Investigation of landslide calamity due to torrential rainfall in Shobara City, Japan

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Abstract

On 16th July 2010, a series of slope failures occurred in the city of Shobara, Japan due to a torrential rainfall. The slope failures were found to have occurred at about 200 locations in a narrow area of 16 km\textsuperscript{2}. Most of the failures in this catastrophic disaster were due to the collapse of the middle portion of a mountainous slope. The primary purpose of this study was to present the landslide event that took place in Shobara City and to explore the mechanism of the multiple planar slope failures in the region. A series of field tests was carried out using a recently developed lightweight dynamic cone penetrometer in the middle portion of the selected planar slopes. The results revealed that the slopes were composed of two soil layers, weathered Rhyolite and Kuroboku. The failures took place at the boundary of these two soil layers. Laboratory tests were conducted to find the relevant parameters for the stability analysis, and laboratory scale model tests were performed to understand the mechanism of the slope failures in this region. The results of the model tests revealed that the Kuroboku layer led to a rise in the water table in the middle portion of the slope, while preventing the downward flow of groundwater. As a result, the failures are found to have initiated and occurred in the middle portion of the mountainous slope. The factor of safety, evaluated based on a slope stability analysis, was found to be close to 1, when the groundwater level rose to the ground surface and the apparent cohesion of the weathered Rhyolite soil over the Kuroboku layer decreased to 1.3 kPa.

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1. Introduction

In recent years, unexpected and extreme rainfall events have caused landslide disasters all over Japan due to abrupt changes in weather conditions. Some of these rainfall events occurred within a few hours, and certain areas received more rainfall than is usually experienced over an entire month ((Ex. Aso, Kumamoto Prefecture in July 2012 (Miyabuchi, 2012); Kii Peninsula in September 2011 (Yamada et al., 2012), Hofu in Yamaguchi Prefecture in July 2009 (Fukuoka et al., 2009)). These events have become more frequent in the past few years and have not only claimed human lives and caused considerable property loss, but have also severely affected the day-to-day activities and the development of the area. On 16th July 2010, a series of slope failures occurred in Shobara City, Hiroshima Prefecture, Japan due to abrupt and heavy rainfall. This torrential rainfall led to simultaneous slope failures at more than 200 locations within a narrow 4 km \times 4 km area. The area where the slope failures occurred is shown in Fig. 1.

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This disastrous event led to the swallowing of trees, the spilling out of streams, and the flattening of villages in mountainous areas as well as the overflowing of roads and farmland. The mountainous areas on both sides of the roads experienced concentrated slope failures. Most of these failures were found to have occurred in the middle portion of the mountain and in slopes having gradients of more than 30°. A failed mass flowed over roads, decimated houses and farmlands in the region, claimed 1 life, and caused 1 major injury. According to the statistics published on the Hiroshima Prefectural Government web site, 12 houses were fully destroyed and 11 houses were partially damaged (Tsuchida et al., 2010; Hiroshima Prefecture, 2014). Moreover, it was reported that 502 farms and agricultural facilities, 65 forestry facilities, 51 rivers, 2 erosion-control barriers, and 21 roads were damaged due to this event (Tsuchida et al., 2010). Therefore, this disaster was considered to be one of the major events, in terms of the number of slope failures, property loss, and the pattern of failures, that has occurred in the region over the past few decades.

The overall purpose of this paper is to present the series of failures that occurred in the city of Shobara, in Hiroshima Prefecture, Japan and to examine the failure mechanism of the slope failures that occurred in the region. Emphasis is given to discovering the failure mechanism of the planar slopes in the area through aerial photographs, site surveys, and site investigations. In addition, a series of in-situ and laboratory tests was conducted to examine the engineering properties of soils and to use the data for stability analyses. Besides that, a series of laboratory model tests was conducted to observe the failure mechanism of the planar slopes for different rainfall intensities. Stability analyses were carried out based on the data collected from field and laboratory tests to gain a better understanding of the slope failures in the region. Based on the site investigations, site and laboratory tests, models tests, and stability analyses, the failure mechanism of the slope failures that occurred in Shobara City is presented in this paper.

1.1. Geology and topography of the affected area

The affected area is composed of weathered remnants of Rhyolite and Kuroboku soils. Rhyolite is an igneous volcanic rock and Kuroboku is a mixture of volcanic ash and organic matter. Kuroboku soil is black in colour and covers most of the area in the region. The thickness of Kuroboku soil in the affected area varies from a few centimeters to a few metres. Weathered Rhyolite is underlined by Kuroboku soil in most parts of the slopes in the area. Its thickness varies from location to location and is about 1.0–1.5 m at the lower part of the slope. Most of the land in the area is mountainous. The angle of the mountainous slopes varies from 17° to 46°.

1.2. Meteorological data in and around the affected area

Rainfall data were collected from the rainfall gauges in and around the affected area, as shown in Fig. 2. Table 1 presents the cumulative rainfall data before and during the slope failures and the maximum hourly rainfall at different gauging stations in the vicinity of the affected area. Fig. 3(a) shows the cumulative rainfall and hourly rainfall at Ohto gauging station from 1st to 17th July 2010 with reference to the data presented on the Hiroshima Prefectural Government web site (Tsuchida et al., 2010). Fig. 3(b) shows the cumulative and hourly rainfall on the day, 16th July 2010, that the series of slope failures occurred. The peak rainfall recorded at Ohto rain-gauge station was 91 mm/h from 3:40 to 4:40 pm on 16th July 2010. However,
the average rainfall over 3 h (from 3.30 pm to 6.30 pm), as shown in Fig. 3(b), was 58 mm/h. On the same day, from 3 pm to 6 pm, the AMeDAS rain gauge, which is located 9 km away from the Ohto gauging station, recorded 65 mm of rainfall. Therefore, the rainfall received throughout the area was not uniform and strong rainfall was concentrated extensively at the disaster site. This can be clearly observed in the data presented in Table 1.

### 2. Visual inspection of the slope failures

A site survey was carried out to closely examine the mechanism of the failures. Most of the failures in this catastrophic event were observed to be failures that occurred in the middle portion of the mountainous slope. This can be clearly seen from the topography map based on LiDAR, which was taken during the aerial survey by Asia Air Survey Co., Ltd., Japan, as shown in Fig. 4. Slope failures are marked by light brown circles and close inspection revealed that most of the failures occurred in mountainous slopes rather than along valleys. Two such planar slope failures, planar slopes A and B, marked in Fig. 4, are illustrated in Fig. 5.

The initiation of the slope failures was found to be from 20 m to 40 m from the ridge of the mountains. Most of these failures occurred on slopes having gradients of more than 30°, and the failed mass flowed across roads and along rivers without accumulation on the failed slopes. Observations were made at the site and it was found that the slope failures occurred at the right bank of the Shinodo River causing damage to houses at the downstream. As shown in Fig. 6, the slopes, indicated by red arrows, slipped downward and flowed onto the roads, and thereafter, to human settlements and farmland. The Shinodo River was found to have been widened (up to 10 m in some places) as a result of the failed mass flowing along the river. Cobblestones and gravel were seen to have accumulated as a layer at the downstream with a height of

### Table 1
Rainfall data for different gauging stations.

<table>
<thead>
<tr>
<th>Station (as shown in Fig. 3)</th>
<th>Cumulative rainfall before July 16th (mm)</th>
<th>Rainfall from 3 pm to 6 pm on July 16th (mm)</th>
<th>Maximum hourly rainfall from 3 pm to 6 pm on July 16th (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nagata</td>
<td>265</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Hiwa</td>
<td>299</td>
<td>8</td>
<td>7</td>
</tr>
<tr>
<td>Shobara</td>
<td>262</td>
<td>3</td>
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</tr>
<tr>
<td>Kawakita</td>
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<td>125</td>
<td>54</td>
</tr>
<tr>
<td>Ohyo</td>
<td>300</td>
<td>72</td>
<td>30</td>
</tr>
<tr>
<td>Ohto</td>
<td>267</td>
<td>173</td>
<td>72</td>
</tr>
<tr>
<td>Saijo Nakano</td>
<td>257</td>
<td>51</td>
<td>33</td>
</tr>
<tr>
<td>Motomura</td>
<td>262</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>
about 3 m. In addition, it was observed that the Rhyolite rock had
been exposed on the riverbed due to the fast movement of the
failed mass and the debris along the river.

Close inspection of the upper part of the collapsed slopes
revealed that there were a number of cavities in the soil whose size
ranged from 5 mm to 15 mm. These are illustrated in Fig. 7. In
recent years, the importance of water flowing through cavities,
called “pipes”, that form on mountainous slopes as a result of the
activities of small animals and rotting plant roots, has been
recognized (Pierson, 1983). These “pipes” have a significant effect
on the occurrence of slope failures under heavy rainfall conditions
(Jones 1994; Kitahara et al., 1994; Uchida et al., 2001). It has also
been pointed out that the “pipes” have a significant effect on water
drainage within the slopes. If drainage is functioning properly, it
results in the lowering of the groundwater level. However,

incomplete drainage of the “pipes”, due to partial or full blockage,
leads to water accumulation in some parts of the slopes. This
results in the generation of high pore water pressure, and sub-
sequently, unstable soil masses. Observations made at the site have
revealed the presence of pipes in the upper parts of the slopes, as
shown in Fig. 7. Therefore, these pipes may have had a full or
partial impact on the failure of the slopes. More data and analyses
are required to draw a sound conclusion on the impact of “pipes”
on the slope failures in the region.

3. Field tests (lightweight dynamic cone penetration tests)

Field tests were carried out at slopes A and B, as illustrated
in Fig. 5, using the recently developed lightweight dynamic cone
penetrometer shown in Fig. 8. It was designed and developed in
France in 1990 (Langton, 1999). It is operated on variable energy
and weighs 20 kg; it can be operated by one person at almost any
location to a depth of 6 m. This device mainly consists of an anvil
with a strain gauge bridge, a central acquisition unit, and a dialogue
terminal. The hammer is of the no rebound-type and weighs
1.73 kg. The stainless steel rods are 14 mm in diameter and 0.5 m
in length. Cones of 2, 4, and 10 cm² in area are available. More
information about the apparatus and its applicability to natural
slopes, especially for weathered Granitic slopes, can be found in
the literature published by Tsuchida et al. (2011, 2014) and
Athapaththu et al. (2006, 2007a, b).

A series of lightweight dynamic cone penetration tests (LWDCPTs)
was carried out on slopes A and B at 5 m grids, as shown in Fig. 9. The average dimensions of the collapsed area of
slope A are 38 m in length and 31 m in width. The average length
and width of the collapsed area of slope B are 32 m and 9 m,
respectively. The average gradients of slopes A and B are 32°
and 30°, respectively. Twenty-six points were selected from slope A,
while twelve points were selected from site B for the in-situ tests.
The LWDCPTs were carried out until the hard layer, i.e., cone
resistance \( q_{da} \), became greater or equal to 10 MPa for the third time
in a row of a particular test. Fig. 10 shows the soundings collected
from eight out of the twenty-six locations for slope A and four out
of the twelve locations for slope B. A close review of the
soundings reveals that the cone resistance was less than 1 MPa
up to 1 m at most of the testing points. Therefore, these weak
Soils may have had a large impact on the failures occurring in the region. Soil layers and their arrangements are shown in Fig. 10(a) and (b). The Kuroboku soil layer showed cone penetration resistance of less than 1 MPa. In most of the locations, the weathered Rhyolite showed cone resistance of more than 1 MPa. The points from A-1 to A-4 and from A-5 to A-8 are marked on the right and left sides of cross section x–x, respectively, as can be seen in Fig. 9(a). Close inspection of Fig. 10(a) revealed that the pair of soundings at points A-1 & A-5, A-2 & A-6, A-3 & A-7, and A-4 & A-8 are almost similar in stratigraphy along the soil profile. The thickness of the soil profile increases towards the foot of the slope. Kuroboku soil was not seen at the top of the collapsed slopes. In addition, the thickness of the Kuroboku soil increased towards the downward side of the slope. A similar pattern of stratigraphy is observed in slope B, as shown in Fig. 11(b). Fig. 11(a) and (b) show the estimated cross sections of slope A (X–X’) and slope B (Y–Y’).

The dotted lines show the estimated sliding surfaces of both slopes. As shown in Fig. 11(a) and (b), the upper part of the collapsed area was covered with weathered Rhyolite and the lower part was covered with Kuroboku soil. The failure took place at the boundary of these two soil layers. Although two slopes were investigated in this study, the rest of the planar slope failures were also more or less similar in stratigraphy; i.e., the upper part of the collapsed slopes consisted of light brown colour weathered Rhyolite and the lower part of the slopes consisted of grey or black colour Kuroboku soil. Hence, the stratigraphy along the slopes may have played a vital role in the slope failures in the region. In the next section, the laboratory test results for these soils will be shown and discussed.

4. Laboratory tests, analysis, and results

Soils samples were collected at a depth of 30 cm from the ground surface at the sites shown in Fig. 9(a) to perform the laboratory tests. Sieve and hydrometer tests were conducted to examine the grain size distribution of the weathered Rhyolite and Kuroboku soils, respectively. Permeability tests were conducted on three undisturbed samples collected from weathered Rhyolite in the upper parts of the failure areas. Besides that, three undisturbed samples were collected from the lower part of the failure areas from the Kuroboku soil. One remoulded sample of Kuroboku soil was prepared for the permeability tests maintaining a void ratio of 2.5, similar to the site conditions. A series of laboratory direct shear tests was carried out from the disturbed soil samples collected from the weathered Rhyolite at the same site. Reconstituted samples were prepared with four different void ratios, ranging from 1.1 to 1.4 in 0.1 increments, and degrees of saturation ranging from 30% to 70% in 20% increments under different normal stress levels.

4.1. Grain size distribution and permeability tests

The grading curves are illustrated in Fig. 12(a). The grain size distribution curve for Masado soil, derived from weathered Granite spread over Hiroshima Prefecture, Japan, is also shown in the same plot. The authors have conducted an extensive field investigation and laboratory experiments for weathered Granite over the past 10 years and have developed sound methodology to assess the slopes susceptible to failure in the Hiroshima region. Therefore, the properties of Masado soil were compared elsewhere in this paper with the properties of the weathered Rhyolite and Kuroboku soils. A graphical representation reveals that all soil samples contained a high content of coarse fraction, mainly sand, and a considerable amount of fine fraction. However, both the weathered Rhyolite and
the Kuroboku soil showed a higher content of fine fraction than that of the Masado soil. Therefore, these three soils may have different shear strength properties.

Fig. 12(b) shows the results of permeability tests conducted for the weathered Rhyolite and Kuroboku soils. Coefficients of permeability for the weathered Rhyolite are higher than those for the Kuroboku soil. This may be due to the presence of a higher content of coarse particles in the weathered Rhyolite than in the Kuroboku soil. The void ratios of the undisturbed Kuroboku samples, for which falling head tests were conducted, were in the range of 2.3–2.7; and consequently, the coefficients of permeability were in the range of $10^{-3}$ cm/s. The difference in the coefficients of permeability in the undisturbed and the reconstituted states of the Kuroboku soil may be due more to the particle arrangement in the original soil structure than that in the remoulded state. The lower values for the coefficients of permeability and the higher values for the void ratios of natural fine grained Kuroboku soil are due to the existence of voids between the agglomerates consisting of fine particles. Kuroboku soil contains more than 50% fine particles; and hence, this soil shows the behaviour of fine grained soils. Based on these facts, it was considered that the shear strength of the Kuroboku soil should be determined as clayey soil.

4.2. Shear strength properties of soils

A series of direct shear tests on unsaturated Rhyolite was carried out with the use of a conventional direct shear apparatus (Tsuchida et al. 2011). The diameter of the shear box was 6.0 cm and the height of the specimen was kept at 2.0 cm, as per the Standards (JGS, 2000). As drainage was permitted from the upper porous plate, but the drainage line was not connected to a water supply, the shear was made under a constant water content when dilatancy was positive, and shear was made under the drained condition when dilatancy was negative. The frictional angle and the apparent cohesion were determined based on the total stress.

In the preparation of the specimens, the measured amount of soil was mixed with water calculated according to the required degree of saturation. The soils were then placed in three layers in the direct shear mould and each layer was given light compaction by a pestle weighing 0.24 kg to avoid crushing the individual particles in the specimen. All the tests were conducted at normal stress levels of 40.0, 80.0, and 160.0 kPa. Shearing was achieved by horizontally displacing the upper half of the direct shear box at a constant shear rate of 0.2 mm/min, as described in JGS 0561-200 (JGS, 2000). Fig. 13(a)–(c) shows plots of the shear stress against the deformation obtained from the direct shear tests for samples with void ratios of 1.20. Fig. 14(a)–(c) is the failure envelopes of the total stress for a void ratio after consolidation of 0.8 for degrees of saturation of 30%, 50%, and 70%.

Fig. 15(a) and (b) shows the variation in friction angle $\phi_d$ and apparent cohesion $c_d$ with void ratio $e$ and degree of saturation $S_r$. Referring to Fig. 13, Eqs. (1) and (2) can be obtained for the apparent cohesion and the friction angle in terms of the void ratio and the degree of saturation, respectively.

$$c_d = 77.0 - 0.41S_r - 26.0e_{\text{Rhyolite}}$$  
$$\phi_d = 44.7 - 6.45e_{\text{Rhyolite}}$$

where $S_r$ is the degree of saturation (%) and $e_{\text{Rhyolite}}$ is the void ratio of Rhyolite. Based on a series of laboratory cone penetration tests conducted for weathered Granitic soils, Tsuchida et al. (2011) proposed a relationship between the void ratio and the penetration resistance, $q_d$, based on LWDCPTs, as shown in Eq. (3).

$$\epsilon_{\text{Granite}} = 1.19 - 0.004 \ln(q_d) - 0.0074S_r(LWDCPT)$$
where \( q_{d5} \) is the cone resistance for an overburden stress of 5 kPa, as given in Eq. (4), and \( S_{r(LWDCPT)} \) is the degree of saturation at the time the LWDCPTs were conducted.

\[
q_{d5} = q_d - 0.01 \times (\gamma'z - 5) \tag{4}
\]

In order to develop relationships to evaluate the shear strength parameters for the weathered Rhyolite, in terms of cone resistance and the degree of saturation, the same shear strength obtained for both soils from direct shear tests was considered. The void ratios for the direct shear tests conducted here ranged from 0.6 to 1.0 and from 1.1 to 1.42 for the weathered Granite and the weathered Rhyolite, respectively. Using these data, a relationship was developed considering the same shear strength properties of both soils, as shown in Eq. (5)

\[
e_{\text{Rhyolite}} = 0.80e_{\text{Granite}} + 0.62 \tag{5}
\]
where $e_{\text{Rhyolite}}$ and $e_{\text{Granite}}$ are the void ratios of the weathered Rhyolite and the weathered Granite having the same shear strength, respectively. Eq. (6) can be obtained by substituting Eq. (5) into Eq. (3).

$$e_{\text{Rhyolite}} = 1.572 - 0.0672 \ln(q_{d5}) - 0.00592S_{r,(\text{LWDCPT})}$$  \hspace{1cm} (6)

Using Eqs. (1), (2), and (6), the friction angle and the apparent cohesion for the weathered Rhyolite can be derived in terms of $q_{d5}$ and $S_r$, as shown in Eqs. (7) and (8).

$$\phi_d = 34.56 + 0.433 \ln(q_{d5}) + 0.038S_{r,(\text{LWDCPT})}$$  \hspace{1cm} (7)

$$c_d = 36.13 + 1.747 \cdot \ln(q_{d5}) - 0.41S_r + 0.154S_{r,(\text{LWDCPT})}$$  \hspace{1cm} (8)

The degree of saturation at the time of the LWDCPTs was 40%; and hence, Eqs. (7) and (8) become Eqs. (9) and (10).

$$\phi_d = 36.08 + 0.433 \ln(q_{d5})$$  \hspace{1cm} (9)

$$c_d = 42.29 + 1.747 \cdot \ln(q_{d5}) - 0.41S_r$$  \hspace{1cm} (10)

Fig. 16 shows the variation in shear strength parameters calculated based on Eqs. (9) and (10) along soil profiles A-4, A-8, and A-9, as shown in Fig. 9(a). As shown in the figure, the shear strength parameters evaluated from the formulas showed a good agreement with those obtained from laboratory direct shear tests. To confirm the validity of Eqs. (6) and (7), in order to determine the shear strength parameters of the weathered Rhyolite, more data are required.

5. Laboratory model tests

The failures of natural Masado slopes or Rhyolite soil slopes have been considered as surface shallow failures, and the infinite slope stability has been used for the analysis (Aboshhi and Sokobiki, 1972; Thi et al., 2003; Tsuchida et al., 2014). However, the field investigations here have shown that the failures of planar slopes took place at the boundary between the weathered Rhyolite and the Kuroboku soil, and that the difference in permeability may have caused a different failure pattern from infinite slopes. To examine the failure mechanism of planar slopes related to the boundary of the two layers, laboratory model tests were conducted. The model was constructed to be 480 mm in width and 790 mm in length, as shown in Fig. 17. The inclination of the modelled slope was kept at 30°. Drainage was provided at the base of the slope. Nine piezometers were installed in the middle of the slope bed at different distances along the length, as shown in Fig. 17. As found previously, the coefficient of permeability of the Kuroboku soil is lower than that of the weathered Rhyolite; and hence, a sand and clay mixture was used to represent the Kuroboku soil, while Toyoura sand was used for the weathered Rhyolite to make the slope in the model test. 90% Toyoura sand was mixed with 10% Mizushima port clay, and this was...
used as the Kuroboku soil. The coefficients of permeability of the Toyoura sand and the sand–clay mixture, measured by constant head permeability tests, were found to be 0.015 cm/s and 0.009 cm/s, respectively. These values are close to those illustrated in Fig. 12(b). A 50-cm-thick layer was formed from the Toyoura sand on the base layer which was made of compacted clay. In order to create similar site conditions, a sand–clay mixture was placed at the lower part of the slope to represent the Kuroboku layer, as shown in Fig. 17. To study the effect of the Kuroboku layer on the stability of the slope, the model tests were carried out with the presence and absence of the Kuroboku layer under different rainfall conditions. The reason for using Toyoura sand and a sand–clay mixture instead of the weathered Rhyolite and the Kuroboku soil is that, the coefficients of permeability of reconstituted Rhyolite soil and Kuroboku soil were much smaller than those of the undisturbed samples shown in Fig. 12, and that the coefficients of permeability for the Toyoura sand and the sand–clay mixture were close to those of the undisturbed Rhyolite and Kuroboku soils.

Table 2 shows the formation of the soil layers and the rainfall intensities for the model tests conducted. Case 1 is a slope without the clay–sand mixture layer in which the rainfall intensity is 40 mm/h. In Cases 2 and 3, the clay–sand mixture soil layer was placed on the foot of the slope as shown in Fig. 15, and the rainfall intensity was 40 mm/h and 20 mm/h respectively. To make the degree of saturation the same as the site conditions, the initial degree of saturation of the Toyoura sand and the mixture soil was kept to 40%.

The modeled slope was observed to have failed in Cases 1 and 2, where the rainfall intensity was 40 mm/h. For both of these cases, a groundwater table formed in the slope. However, in Case 3, in which the rainfall intensity was 20 mm/h, no groundwater table was formed and slope failure did not occur. Fig. 18(a) and (b) shows the variation in groundwater table recorded from the piezometers installed along the slope with

![Fig. 13. Shear stress vs shear deformation curves.](image1)

![Fig. 14. Failure envelopes.](image2)
time for Cases 1 and 2. Fig. 19(a) and (b) illustrates the water table variation along the slope just before the failure. Fig. 20 (a) and (b) shows two plates corresponding to the failure pattern of Cases 1 and 2, respectively. Observations revealed that the surface of the soil cracked and flowed downward. The time for the initiation of the slope failures was 21 min in Case 1 and 17 min in Case 2. In Case 1, the groundwater level rose rapidly just before the failure occurred; the water level rose to 20 mm 1 min before the failure and it was at 30 mm when the failure occurred. In a similar manner, the groundwater table rose up suddenly 3 min before the failure occurred in Case 2. In that case, the water table had risen to nearly 30 mm at the time of the failure.

In Case 1, the failure was initiated at the lower part of the slope and the soil mass moved downward. Although the failure progressed to the upper part, resulting in failure throughout the
slope, the depth of the soil mass was small, showing shallow surface failure. The failed soil mass accumulated at the lower part of the slope. However, sliding occurred near the surface and no deep sliding was observed. On the other hand, in Case 2, with the sand–clay mixture layer, although the failure initiated near the boundary of the sand–clay mixture layer, a circular failure was observed in the Toyoura sand layer above the boundary, and the failure plane reached the base layer as the surface of base layer was observed. In Fig. 18(b), it was seen that the water table rose to Toyoura sand under the sand–clay mixture layer. It is considered that the pore pressure in the sand–clay mixture did not rise so much due to the low permeability and that the sand–clay mixture layer did not reach the failure condition. Subsequently, the sand–clay mixture layer was covered with Toyoura sand which moved from the upper part of the slope by means of the circular shape failure. The behaviour of the two layers observed in Case 2 is similar to that of the slope failures occurring in the city of Shobara where the weathered Rhyolite covered the Kuroboku soil layer in most of the locations.

6. Slope stability analysis and discussion

In order to examine the mechanism of the failures, stability analyses were conducted based on the Simplified Janbu’s method (Janbu, 1954). Fig. 11(a) shows the cross section of the slope where the depth of the profile was determined based on LWDCPT data. The slope was divided into a number of slices and the minimum factor of safety was calculated based on a developed computer program. Table 3 shows the parameters for the stability analyses, where the apparent cohesion \( c_d \) and the friction angle \( \phi_d \) of the weathered Rhyolite were obtained from Eqs. (9) and (10). The degree of saturation of the soil was assumed to be 70% and 100% at the upper and lower slopes of the ground, respectively. The shear strength parameters for the Kuroboku soil were determined based on fully saturated conditions. This is due to the fact that the soil contains about 50% fine particles, as mentioned in Section 4.1, and the stability analysis was carried out considering heavy rainfall conditions. The friction angle of the Kuroboku soil was taken as zero and undrained cohesion, \( c_u \) was evaluated based on Eq. (9) proposed by Matsui et al. (2010).

\[
c_u = \frac{q_d}{26.9}
\]

Stability analyses were conducted under 2 conditions: Analysis 1 considers the condition without the water table and the degree of saturation of the weathered Rhyolite was kept to 70%, while analysis 2 considers the water table at the ground surface due to the existence of a low permeable layer at the lower part of the slope, in which the apparent cohesion,
calculated from Eq. (10), was reduced to 1.3 kPa due to an increase in the degree of saturation to 100%. The factors of safety calculated for these two cases are shown in Table 4. In analysis 1, the factor of safety was found to be 4.54 and the slopes are unstable conditions. However, as in analysis 2, the factor of safety was found to be drastically reduced to 1.10, making the slope critical to failure. If the slope failed in reality, the factor of safety would be less than 1.0. In the stability analysis, the change in void ratio of the Rhyolite soil was not considered. However, due to the up flow of the ground water, the void ratio of the Rhyolite soil may be increased, and the friction angle and the apparent cohesion may be reduced further.

In summary, the mechanism of failure of planar slope A studied here is considered as follows:

1. Due to the heavy rainfall, rain water infiltrated into the slope. The water flow in the slope was partially blocked by the low permeable Kuroboku layer. The groundwater level rose to the surface.
2. The weathered Rhyolite soil was saturated and the apparent cohesion dropped to about 1 kPa, which put the slope in a critical situation. When the up flow of water might have resulted in an increase in void ratio and a decrease in further apparent cohesion and friction angle, sliding took place.

7. Conclusions

The aim of this research was to examine the mechanism of multiple planar slope failures that occurred due to torrential rainfall on 16th July 2010 in Shobara City, Japan. A series of site surveys, field tests, laboratory tests, model tests, and stability analyses were conducted to gain a better understanding of the slope failures in the region. Lightweight dynamic cone penetration tests and a series of laboratory direct shear and permeability tests were conducted to better understand the slope failures occurring in Shobara City. Moreover, laboratory scale model tests were conducted to observe the failure in two layered soils. Based on the data and the results, the following conclusions are drawn from this study:

1. The heavy rainfall which lasted for 3 h in the city of Shobara, Japan was the main cause of planar slope failures. Slopes began to collapse approximately 2 h after the heavy rainfall. Observations made during the site visits revealed that the most of the failures initiated 20–40 m below the ridge of the mountain.

### Table 3
Parameters for stability analyses.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Weathered rhyolite</th>
<th>Kuroboku soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$ (kN/m³)</td>
<td>16.2</td>
<td>11.6</td>
</tr>
<tr>
<td>$\gamma_{sat}$ (kN/m³)</td>
<td>17.6</td>
<td>13.8</td>
</tr>
<tr>
<td>$\delta_r$ (%)</td>
<td>70 (above the water table)</td>
<td>100</td>
</tr>
<tr>
<td>100 (below the water table)</td>
<td>180</td>
<td></td>
</tr>
<tr>
<td>$c$ (kPa)</td>
<td>1.29 (below the water table) (Eq. (10))</td>
<td>18.0 (Eq. (11))</td>
</tr>
<tr>
<td>$\phi$ (°)</td>
<td>36.4° (Eq. (9))</td>
<td>0.0</td>
</tr>
</tbody>
</table>

### Table 4
Conditions and factors of safety for stability analyses.

<table>
<thead>
<tr>
<th>Analysis case</th>
<th>Ground water table</th>
<th>Apparent cohesion of weathered rhyolite</th>
<th>Factor of safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis 1</td>
<td>None</td>
<td>13.6 kPa (Eq. 8)</td>
<td>4.55</td>
</tr>
<tr>
<td>Analysis 2</td>
<td>At ground surface</td>
<td>1.29 kPa (Eq. 8)</td>
<td>1.10</td>
</tr>
</tbody>
</table>

![Fig. 20. Shape of sliding. (a) Case 1. (b) Case 2.](image)
(2) Visual inspection of the site and the data from field tests by LWD CPTs revealed that the slopes composed of two soil layers, namely, weathered Rhyolite and Kuroboku soil. The failures took place at the boundary of these two soil layers. The stratigraphy along the slopes may have played a vital role in the slope failures in the region.

(3) Laboratory tests on both soils revealed that the Kuroboku soil is less permeable than weathered Rhyolite. Due to the significant difference in permeability between both soils, rainwater that infiltrated and flowed into the weathered Rhyolite was blocked near the Kuroboku layer. This caused the accumulation of water which resulted in the rise in pore water pressure at the boundary between the two soil layers. This may have been the potential cause of the occurrence and the acceleration of the failures.

(4) Two formulas were developed to determine the shear strength parameters for the reconstituted Rhyolite soil based on the void ratio and the degree of saturation.

\[
\phi_d = 44.7 - 6.45 e_{\text{Rhyolite}} \quad \text{and} \quad c_d = 77.0 - 0.41 S_r - 26.0 e_{\text{Rhyolite}}
\]

Where \(e_{\text{Rhyolite}}\) is the void ratio of Rhyolite soil, \(S_r\) is the degree of saturation. Based on the established formulas developed for weathered Granite, new formulas for estimating the shear strength parameters for weathered Rhyolite were derived as follows:

\[
\phi_d = 36.08 + 0.433 \ln(q_{ds}) \quad \text{and} \quad c_d = 42.29 + 1.747 \cdot \ln(q_{ds}) - 0.41 S_r
\]

where \(q_{ds}\) is the cone resistance for an overburden stress of 5 kPa.

(5) Laboratory model tests were conducted to study the effect of the boundary between the upper sand layer and the lower sand–clay mixture layer. In the model tests carried out on sand and sand–clay mixture layers, the failure was initiated near the boundary of the two layers. A circular shape failure was observed in the upper sand layer above the boundary, where the failure plane was deep enough to reach the base layer. The behavior of the two layers observed in Case 2 is similar to that of the slope failures occurring in the city of Shobara where weathered Rhyolite covered the Kuroboku soil layer in most of the locations.

(6) In the stability analyses, a higher factor of safety was observed with the absence of the groundwater table. When the groundwater table was at the ground surface, the factor of safety was found to be 1.10, a critical situation for failure.

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