the first pouring. The mudstone specimens, however, did not exhibit any further changes during the subsequent wetting/drying cycles, and their shapes were kept intact (estimated slaking class = 1).

Direct Shear Test

(1) Test procedure and materials

Samples subjected to direct shear tests include the mudstones retrieved from the Hattian Bala landslide dam and a gravely soil consisting of crushed sandstones from a quarry in Japan (so-called Chiba-gravel). Since the in-situ sandstones were too large to be used in the direct shear tests and too solid to be crushed by a standard hammer, we used the Chiba-gravel as a substitute for the crushed in-situ sandstones with a smaller particle size. Even though they are categorized as the same sandstone, their detailed mineral composition might differ. However, it was confirmed that the Chiba gravel had the same slaking class (=0) as the in-situ sandstone.

The particle size of mudstone and Chiba gravel in direct shear tests was adjusted to the same size in order to simplify the testing condition and to focus on the effect of slaking of different geomaterials on the mechanical properties. Both specimens of mudstone and Chiba gravel with sub-angular particles were prepared by removing particles finer than 2 mm and larger than 4.75 mm as a necessary adjustment to the apparatus dimensions, then oven-dried at a temperature of 100°C. The specimens were not thoroughly compacted to prevent particle breakage and to realize similar conditions as those of the landslide dam, which was dynamically deposited without strong compaction. The specimen densities are shown in Table 3. In this report, relative density of the specimens was not evaluated. Since the amount of in-situ mudstone sample was limited, the test for minimum density (JGS0162–2009) was not carried out since it could cause particle breakage due to compaction.

A schematic view of the advanced direct shear apparatus used in this study is shown in Fig. 10. The inside specimen size is 12 cm × 12 cm × 12 cm. The apparatus has the following essential features: 1) a feedback control of the normal stress (applied by two air pistons) to accurately keep the upper shear box at level; 2) a possible feedback control on both normal loads and shear load to impose any prescribed stress path in the shear stress-normal stress space; 3) a lower shear box moving on a very low-friction rail, yet with two friction load cells to evaluate any friction at the bottom of the lower shear box; 4) shear displacements between the upper and lower boxes directly measured with a non contact laser transducer; 5) shear load applied by a high precision gear loading device driven by a servo-motor, allowing easy and exact control of arbitrary shear displacement ratio as well as sustained loading (by a feedback control on the shear load). The importance of all these features to obtain reliable data of direct shear tests on granular materials was demonstrated by Shibuya et al. (1997), Qiu et al. (2000) and Wu et al. (2008). More details of the apparatus are given by Duttine et al. (2008, 2009).

The initial opening between the two boxes was set at 2 cm. Other initial test conditions are listed in Table 3. In this study, the loading process during the test consisted of three stages, as shown in Fig. 11: 1) the specimen was subjected to shear and average normal stresses keeping their ratio $R = \tau/\sigma_v$ set at either 0.5 or 0.8. Both $\tau$ and $\sigma_v$ were steadily increased up to respectively 30 kPa and 100 kPa for $R = 0.5$, and 80 kPa and 100 kPa for $R = 0.8$. $R$ value of 0.5 simply corresponds to the approximate maximum slope angle (25 to 30 degrees) of the Hattian Bala landslide dam while $R = 0.8$ was chosen for comparison. 2) After the prescribed $\tau$ and $\sigma_v$ values were
reached, these values were kept constant for six hours (i.e., creep loading). 3) A monotonic shear loading was applied at a constant shear deformation rate of 0.2 mm/min under constant normal stress condition until the specimen’s residual state was reached. The slaking effects were evaluated by applying to another sets of specimens the following alternative step: 2 bis) creep loading for total 6 hours including a saturation process 3 hours after the start of creep loading. The saturation process was carried out by pouring the distilled water from the bottom of the lower shear box until the specimen was fully immersed. As shown in Fig. 10, the over‰owed water is received from the opening between upper and lower shear boxes by an acrylic box that is ﬁxed on the lower box. This creep loading process represent a situation that slope ground is saturated by rain fall under constant stress condition.

(2) Test result

Figures 12 and 13 show the time histories of shear displacements for the Chiba gravel specimens (CG1, CG2) and for the mudstone specimens (MS1, MS2), respectively. The specimens of CG2 and MS2 were subjected to the alternative step loading history (including saturation) while the specimens CG1 and MS1 were subjected to the original loading history mentioned above. The time \( t = 0 \) corresponds to the start of creep loading. In the case of CG1 and MS1, the creep shear displacements converged on constant values after several tens of minutes.

On the other hand, the saturating process of the specimens CG2 and MS2 (3 hours after creep loading has started) caused a drastic increase of the shear displacements. This increase is particularly remarkable for the mudstone specimen (MS2), whose slaking class is higher than those of Chiba gravel and in-situ sandstone. The shear displacement of the mudstone specimen increased by about 3 mm, approximately seven times larger than the values for the Chiba gravel specimen (CG2). Figure 14 shows the creep test result for the mudstone specimen (MS3) consolidated at a stress ratio of \( R = 0.8 \). The slaking effects were remarkable in that shear displacement do not stop until failure.

Figures 15 and 16 show the relationship between the shear stress ratio, \( R \), and the shear displacement, \( s \), together with the associated volume change (i.e., the vertical displacement, \( d \)) for the specimens of CG1, CG2 and MS1, MS2 respectively. The values of \( s \) and \( d \) were set at zero at the end of step 2 or 2bis. In addition, the values of friction angles, \( \phi_d \), (assuming apparent cohesion \( c_d = 0 \)) at peak strength were also indicated in these figures.

---

### Table 3. Tested materials

<table>
<thead>
<tr>
<th>No.</th>
<th>Sample</th>
<th>Specimen condition during creep at ( R = 0.5 )</th>
<th>Stress ratio during creep, ( R )</th>
<th>Initial density before ML, ( \rho_i ) [g/cm³]</th>
<th>Density after ML, ( \rho_f ) [g/cm³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CG1</td>
<td>Chiba gravel</td>
<td>6 hr (dry)</td>
<td>0.5</td>
<td>1.393</td>
<td>1.450</td>
</tr>
<tr>
<td>CG2</td>
<td>Chiba gravel</td>
<td>3 hr (dry) and 3 hr (saturated)</td>
<td>0.5</td>
<td>1.417</td>
<td>1.434</td>
</tr>
<tr>
<td>MS1</td>
<td>Mudstone</td>
<td>6 hr (dry)</td>
<td>0.5</td>
<td>1.393</td>
<td>1.458</td>
</tr>
<tr>
<td>MS2</td>
<td>Mudstone</td>
<td>3 hr (dry) and 3 hr (saturated)</td>
<td>0.5</td>
<td>1.419</td>
<td>1.486</td>
</tr>
<tr>
<td>MS3</td>
<td>Mudstone</td>
<td>3 hr (dry) and saturated (failed)</td>
<td>0.8</td>
<td>1.419</td>
<td>—</td>
</tr>
</tbody>
</table>

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**Fig. 11. Stress-path during direct shear test**

**Fig. 12. Effect of slaking on creep deformation on Chiba gravel (as sandstone) with \( R = 0.5 \)**

**Fig. 13. Effect of slaking on creep deformation on mudstone with \( R = 0.5 \)**
Fig. 14. Effect of slaking on creep deformation on mudstone with $R = 0.8$

Fig. 15. Effect of slaking on stress-deformation characteristics on Chiba gravel (as sandstone)

Fig. 16. Effect of slaking on stress-deformation characteristics on mudstone

15 shows that the slaking effects on the $R$-$s$ and $d$-$s$ relations during subsequent monotonic loading are less significant for Chiba gravel, while in Fig. 16, the saturated mudstone specimen (MS2) exhibits largely different stress-displacement features from those for dry specimen (MS1). Neither the clear peak stress ratio nor post-peak stress softening appeared and the $R$-$s$ curve converged monotonously on the same residual state as dry specimen (MS1). Consequently, the stress level at which significant plastic (or irreversible) deformation develops (i.e., the start of yielding or yield stress) is different between these two specimens. The saturated specimen (MS2) continued to exhibit contractive behaviour during the shearing. Such stress-displacement characteristics are true of typical soft soils, indicating that the slaking process causes the examined mudstone material to disintegrate and therefore causes remarkable changes in its mechanical features.

DISCUSSIONS

About a half of the annual rainfall occurs during the monsoon season in Pakistan. Therefore, it was expected that the landslide dam would be breached during monsoonal times. However, the landslide dam was breached not in monsoon season but in February 2010 during off-monsoonal dry season. The significant erosion at the toe part of the landslide dam also appeared during dry winter time (between November 2008 and June 2009) and no clear landform change was observed during monsoon seasons (between June and November 2008, 2009). From the accelerated rock slaking tests performed on the mudstone and the sandstone specimens retrieved from the site, it can be concluded that the slaking level of the mudstone was slightly higher than that of the sandstone. The results of direct shear tests performed on the mudstone and the Chiba gravel specimens suggest that significant slaking-induced shear displacement can take place under constant shear and normal stress conditions, to a greater extent for the mudstone specimens than for the Chiba-gravel specimens, a substitute for the in-situ sandstones. The stress-displacement characteristics of the slaked mudstone were clearly weakened during subsequent monotonic shear loading.

All the above suggests that both unseasonal rains in dry winter seasons and slakable feature of the mudstone may have been responsible for both the significant erosion that occurred in winter from 2008 to 2009 and the breaching of the landslide dam in February 2010. Panabokke and Quirk (1957) reported that the slaking level of clay aggregates became higher as the initial water content of the specimen became lower. It is to be noted that the dam failed on February 9 because of rain after the most severe drought which followed the 2005 earthquake (see Fig. 8). In fact, the accumulated precipitation from October 2009 to January 2010 represents only 27% of the average accumulated precipitation during the same periods, computed between the 2005 earthquake and the breaching. Therefore, it can be inferred that the significant slaking of the mudstones due to rain fall in February 2010 after the long dry period caused a rapid reduction in the strength of the materials of the dam body, and/or accelerated the washout of fine substances of slaked mudstones by the spillway discharge. These processes are likely to have been responsible for the destabilization of Hattian Bala...
landslide dam. It is to be noted that there are still a noticeable number of unstable soil masses remaining with their toe soils all scoured deep. If one of these unstable debris masses slides and blocks the stream, it will again pose a threat to people living downstream. Further investigations in this respect are in progress.

CONCLUSION

On February 9 2010, a huge landslide dam at Hattian Bala, Azad Jammu and Kashmir State, Pakistan, comprising about 85 million m³, was breached by incessant rains. This landslide dam was formed by the 2005 Kashmir earthquake. Since June 2008, landform changes and potential settlements that were thought likely to affect the stability of the dam were monitored. The present paper reports the results of the regular field surveys as well as the associated laboratory tests. Attempts were made to explain the cause of the breaching of the landslide dam. The conclusions of this study are summarized as follows;

a) The Hattian Bala landslide dam consists of both crushed sandstones and mudstones of Murree formation. The mudstones were expected to be weathered easily because of their slakable features.

b) Erosion at the toe part of landslide dam was observed in June 2009 after dry winter while no clear landform change was noticed in the surveys of November 2008 and 2009 after monsoon seasons.

c) Based on the results of accelerated rock slaking tests, the slaking level of both the sandstone and mudstone was found to be relatively low. However, results from advanced direct shear tests confirmed the significant slaking-induced shear deformation of the mudstones under constant shear and normal stresses. Moreover, the stress-displacement characteristics of the slaked mudstone severely deteriorated during subsequent monotonic shear loading.

d) Considering the fact that the Hattian Bala landslide dam was breached by unseasonal heavy rainfall after a drought, the mudstones that mainly formed the landslide dam body likely suffered significant slaking. This phenomenon is likely to have been responsible for the destabilization of the landslide dam.

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