INTERPRETATION OF BREACHING FAILURE OF HATTIAN BALA LANDSLIDE DAM FORMED BY 2005 KASHMIR EARTHQUAKE WITH EXPERIMENTAL APPROACH

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ABSTRACT: On February 9, 2010, a natural landslide dam at Hattian Bala of Pakistan, which was formed in the Oct 8, 2005 Kashmir earthquake, failed due to incessant rains. The landslide dam was formed of mainly crushed mudstones and sandstones which can be easily deteriorated through their slaking process. Under this threat, we had made four surveys to monitor the landform changes at about half-year's regular interval since June 2008. The back erosion at the toe part of the dam was first observed in the third field survey of June 2009. GPS-measured displacements conducted in both June and November 2009 showed a settlement of the crest part and slight uplift near the toe where the overflowed water fell in the eroded gully. We performed slaking and direct shear tests on prepared samples of crushed mudstones and sandstones with some materials taken in-situ to understand slaking effects on their stress-deformation characteristics. A significant creep deformation and a reduction in the peak strength were observed as the slaking developed in the mudstone specimens that might be responsible for triggering the landslide dam failure.

Key Words: Kashmir earthquake, Landslide dam, Slaking, Direct shear test

INTRODUCTION

The October 8, 2005 Kashmir earthquake of magnitude 7.6 Mw (USGS) hit seriously Kashmir and other northern areas of Pakistan. The number of victims in the catastrophe amounts to 87,350 in Pakistan (Hussain et al., 2006). The earthquake triggered a huge landslide that buried more than 1,000 people in Parhore valley of Hattian Bala. The landslide debris formed natural dam comprising about 85 million m^3 and blocked the streams of the Karli and Tung valley as shown in Fig. 1 and Photo 1. Two lakes impounded by the dam had been threatening downstream villages with the possible flooding risk. With its scale and serious threat, the landslide dam has been attracting attentions of many researchers including Dunning et al. (2007), Owen et al. (2008) and Schneider (2009).

On 9th February 2010, the landslide dam failed after 5 days of rainfall as shown in Photo 2. The discharged water reportedly destroyed dozens of houses and killed one child, and the flood inundated several downstream areas (The News, 2010). Sattar et al. (2010) conducted dam break analysis before

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this event and reported that though the Karli and Jhelum rivers would be capable of conveying the major flood wave, infrastructures on low terraces could be affected by the flood wave.

Since the landslide dam appeared in the 2005 Kashmir earthquake, we had been paying attention to any possible changes that would affect the stability of the dam. The landslide dam was composed mainly of mudstones and sandstones which used to form the original slope. These sedimentary rocks, especially mudstones, are particularly susceptible to potential slaking due to seasonal climate change which can have caused the destabilization of the dam.

Since June 2008, we have been conducting field surveys twice a year to measure the landform changes of the dam. The effects of slaking on the mechanical properties of mudstones and sandstones retrieved from the site were examined by performing a series of drying-wetting tests, i.e. a series of so-called accelerated slaking tests. Meanwhile, direct shear tests on the mudstone retrieved from the site and on Chiba gravel (gravelly soil representing sandstone), were conducted in order to verify the slaking effects on their stress-deformation characteristics.

In this report, the landform changes of Hattian Bala landslide dam based on the field surveys between June 2008 and November 2009 are first presented, and then attempt is made to explain the mechanism of the failure of landslide dam through the experimental approach.

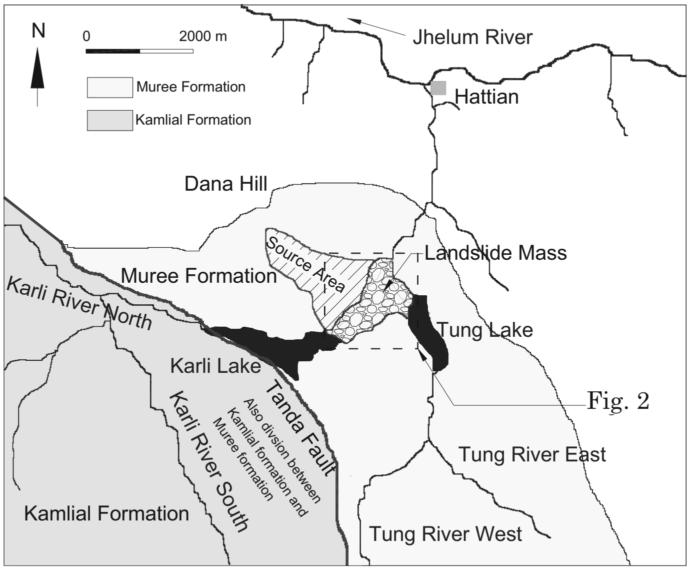


Figure 1 Schematic map of Hattian Bala landslide area. The volume of the landslide dam before collapse was estimated at 85 million m^3 and it impounded Karli lake of 61.7 million m^3 and Tang lake of 3.7 million m^3. (after Sattar et al., 2009)

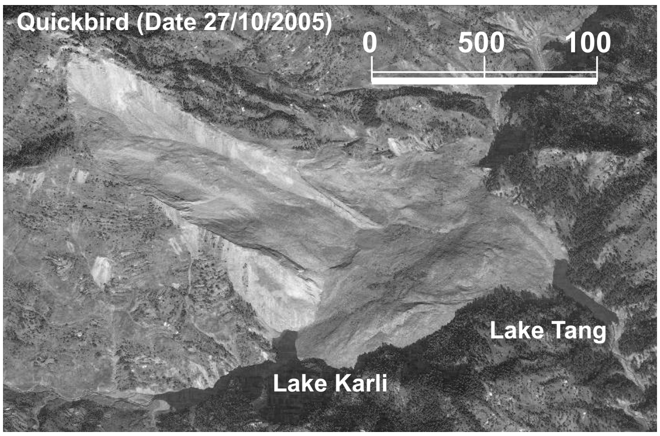
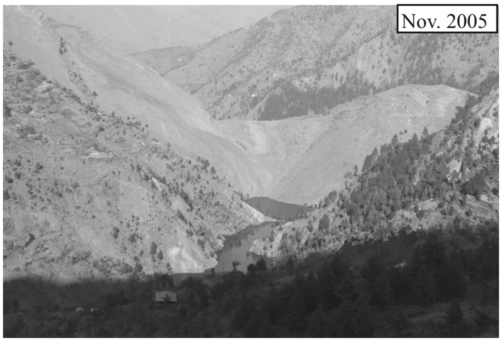


Photo 1 Overview of the Hattian Bala landslide dam.



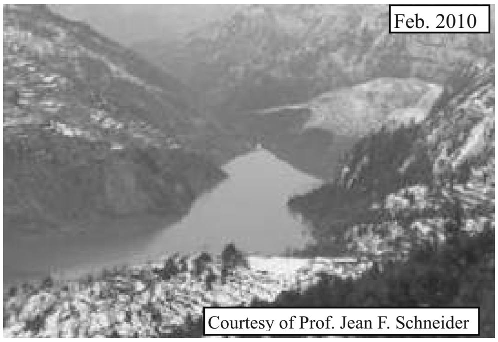


Photo 2 Hattian Bala landslide dam before and after failure (photography from upstream)

LANDFORM CHANGES AT HATTIAN BALA LANDSLIDE DAM

Field observations

The landslide dam formed by the 2005 Kashmir earthquake consists of both sandstones and mudstones from Murree formation. Deposited debris mass was composed of rocks that were expected to be weathered easily because of their weak and slakable features as shown in Photo 3.

Photos 4 to 8 compare scenic photos of landslide dam taken in June 2008 and November 2009. The landform changes of the dam seemed to be insignificant except for the backward erosion at the toe part of landslide mass first observed in June 2009.

Photo 3 Mudstone of Murree formation weathered by slaking around the middle portion of landslide dam (point C, c.f. Fig.2)

Photo 4 Dam water flow into the spillway at the crest part (point A, latitude 34.137885°, longitude 73.732342°, c.f. Fig. 2). The spillway seems to be unchanged after 2 years.

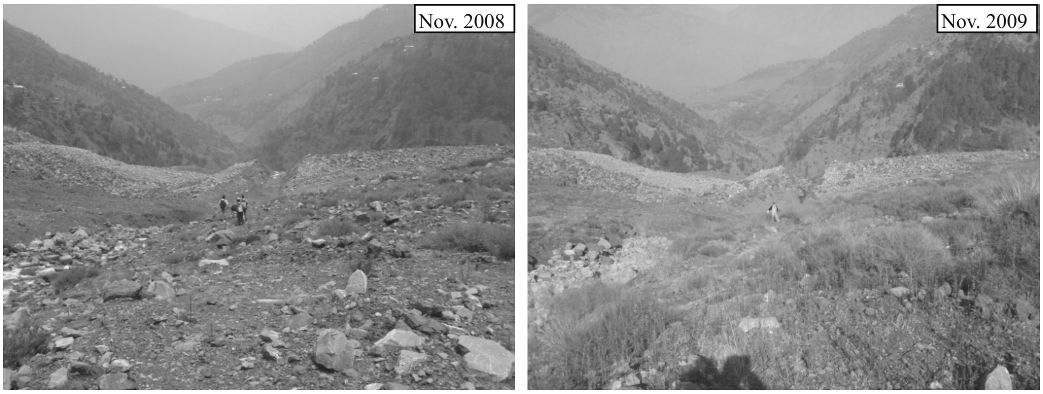


Photo 5 Middle portion of the dam (photography the downward from point B, latitude 34.138185°, longitude 73.730911°, c.f. Fig. 2)

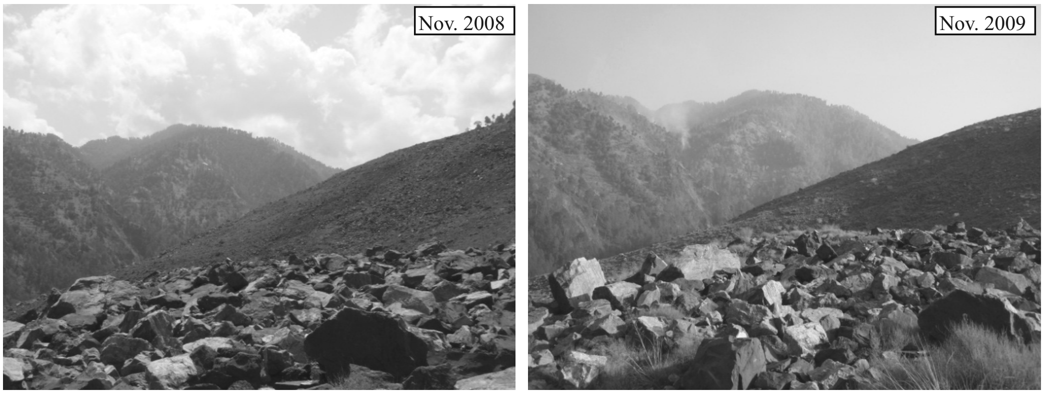


Photo 6 Middle portion of the dam (photography the westward from point B, latitude 34.138185°, longitude 73.730911°, c.f. Fig. 2). There was little change on the dam surface for 2 years)

Photo 7 Source area of landslide mass (photography from point C, latitude 34.138562°, longitude 73.728825°, c.f. Fig. 2). The spillway can be seen near side of the source area. There was little change on the slope for 2 years.

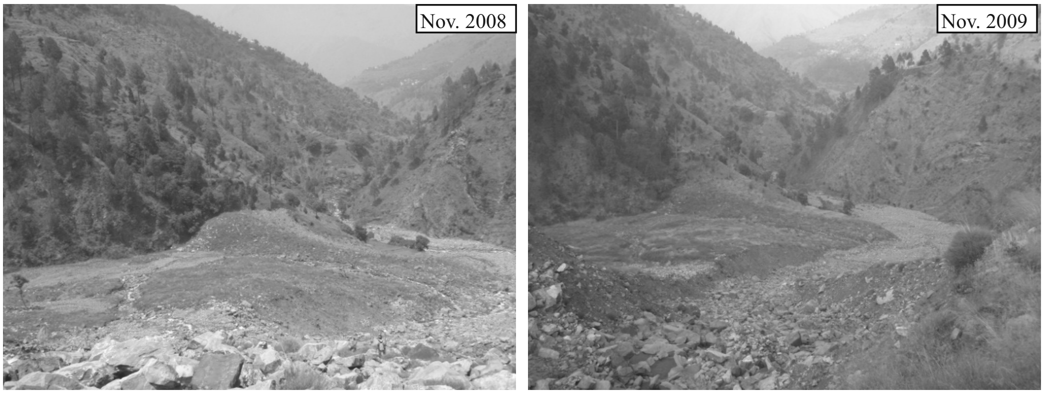


Photo 8 Toe part of the dam (photography the downstream from point D, Latitude 34.138564°, Longitude 73.725544°,c.f. Fig. 2). Significant erosion could be observed along spillway in the survey in June 2009 after that in November 2008.

GPS measurement

In order to investigate the deformations of the landslide dam in a quantitative mannar, we measured longitudes, latitudes and elevations of points marked on the landslide dam with dual-frequency Differential Global Positioning System (DGPS) in June and November 2009. In-plane displacement vectors of the points marked on both longitudinal and transverse lines are superimposed on the contour map of the landslide dam in Fig. 2, while the vertical displacement vectors are shown in Fig. 3. Note that the longitudinal line runs along the spillway while the transverse line extends from the toe of source hill to the eastern end of the debris mass.

Meanwhile, the perimeter of the erosion gully shown in Photo 8 was also measured with the GPS receiver in June 2009 and November 2009. The two perimeters traversed at different times of the year were about identical with each other (Fig. 2), and showed little sign of progressive erosion during the half-year period.

Figs. 2 shows that in-plane deformations are up to or comparable to 5 cm, No trend for the in plane movement vectors was observed indicating that the local deformations were dominant at the marked GPS points instead of integral body movement. On the other hand, the maximum settlement of about 10 cm was reached near the crest with the greatest thickness of dynamically placed loose debris mass presumed, while a slight uplift was observed where the spillway water falls in the gully eroded near the toe of the debris mass.

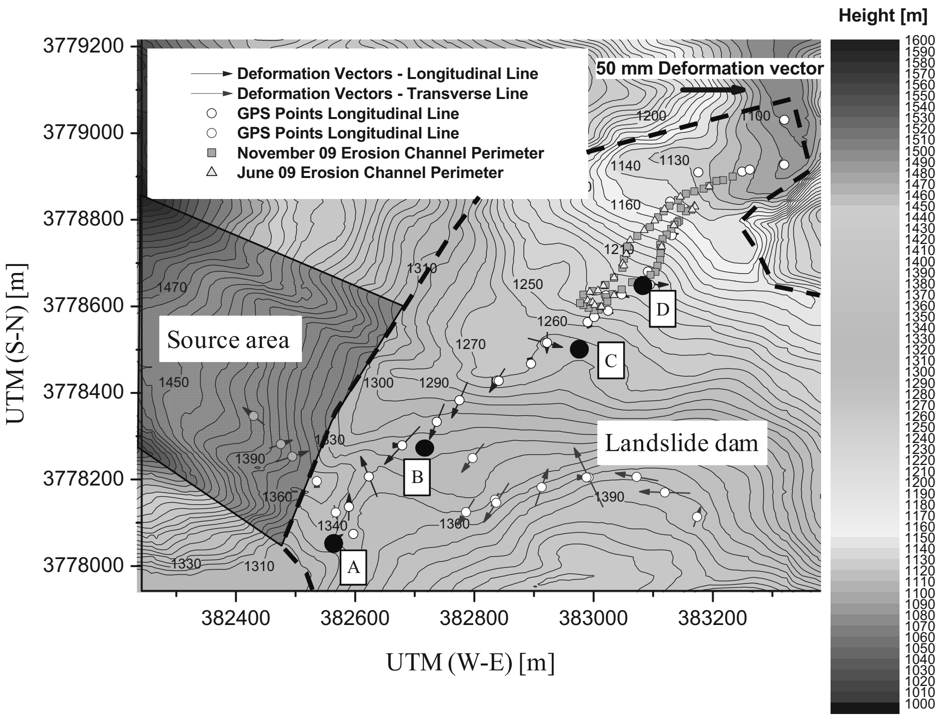


Figure 2 Deformation vectors superimposed on contour map generated using 5 m resolution DEM (after Sattar et al., 2010)

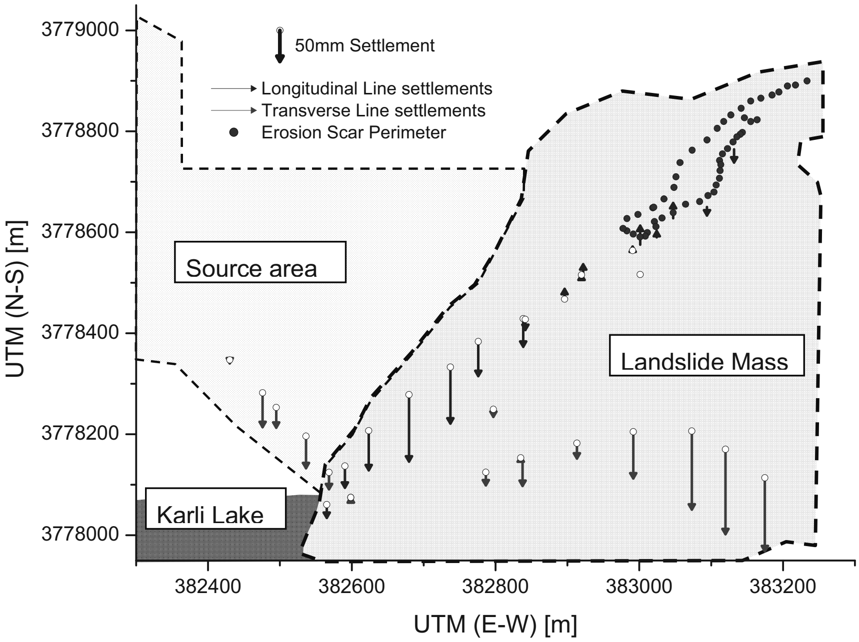


Figure 3 Settlements of landslide dam from June 2009 till November 2009 (after Sattar et al., 2010)

SLAKING TEST

Many cones of weathered fragments of mud rocks (Photo 3) and reddish water marks of finer substances remaining on rocks along the spillway (Photo 8) indicate that the mudstones there are highly susceptible to deterioration due to both weathering and slaking. The weathering/slaking process of the mudstones and sandstones may have accelerated removal of finer substances from the interior of the dam by seeping waters, and thus the breaching of the dam. To discuss this possibility, simple slaking tests were conducted on the rock samples taken from the landslide mass.

Testing procedure

Accelerated rock slaking tests were conducted according to the guideline No. JGS 2125-2006 edited by the Japanese Geotechnical Society.

Each specimen, approximately 50 cm^3 in volume, was successively subjected to oven-drying at a temperature of 40 °C for 24 hours followed by a complete immersion in distilled water for another 24 hours time. In the immersion process, distilled water was poured into a 20cm-diameter container until the specimen is fully immersed (approximately within 1 minute). This drying-wetting cycle was repeated three times.

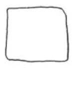
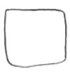
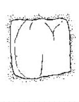
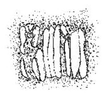
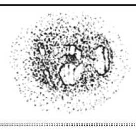
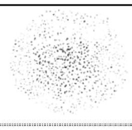
To see the changes in shapes and colors of beddings and/or laminae, each specimen was photographed before the immersion, within 1 minute (t= 0 min hereafter), at 30 min., 1 hour, 2 hours, 6 hours. and 24 hours elapsed times after pouring. Then the slaking extents of all samples were classified based on the features of deterioration illustrated in Table 1.

Test results

Figures 4 and 5 show inferred slaking classes of sandstone and mudstone specimens with respect to the immersion time. Photos 9 and 10 show respectively the pictures of the sandstone and mudstone specimens. In both, pictures to the left and right were taken before testing and after the third drying-wetting cycle, respectively.

Fig. 5 and Photo 10 show that the mudstone specimen is sensitive to the duty cycle for accelerating slaking, while Fig. 4 and Photo 9 show little sign of slaking of the sandstone specimen (estimated slaking class = 0). Small cracks and small bubbles showed up on the surface of the mud specimen 30 minutes after the first pouring. The mudstone specimen however did not show any further changes in the subsequent wetting/drying cycles, and their shapes were kept intact (estimated slaking class = 1).

Table 1 Definition of slaking classes (after guideline JGS 2125-2006)

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

A: Typical states for mudstone or fine-grained tuff B: Typical states for siltstone, sandstone or coarse-grained tuff

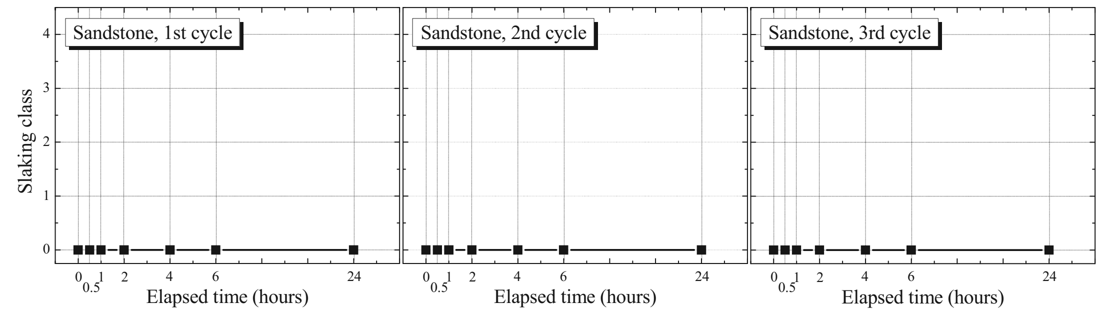


Figure 4　Change in slaking class of sandstone sample during accelerated slaking test

Photo 9　Sandstone samples before and after accelerated slaking test

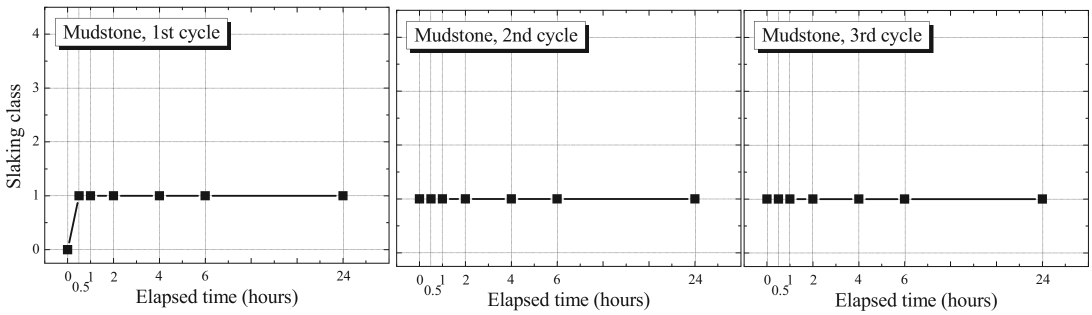


Figure 5　Change in slaking class of mudstone sample during accelerated slaking test

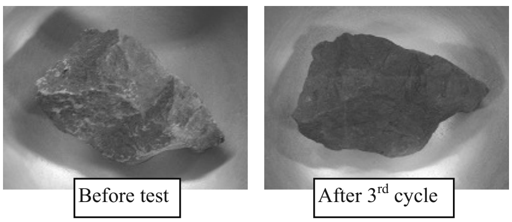


Photo 10　Mudstone samples before and after accelerated slaking test

DIRECT SHEAR TEST

In order to investigate the slaking effect of the rock materials on their strength, a series of direct shear tests were performed. However, sandstones on site were a little too much solid for the direct shear tests. Therefore samples used in the tests were the mudstones from the landslide site and gravelly soils comprising crashed sandstones (Chiba-gravel in Japan). The Chiba-gravel examined was highly resistant to slaking, and was considered as a good substitute of the sandstone specimen from the landslide site.

Specimens of mudstone and Chiba gravel were prepared by removing particles finer than 2 mm and larger than 4.75 mm, then oven-dried at a temperature of 100 °C. The specimens were not compacted to prevent particle breakage and to realize the similar conditions of the landslide dam, which was deposited loose without compaction.

Testing apparatus and procedure

The direct shear apparatus used in this study is shown in Fig. 6. Vertical and shear stress components, σ_v and τ were measured with the load cells, while the vertical and shear displacements (d and s) were measured with the Linear Variable Differential Transformers (LVDTs). The signals from these sensors are conditioned by a computer and returned to the loading system to realize either prescribed displacement or loading rates (See more details in Duttine et al., 2008). The initial specimen size is 12 cm in widths and 14 cm in height, having 2 cm opening between the upper and lower boxes (No. 9 and 10 in Fig.6).

Testing conditions for the examined specimens are shown in Table 2. In this study, loading process during the test consists of three stages as shown in Fig. 7. First, the specimen was subjected to consolidation with shear stress ratio, τ/σ_v, set at 0.5, and both stresses τ and σ_v steadily increased up to 50 kPa and 100 kPa, respectively. The shear stress ratio of 0.5 was determined simply from the maximum slope angle (about 28 degrees) of Hattian Bala landslide dam. After τ and σ_v reached the initial stress condition, the specimen was subjected to a sustained loading (creep loading) for six hours followed by a monotonic shear loading until it failed. In addition, in order to understand the slaking effects of the rock materials on their stress-deformation characteristics, some specimens were subjected to saturation after 3 hours creep while keeping the initial stress condition (τ= 50 kPa and σ_v= 100 kPa).

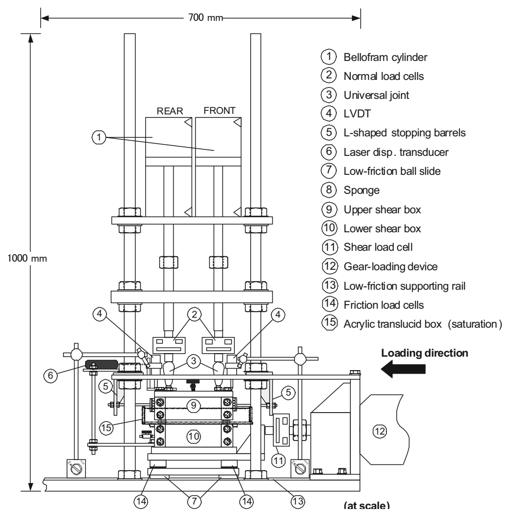


Figure 6 Direct shear apparatus

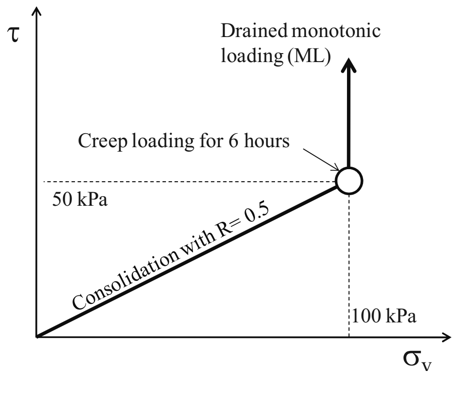


Figure 7 Stress-path during direct shear test

Table 2 Tested materials

No.	Sample	Specimen condition during creep at R= 0.5	Initial density ρ_0 [g/cm^3]	Density before ML ρ_1 [g/cm^3]
CG1	Chiba gravel	6 hr (dry)	1.393	1.450
CG2	Chiba gravel	3 hr (dry) and 3 hr (saturated)	1.393	1.455
MS1	mudstone	6 hr (dry)	1.393	1.458
MS2	mudstone	3 hr (dry) and 3 hr (saturated)	1.393	1.498

Test results

Figures 8 and 9 show time histories of shear displacements of Chiba gravel (CG1, CG2) and mudstone specimens (MS1, MS2), respectively. During creep loading, initial stress components (τ= 50 kPa and σ_v= 100 kPa) were kept unchanged. In the case of CG1 and MS1, the creep shear displacement gradually increased as the time went on, and then the buildup of displacements was almost terminated after 6 hours creep.

Once the specimens were saturated however in the middle of their creeping process, they exhibited a sudden increase in the deformation rate because of their slaking features. This increase is particularly clear in the mudstone specimen (MS2). The sudden increase in the shear displacement of 2 mm for the mudstone specimen is approximately eight times as large as the one for the Chiba gravel specimen (CG2).

Figures 10 and 11 show the variations of shear stress ratio, τ/σ_v, and shear displacement, s, with respect to vertical displacement, d, in the monotonic loading tests of Chiba gravel and mudstone, respectively. It is noted in these figures that the monotonic loading started at τ/σ_v = 0.5 because the specimens had been subjected preliminarily to shear stress condition (τ= 50 kPa and σ_v= 100 kPa) in their creep tests.

The stress-deformation curves for Chiba gravel samples in Fig. 10 are less sensitive to either drying or wetting process, while those for the mudstone specimens are highly susceptible to the change in the degree of saturation. The curves for dry specimen of mudstone (MS1) in Fig 11 exhibit clearly some dilative and strain-softening features like those for Chiba gravel samples (CG1 and CG2). On the other hand, the saturated mudstone specimen (MS2) did not show the strain-softening behavior and the specimen continued to be compressed during the shearing without exhibiting any dilative features. Such stress-deformation characteristics were generally common to soft soil materials, indicating that the mechanical properties of the examined mudstone deteriorated through its slaking process due to wetting during the creep loading.

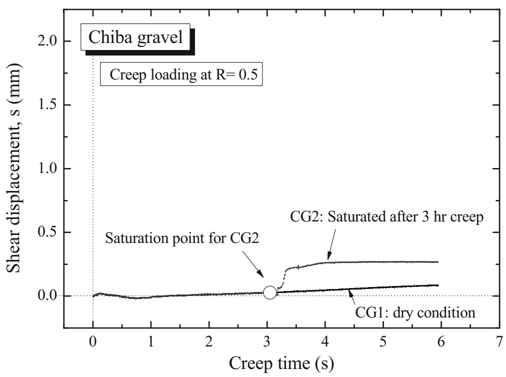


Figure 8 Effect of slaking on creep deformation on Chiba gravel (as sandstone)

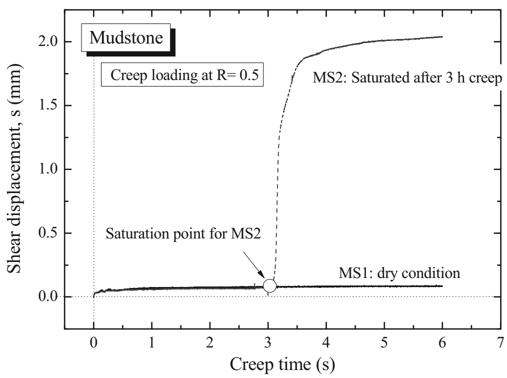


Figure 9 Effect of slaking on creep deformation on mudstone

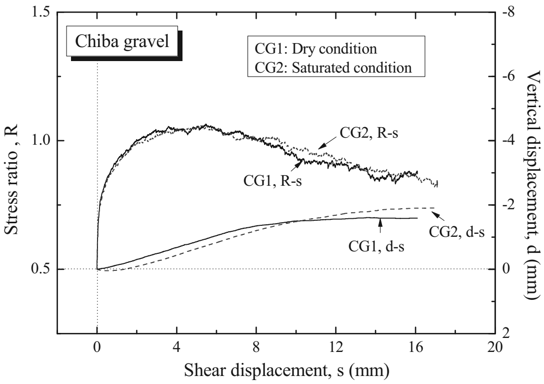


Figure 10 Effect of slaking on stress-deformation characteristics on Chiba gravel (as sandstone)

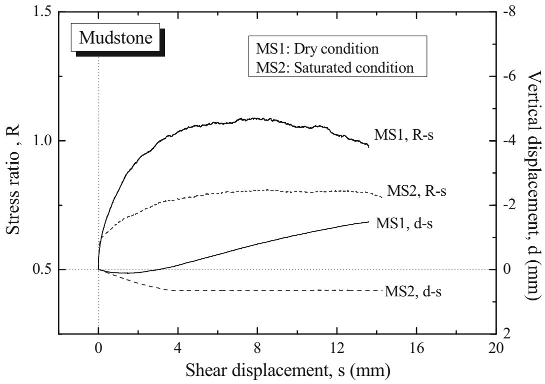


Figure 11 Effect of slaking on stress-deformation characteristics on mudstone

DISCUSSION AND CONCLUSIONS

From the slaking test performed on the mudstone and sandstone samples retrieved on the site, it can be derived that the slaking level of the mudstone was slightly higher than the one for the sandstone. From the direct shear tests performed on the mudstone and Chiba gravel samples, it can be concluded that the slaking-induced shear deformation of mudstone at constant shear stress was much larger than that of Chiba-gravel, a good substitute of the extremely hard in-situ sandstones. The peak strength of mudstone specimen has weakened through its slaking process before shearing.

Based on the above features of mudstones examined herein, it will be justified to think that slaking that has been built up in the interior of the landslide dam was a possible cause of accelerating the dam breaching. It is to be noted that the dam failed on 9th February by the heavy rain after drought. Murasawa and Konishi (1986) performed a series of slaking tests on mudstone samples having different initial degrees of saturation. They reported that the slaking level became higher as the initial degree of saturation of the specimen became lower, and the slaking could not be active unless the initial degree of saturation was lower than 80 %.

As we mentioned previously, there was significant erosion at the toe part of landslide dam during dry season (between November 2008 and June 2009) while no clear and visible sign of landform change was observed during rainy seasons (between June 2008, 2009 and November 2008, 2009). In fact, it was exactly dry season when the landslide dam failed. It may be considered that the significant slaking has caused rapid reduction in the strength of the dam that was mainly formed by mudstones and therefore buildup of deformations simultaneously, and then the dam body failed due to the increase in the water level of the Karli lake, which can have caused an increase in a hydraulic grade in the dam body as well as the spillway discharge, accelerating the progressive breaching.

At the present time, we need to gather as many pieces of information as possible from the site and from other sources in order to understand the failure process of the Hattian Bala landslide dam. Although the landslide dam already failed, unstable debris mass is probably remaining there clogging partly of the path of smaller amount of water impounded behind the debris mass (see Photo 2). Further investigations related to this issue are mandatory and the results will be reported by the authors in the near future.

ACKNOWLEDGMENT

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