EVALUATION OF RUN-OUT DISTANCES OF SLOPE FAILURES DURING 2004 NIIGATA-KEN CHUETSU EARTHQUAKE

YOSHIMICHIT TSUKAMOTO\(^{(a)}\), KENJI ISHIHARA\(^{(a)}\) and YASUHIDE KOBARI\(^{(a)}\)

ABSTRACT

During 2004 Niigata-ken Chuetsu Earthquake in Japan, a large number of landslides occurred on natural slopes, especially at the hillsides in the region of Yamakoshi. In many of the large slides, the debris has travelled through a fairly long distance, aggravating the disaster caused by the landslides. In recognizing its importance, case studies were undertaken on the run-out distance of the landslides at several sites at Higashi-Takezawa, Mushigame and Naraki. Case studies are also undertaken for slope failures involving the man-made deposits behind retaining walls surrounding the residential hill at Takamachi-Danchi in Nagaoka. In the first section of the present study, a simple analytical method is introduced based on the energy principle, in which the residual strength is taken up as a sole parameter to determine the run-out distance combined with the geometry of the landslides. The slope failure is herein assumed to consist of two phases, sliding and spreading, and the sliding distance is defined as the length of a slope on which the mass of soils slides down, and the run-out distance is determined as the one on a gentle slope or flat plane on which the phase of spreading occurs. Soil samples were retrieved from the sites of landslide, and laboratory triaxial tests are conducted on unsaturated soil samples with varying water contents. The residual shear strength thus obtained was used as an input parameter in the simple analysis to forecast the run-out distance. The outcome of the present study is presented in a form of simple charts in which the run-out distance is expressed as function of relevant geometrical parameters and the residual shear strength of soils involved in the landslide.

Key words: landslide, residual strength, run-out distance (IGC: D6/E6)

INTRODUCTION

On October 23, 2004, a devastating earthquake measuring a magnitude of $M = 6.8$ struck the hilly regions in south Niigata at a local time of 17:56, followed by two huge aftershocks measuring $M = 6.0$ and 6.5 at 18:11 and 18:34 on the same day. Due partly to the heavy rainfall three days preceding the earthquake, the hilly region was severely plagued during the earthquakes by the occurrence of numerous landslides, especially in the mountain area of Yamakoshi, where tertiary deposits mainly prevail. Figure 1 shows the epicentres of the main shock and the subsequent two aftershocks and also the locations where large landslides occurred. The reverse fault plane hidden locally in the direction of north-east—south west are purported to have been responsible for triggering the sequence of these earthquakes, and the areas devastated by numerous landslides were located on the side of hanging-wall of the fault. The area of Yamakoshi forms a hilly region typically with 300 to 400 m high above the sea level.

When it becomes possible to more accurately estimate the run-out distance of collapsed soil debris from expected sources of landslides, the quality of landslide hazard maps would be improved. Therefore, from the viewpoint of landslide hazard mitigation, the estimate of run-out distance would be among the greatest concerns. The issue of the post-failure run-out distance has been the subject of concern, and numerous attempts have been made to clarify the mechanism of debris flow based on case studies and analyses. Comprehensive overview and summary on the outcomes of these studies are given by Hunter and Fell (2003). In the present study, case studies are undertaken focusing on the run-out distance of the landslides, which occurred during the 2004 Niigata-ken Chuetsu Earthquake and also during other past earthquakes.

SITES OF LANDSLIDE INVESTIGATED

A number of landslides were triggered on natural slopes on both sides of the valley along Imo river. The locations of the landslides are shown in Fig. 1. As illustrated schematically in Fig. 2, the stratification of the sedimentary rocks composed of sandstone and mudstone is inclined about 15 to 20 degrees towards the west forming the out-face dipping surface, whereas the western face

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along the Imo river constitutes the in-face dipping slopes which are inclined steeper with the angle of 30 to 40 degrees. As illustrated in the inset of Fig. 2, the sandstone layer is less cemented because of the action of weathering induced in conjunction with readily seeping water as compared to the portion of less permeable mudstone. Thus, the characteristic differences prevailed in the mode of slope failures, as follows, between the landslides on the eastern and western slopes of the Imo river.

(1) On the out-facing dipping slopes on the eastern slopes, many patches of paddy fields and carp-raising ponds had been cultivated, and the majority of landslides occurred at depths of 5 to 10 metres along the interface between the sand and clay layers, leaving clear-cut slickensides exposed on the sliding surface.

(2) It is conceived that the sliding has occurred within the least cemented water-saturated sand deposits atop the mudstone. Because of the large depth and wider area of sliding, the volume of the landslides was relatively large, involving the soil mass of about 10,000 to 50,000 m$^3$. On the contrary, the landslides on the in-face dipping surface on the west side were shallow in depth on the order of 3 to 5 m, and the soil mass involved was about 1,000 to 10,000 m$^3$.

**Landslide at Higashi-Takezawa**

The Imo river flows from the north to south direction in the area of Yamakoshi. The site of landslide at Higashi-Takezawa is located at the left bank of Imo river, as shown in Fig. 1. The left bank of this river is known to form geologically the dipping slope structures, which are known to be susceptible to slip failures. The stream of this river was blocked at several locations by massive piles

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**Fig. 1.** Epicentres and locations of sites of landslide studied

**Fig. 2.** Geological features associated with landslides

*Photo 1. Bird’s eye view of landslide at Higashi-Takezawa (after Professor H. Marni)*

*Photo 2. Close view of landslide at Higashi-Takezawa (This landslide occurred on the gentle slope at the left bank of Imo river, which forms an out-facing dip structure)*
of the debris induced by the landslides during earthquakes. The upstream riverbeds at such locations were flooded and the natural water reservoirs were formed, which were found to be narrowly in danger of breach after the earthquake. The investigated landslide at Higashi-Takezawa was one of them, forming the natural reservoir behind the debris pile. Along the Imo river, the soil deposit consists of weathered sandstone of tertiary era, and this material collapsed and moved downwards to the riverbed. Numerous landslides were generated in this region in such tertiary deposits of weathered sandstone.

The bird's eye view and close view from the south of the landslide at Higashi-Takezawa are shown in Photos 1 and 2, and the plan and side views of the site of landslide at Higashi-Takezawa are shown in Fig. 3. During the reconnaissance traversing at the site, a clearly visible slip plane was observed being exposed over the headwall portion of the landslide with an angle of 20 degrees. This smooth mudstone surface had an appearance resembling what is called the slickenside. The sliding debris appeared to be composed predominantly of sand. It is most likely that due to the heavy rainfall preceding the earthquake, the groundwater had been accumulated above the less permeable surface atop the mudstone, and consequently the liquefaction-type failure might have occurred at some portions of the less cemented sand deposit, leading to the entire soil mass moving down. In the present study, the process of this kind of sliding is postulated to consist of two phases, that is, the sliding of soil mass on the slope and the spreading of debris over the field downhill.

**Landslide at Mushigame**

The location of the landslide at Mushigame is also shown in Fig. 1. The tertiary deposits of weathered mudstone are distributed over the slope, on which a number of fish ponds and paddy fields were constructed in a form of multi-shelves. Figure 4 shows the plan and side views of the site of landslide at Mushigame. The deep-seated slip failure was found near the top part of the landslide, followed by spreading of debris spilling over the road downhill. From the inspection at the site, it is not known exactly where it was originated from, though there was a trace of a grey-coloured layer of non-cemented sandstone in the debris. Therefore, the heavy rainfall might have seeped within this relatively permeable layer. Then, the collapse might have been triggered by the shaking during the earthquake. As shown later in Fig. 12, the soil sample from Mushigame can be disintegrated into aggregates. Therefore, it is most likely that the fragmentation must have played an important role in the "spreading" phase. This landslide is also characterized by the two phases of sliding and spreading.
As illustrated in Fig. 2, the right bank of this river consists of steeply dipping in-face slope structures. Surficial slips from the top of the cliff took place during the earthquake at several locations along the Imo river. The landslide at Naraki was one of such landslides. As shown in Fig. 5, the headwall of the landslide had a slope angle of over 35 degrees and spanned widely, resulting in total in the great amount of debris cascading down the valley.

**Flow Failures of Man-made Fills at Residential Hill of Takamachi-Danchi**

The residential district of Takamachi-Danchi is located on the elevated hilly terrace surrounded by the alluvial flat plain in Nagaoka City. There are several dozens of houses in this residential district. Some portions of the steep slopes surrounding the terrace about 50 m high were reinforced with retaining walls with backfills of silty sands. During the 2004 Niigata-ken Chuetsu Earthquake, the retaining walls at four portions of the hillsides had collapsed and were carried downhill together with sliding debris. At the three portions, the retaining walls were broken apart and were involved in the landslides. The above three sites were chosen for the reconnaissance and detailed investigations were conducted in the present study.

Figure 6 shows the plan and side views of the slope failure observed at site No. 2. In the present study, the process of landslides is assumed to consist of two distinctive phases, sliding and spreading. In the case of landslides on natural slopes, the first phase was characterized by the slide-down of the collapsed soils on slopes, whereas the second phase was characterized by the run-out in which the debris is involved in spreading over gently sloped fields. In the present case, the boundary

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**Landslide at Naraki**

The site of landslide at Naraki is located at the right bank of the Imo river, as shown in Fig. 1. Figure 5 shows the plan and side views of the site of landslide at Naraki.
between the two zones, sliding and spreading, is assumed to be located at the toe of a retaining wall. Figures 7 and 8 show similar sets of the plan and side views of the failures observed at sites No. 3 and 4, respectively.

SIMPLE ANALYSIS BASED ON ENERGY PRINCIPLE

General Framework of Simple Analysis

It is apparent that landslides at a fairly rapid speed pose danger and devastating effects to downslope areas. These are cited as debris flows, earth flows, rock avalanches, and flows of liquefied sands. The process of debris flow was studied and the analyses of run-out distance were undertaken in the past including the studies by Hsu (1975), Hungr (1995), Davies et al. (1999), Davies and McSaveney (2002) and Hunter and Fell (2003). The main topics focused by Davies et al. (1999) and Davies and McSaveney (2002) are the long run-out distances of rock avalanches in which the fragmentation process was considered to be a necessary mechanism to explain such long run-outs. The findings from the study of Hungr (1995) were the fact that the collapsed soil mass moves in a retrogressive manner during rapid landslides in such a manner that the front end of the collapsed soil mass experiences the coefficient of passive earth pressure while the rear end experiences active values, leading to different velocities of the movement between the front end and rear end. In the continuum model developed by Hungr (1995), the multiply aligned lumped mass flow model was used with the flow resistance terms of various rheological functions. However, various rheological properties are the necessary parameters which are often difficult to quantify. In the present study, the residual shear strength ratio is taken as a sole parameter together with the geometry of landslides, controlling the run-out distances during landslides, and on this basis a simple analytical method is introduced, as follows.

The details of the proposed model are illustrated in Fig. 9. A mass of soil expected to collapse is represented by a rectangular block located at the position “A” sitting on the slope with an angle $\alpha$. At this position, the block has a height $H_o$ and length $L_o$. When the block of soils is subjected to perturbation during seismic shaking, failure is triggered and the soil block at “A” is assumed to move downwards to position “B” until the right-hand side of the bottom face of the block reaches the boundary between the regions of sliding and spreading. This process is assumed to correspond to the sliding phase. During this sliding phase, the potential energy of position is rapidly lost, compensated with the friction energy loss induced at the bottom face of the block. It is then assumed that the block of soils at “B” now sitting on the gentle slope with an angle $\beta$ is subjected to spreading deformation, keeping the hyperbolic shape while the
length of the bottom face becomes greater. The block of soils eventually reaches position “C”. At this position, the hyperbolically shaped block has a height of \( H \) and a length of \( X_t \), as shown in Fig. 9. This model involving the hyperbolically shaped soil block was first developed by Hungr (1995) to simulate the break of dams. The process of the dam break was assumed to correspond to the spreading phase described above. In the original model by Hungr (1995), the source of available energy to be spent in the base friction was assumed to come solely from the change in the potential energy in height during spreading. However, in the model proposed herein, the available energy emerging from the change in height in

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**Fig. 8.** Plan view and side view of flow failure of reclaimed deposits at site No. 4 of Takamachi-Danchi

**Fig. 9.** Schematics illustrating the assumptions in the simple analysis

**Table 1. Summary of parameters inferred from case studies**

<table>
<thead>
<tr>
<th>Site</th>
<th>Sliding distance ( L_s ) (m)</th>
<th>Run-out distance ( L_r ) (m)</th>
<th>( H_s ) (m)</th>
<th>( L_o ) (m)</th>
<th>( \alpha ) (°)</th>
<th>( \beta ) (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Higashi-Takezawa ²</td>
<td>107</td>
<td>147</td>
<td>26</td>
<td>168</td>
<td>17</td>
<td>0²</td>
</tr>
<tr>
<td>Mushigame</td>
<td>85</td>
<td>115</td>
<td>23</td>
<td>102</td>
<td>23</td>
<td>0²</td>
</tr>
<tr>
<td>Naraki ³</td>
<td>95</td>
<td>146</td>
<td>13</td>
<td>106</td>
<td>33</td>
<td>0³</td>
</tr>
<tr>
<td>Takamachi-Danchi No. 2³</td>
<td>9.5</td>
<td>25</td>
<td>7</td>
<td>12.5</td>
<td>19</td>
<td>17</td>
</tr>
<tr>
<td>Takamachi-Danchi No. 3³</td>
<td>10</td>
<td>24.5</td>
<td>6</td>
<td>13</td>
<td>28</td>
<td>12</td>
</tr>
<tr>
<td>Takamachi-Danchi No. 4³</td>
<td>16</td>
<td>57</td>
<td>8</td>
<td>23</td>
<td>19</td>
<td>13</td>
</tr>
<tr>
<td>Tsukidate ⁴</td>
<td>40</td>
<td>102</td>
<td>6.5</td>
<td>50</td>
<td>12</td>
<td>0⁴</td>
</tr>
<tr>
<td>Kanan</td>
<td>32</td>
<td>58</td>
<td>7</td>
<td>28</td>
<td>18</td>
<td>0⁴</td>
</tr>
<tr>
<td>Kodobuki-Danchi ⁵</td>
<td>84</td>
<td>94</td>
<td>17</td>
<td>70</td>
<td>15</td>
<td>0⁵</td>
</tr>
</tbody>
</table>

² 2004 Niigata-ken Chuetsu Earthquake (October 23, 2004)
³ 2003 Miyagiken-oki Earthquake (May 26, 2003)
⁴ 2003 Miyagiken-oki Earthquake (July 26, 2003)
⁵ 1978 Miyagiken-oki Earthquake (June 12, 1978)

² The slope angles of the “spreading” region in these cases are assumed as \( \beta = 0 \).
both the phases is assumed to be spent in the base friction with equally mobilized residual shear strength in the sliding and in the spreading phases. In the following, these two phases are characterized by the sliding (initial) distance, "$L_i$", and the run-out distance, "$L_0$", as indicated in Fig. 9.

It is of interest to make comparison of these two distances with reference to the case studies shown in Figs. 3 to 8. The dimensional parameters related to the scale of landslides as indicated in Fig. 9 are inferred from the cross sections of landslides, and these are summarized in Table 1. Herein, based on the observations on the pre-failure and post-failure cross sections of the sites of landslide, the boundary between the phases of sliding and spreading was determined at the toe of the pre-failure surface. The average angle of a sliding surface $\alpha$ was then determined. The maximum depth of a run-out deposit was taken up as $H_o$, and the value of $L_o$ was determined by assuming that the total volume of a run-out deposit estimated from the cross section would be equal to the volume of the rectangle, $H_o \times L_o$. The value of $L_i$ was determined as the distance from the boundary between sliding and spreading to the centre of gravity of the original rectangle. The values of the run-out distance, $L_0$, thus inferred are plotted against the sliding distance, $L_i$, in Figs. 10 and 11. It can be seen that practically for all cases studied, the run-out distances are generally larger than the sliding distances. The data obtained from the other case studies are also included in Fig. 10, concerning the collapse of man-made fills at Kotobuki-ya in Shiroishi of Miyagi Prefecture during 1978 Miyagiken-oki Earthquake, and also rapid landslides involving loosely deposited fills on natural slopes, which occurred at Tsukidate and Kanam in Miyagi Prefecture during the two earthquakes in 2003, Sanriku-Minami Earthquake on May 26 and Miyagiken-Hokubu Earthquake on July 26.

The concept of the model proposed herein based on the energy principle is illustrated in Fig. 9. During the entire process from position "A" to "C" via "B", the energy of position, $E$, available is given as follows,

$$E = \gamma H_o L_o \Delta h,$$  \hspace{1cm} (1)

where $\gamma$ is the unit weight of the soil mass and $\Delta h$ is the difference in the height between the centres of gravity of the original rectangle "A" and of the hyperbolic shape "C". During the first phase of "sliding" with the movement of the non-deformed block of soils from the position "A" to "B", the work done by the residual shear strength mobilized along the bottom face of the block is expressed as follows;

$$W_1 = \tau_i L_o \times \frac{L_i}{\cos \alpha},$$  \hspace{1cm} (2)

where $\tau_i$ is the residual shear strength mobilized at the bottom face of the block. During the phase of spreading with the block of soils changing its shape from the rectangle "B" to the hyperbolic shape at the position "C", the volume of the soil block is assumed to remain unchanged, and therefore, the following equation is proved to hold;

$$H_o L_o = \frac{2}{3} H_i L_i.$$  \hspace{1cm} (3)

During the spreading phase, the work done by the residual shear strength mobilized along the bottom of the block is expressed with reference to the inset of Fig. 9, as follows:

$$W_2 = \int_{L_i/2}^{\sqrt{L_i^2 \cos \beta}} \tau_i x \times 2 \, dl = \frac{\tau_i}{4} \int \left( L_i \times L_2 \right)^2 - L_2^2 \right).$$  \hspace{1cm} (4)

For simplicity sake, the residual shear strength mobilized along the bottom of the block during the two phases of soil movements is assumed to be the same. Thus, by equating the energy of position available and the work done during the entire process, the following equation is obtained:
\[ E = W_1 + W_2. \] (5)

It is to be noticed here that the energy dissipated during deformation of the entire volume of the soil mass in the spreading phase is not taken into account in this proposed model.

\[
\frac{\tau_t}{\gamma H_o} = f \left( \alpha, \beta, \frac{H_o}{L_o}, L_t, L_i \right) = \left( \frac{H_o}{L_o} \right) \cos \alpha + \left( \frac{L_i}{L_o} \right) \tan \alpha + \left( \frac{L_t}{L_o} \right)^2 + \left( \frac{L_i}{L_o} \right)^3 + \frac{1}{4 \left( \frac{H_o}{L_o} \right)} \left( \frac{L_t}{L_o} \right) + \frac{1}{4 \left( \frac{H_o}{L_o} \right)} \left( \frac{L_i}{L_o} \right). \] (6)

Herein, it is to be noted that the residual shear strength ratio, \( \tau_t/\gamma H_o \), calculated from the above analysis is equal to the ratio of the residual shear strength to the effective overburden stress, \( \tau_t/\sigma' \), as follows;

\[
\frac{\tau_t}{\gamma H_o} \equiv \frac{\tau_t}{\sigma'}. \] (7)

\[
\left\{ \begin{array}{l}
\left( \frac{1}{4 \cos^2 \beta \left( \frac{H_o}{L_o} \right)} \left( \frac{L_t}{L_o} \right)^3 - \frac{2 \tan \beta}{5} \left( \frac{L_t}{L_o} \right)^2 + \left[ \frac{\tau_t}{\gamma H_o} \left( \frac{L_i}{L_o} \right) \right] \frac{1}{\cos \alpha} \left( \frac{L_i}{L_o} \right) + \frac{9 \cos^2 \beta \left( \frac{H_o}{L_o} \right)}{16} \right.
\end{array} \right. = 0. \] (8)

Evaluating Residual Shear Strength Ratio

Based on the general framework of the simple model as expressed by Eq. (5) combined with Eqs. (1) to (4), the ratio of the residual shear strength to the initial overburden stress can be expressed as follows;

It is to be noted that the run-out distance ratio, \( L_t/L_o \), is now expressed in a form of the third order algebraic equation. There are basically three solutions for \( L_t/L_o \) satisfying Eq. (8). However, the ratio of the distance, \( L_t/L_o \), needs to take a positive value and the run-out distance should become greater as the residual shear strength of soils becomes lower. It is easy to find one solution satisfying such conditions, which is the largest among the three. Therefore, the above Eq. (8) implies that given the geometry of the problem, the run-out distance ratio, \( L_t/L_o \), can be estimated from the initial distance ratio, \( L_t/L_o \), based on the residual shear strength ratio, \( \tau_t/\gamma H_o \).

SIMPLE LABORATORY TRIAXIAL TESTING METHOD

General Considerations

In estimating the run-out distance of landslides, the residual shear strength would be of key importance, which is mobilized at a largely deformed state prevailing in the vicinity of the sliding surface. Since the materials involved in the process of sliding and spreading are not necessarily saturated, it would be desirable to know the residual shear strength in non-saturated states in general. With respect to rapid landslides such as those observed during recent earthquakes, there would be little time for the sliding soil mass to change their volume in the course of rapid movement. Therefore, it would be reasonable to assume that rapid landslides would take place under conditions of little or practically no volume change. In order for a potentially contractive soil to keep its volume unchanged during shearing, the overburden stress acting initially on the soil mass needs to be reduced. In such circumstances during deformation, a portion of the initial overburden stress must be temporarily carried by other substances such as air-containing water or dust-containing air existing in the voids of flowing soil mass. Without scrutinizing this aspect, it will be simply assumed in the present test scheme that the volume of partly saturated soils be maintained almost unchanged during shearing. To achieve this condition, the initial overburden stress was reduced in the triaxial tests during the application of shear strain.

Test Apparatus and Test Procedures

The method of laboratory testing adopted for the present triaxial tests is described below (Sawada et al. 2006). The large triaxial test apparatus equipped with an inner cell was used. This apparatus can accommodate cylindrical triaxial soil specimens of 12.0 cm in diameter and 24.0 cm in height. The top of the inner cell has an open mouth with a diameter slightly larger than that of the axial rod so that even a small change in the volume of the sample inside could be monitored by up-and-down movement of the water level in the narrow annual space between the axial rod and the mouth of the inner cell.
The constancy in volume was implemented in drained loading conditions by leaving open the valve to the piezometer line. The non-saturated soil specimen was prepared with the method of wet tamping, and an equal confining stress of \( \sigma_c = 98 \text{ kPa} \) was applied to both the outer and inner cells to produce a state of consolidation, whereby leaving air in the upper part of the cell so that the water in the outer cell stays at the same level as that of the inner cell. The soil specimen was then axially loaded at a constant strain rate. Since the inner cell is subjected to an equal pressure from both inside and outside, there is no lateral deformation of the inner cell. Thus, a slight change in the water level in the mouth was considered to indicate the volume change of the specimen. The cell pressure was increased or decreased so that the water level inside and outside is maintained coincident during the constant rate axial strain application. Since the specimen generally begins to increase its volume during the early phase of shearing, it was necessary to reduce the cell pressure to keep the volume of the specimen unchanged. The cell pressure was reduced until it became equal to zero. However, the axial loading was continued until the axial strain of 10% was achieved.

### Soil Samples

The grain size distributions of the soil samples retrieved from the sites of landslide at Higashi-Takezawa, Mushigame and Naraki are shown in Fig. 12. The soil samples taken from the sites at Higashi-Takezawa and Naraki were of disintegrated sandstone. On the other hand, the soil sample from the site at Mushigame was of weathered mudstone, and was found to be rather cohesive with the plasticity index of \( I_s = 26 \). The sample as recovered from the site consists of many granules of different sizes. This soil sample was mechanically crushed to granules of approximately equal sizes of about \( D_0 = 2 \text{ mm} \), and used in triaxial tests. The grain size distributions at the mechanically crushed state and the completely crushed state are indicated in Fig. 12. The grain size distributions of the soil samples retrieved from the sites of Takamachi-Danchi No. 2 and No. 3 are shown in Fig. 13.

**Test Results**

It was found in the earlier study that amongst various factors including density and water content of soils, the residual strength of non-saturated soils is predominantly affected by the water content, (Ishihara et al., 2005; Tsukamoto and Ishihara, 2005). Based on the outcome of this earlier study, a series of the tests were conducted on the soil samples by using the procedure as described above. With an aim to examine effects of water content on the residual strength of non-saturated soils, the tests were carried out on specimens all consolidated to a confining stress of \( \sigma_c = 98 \text{ kPa} \).

The residual shear strength of soils is defined as half of the deviator stress, \( S_u = q/2 \), at a largely deformed state, and the residual shear strength ratio or normalized residual shear strength is defined as \( S_u/\sigma_c \). The values of the residual shear strength ratio thus determined are plotted against the water content in Figs. 14 to 18 for each of the soils recovered from the sites of landslide.

In the case of the soil sample from Higashi-Takezawa, it can be seen in Fig. 14 that the residual shear strength ratio takes a nearly constant value of 0.24 up to the water content of about 20%, but it is reduced sharply to a value
of 0.03 with increasing water content. In the case of the soil from Mushigame, it was difficult to prepare soil specimens with a saturation ratio in excess of $S_r = 60\%$, and the data at a water content greater than $w = 40\%$ were not obtained in the present study. However, as seen from Fig. 15, the residual shear strength ratio may be inferred by extrapolation to take values ranging from 0.32 to 0.27 at the water content of 10 to 20\%. In the case of the soil from Naraki, the residual shear strength ratio is found to change from $S_{mr}/\sigma_r' = 0.19$ to almost 0, as shown in Fig. 16. The plots of the residual shear strength ratio against water content for the soil samples taken from Takamachi-Danchi No.2 and No.3 are displayed in Figs. 17 and 18.

**COMPARISON BETWEEN SIMPLE ANALYSES AND TRIAXIAL TESTS**

**Residual Shear Strength**

The assumptions adopted in the simple analysis leading to Eqs. (6) and (8) imply that the residual shear strength ratio $\tau_r/\gamma H_0$ can be determined by the parameters related to geometry of landslides such as $L_c/L_o$, $L_t/L_o$, $H_0/L_o$, $\alpha$ and $\beta$. Therefore, by reading off the values of these parameters from each of the cross sections as listed in Table 1, it is possible to infer the value of the residual shear strength ratio from back-analysis using the relation of Eq. (6). On the other hand, the soil sample was taken from each of the sites, and the range of the residual shear strength may be determined from the data of the triaxial tests in the laboratory. Therefore, it would be of interest to make comparison of the values of the residual strength ratio thus obtained from the simple analysis and triaxial tests. Such comparison is displayed in Figs. 19 and 20. It is to be noted, however, that the residual shear strength from laboratory tests is dependent on the water content, and therefore, the water content likely to exist at the site of landslide must be estimated. It is a difficult task to determine the in-situ water content precisely and there-
fore the most likely range of water content or saturation ratio was estimated as indicated in each of the diagrams shown in Figs. 14 to 18.

In Fig. 19, the values of the residual shear strength ratio for the case studies on natural slope failures are plotted. The data for the case studies on the rapid landslides at Tsukidade and Kanan are also included. The laboratory triaxial test results on the soil sample from Tsukidade were previously reported by Ishihara et al. (2005). The range of the residual shear strength ratio changing with the water content, w, or saturation ratio, Sr, is indicated for each soil sample, and the value of the residual shear strength ratio corresponding to Sr = 70% is indicated as a dark round symbol, based on the test results shown in Figs. 14 to 16. It is to be noted here that the saturation ratio, Sr, is chosen rather than the water content, w, as a primary parameter controlling the degradation of the residual strength with increasing water content. This choice is based simply on the fact that, while the water content is widely variable depending on the grading of soils, the saturation ratio is a parameter which may be more intuitively estimated to grasp degree of non-saturated state of various soils. It is seen in Figs. 14 to 16 that it is almost in the range from Sr = 0% to 80% that the residual shear strength tends to degrade, while one cannot see any tendency in the range from Sr = 0% to 60%. It is noteworthy that the values of the residual shear strength ratio corresponding to Sr = 70% thus indicated are generally in fairly good agreement with the values inferred from the simple analysis, except for the case of Naraki. One of the reasons for this difference might be the fact that this landslide occurred on the steep slope of over 35 degrees, and the debris flowed into the Imo river and spread further up to the opposite bank of the river. It is noted in Fig. 19 that the residual shear strength ratio takes a value of 0.09 to 0.17 for soils derived from sandstone, while it takes a higher value of about 0.23 for soils derived from mudstone.

Similar comparisons for the case studies on the failures of fill deposits behind retaining walls are made in Fig. 20. It is seen that the values of the residual shear strength ratio are generally twice as much as those obtained from triaxial tests. Herein, the soil sample from the site No. 4 was not available for triaxial testing. In the light of the fact that the fill materials were practically the same between the two nearby sites, the results of the simple analysis on the site No. 4 is compared in Fig. 20 with the data obtained from the triaxial tests on the No. 3 soil sample. There are significant differences between the values back-calculated and laboratory-determined. It would appear likely that the presence of the retaining walls blocked free movement of the landslide debris, thereby shortening the distance of run-out, which would have been larger if there were no such obstacles.

Run-out Distance

Another way in which the estimate is compared would be to plot the run-out distances actually observed against those which are computed through the formula in Eq. (8), based on the residual shear strength ratio obtained from the laboratory tests. The values of the run-out distance ratio, Lr/Loo, observed in the fields for each case of the landslide are plotted against those estimated from the simple analysis in Fig. 21. The run-out distance ratios, Lr/Loo, are estimated from Eq. (8), by taking the geometrical parameters of Hoo/Loo, Lo/Loo, a and b, and by assuming the laboratory-determined residual shear strength ratio, τtr/σ′r, to be equal to that corresponding to Sr = 70%. It is of interest to see that the run-out distance ratios predicted for the cases of 2004 Niigata-Ken Chuetsu Earthquake are generally greater than those actually observed. For the cases of 2003 Miyagi-ken earthquakes, the predicted values give the run-out distances which are smaller than those actually observed.

For all practical purposes, it is of use to plot the results of the simple analysis derived from Eq. (8) in terms of the
run-out distance ratio, $L_i/L_o$, against the initial distance ratio, $L_i/L_o$. Figure 22 shows such plots for the case of Higashi-Takezawa, in which $L_o/H_o=0.155$, $\alpha=17$ degrees and $\beta=0$. The dark round point corresponds to the values of $L_i/L_o$ and $L_i/L_o$, which are observed in the cross section indicated in Fig. 3. The equal contour lines with respect to the residual shear strength ratio, $\tau_i/\gamma H_o=\tau_i/\sigma_i^*$, are also indicated in this diagram, correlating the run-out distance ratio $L_i/L_o$ with the initial distance ratio $L_i/L_o$. It is also seen in Fig. 22 how sensitive the run-out distance would be against the residual shear strength of soils. In Fig. 22, by inferring the values of the residual shear strength ratio at $S_i=60\%$, 70% and 80% from the $\tau_i/\sigma_i^*-S_i$ relation shown in Fig. 14, the equal contours for the respective values of $S_i$ are also shown. The same diagrams plotting the values of $L_i/L_o$ against the values of $L_i/L_o$ are shown in Figs. 23 and 24 for the cases of Mushigame and Naraki, respectively.

**Method of Estimating the Run-out Distance**

The estimate of the run-out distance may be made via the procedure as follows.

1. For a given slope, it is necessary to assume a potential sliding surface on which a landslide is postulated to take place. This may be made by tracing the zone of weak soil deposits within the slope, based on the geological and topographical setup of the slope under consideration.

2. Drawing upon the array of information as above, geometrical parameters such as the angles of generic portion of the landslide, $\alpha$, and of the spreading portion, $\beta$, are known. The distance of sliding, $L_i$, defined in plan view through which the mass may travel down must also be assumed in advance.

3. The residual shear strength ratio also has to be known. This may be made by testing soil samples recovered from the deposits representative of those
likely to be involved in the landslide.

(4) Having known all the parameters as above, the formula of Eq. (8) is used to determine the value of $L_t/L_o$, and hence the run-out distance $L_t$.

CONCLUSIONS

The run-out distances of landslides on natural slopes and fill deposits were examined based on the case studies observed during 2004 Niigata-ken Chuetsu Earthquake. The simple analytical method was introduced based on the energy principle, in which the residual shear strength ratio is a sole parameter to determine the run-out distance. By employing this simple analysis, the residual shear strength ratio was back-calculated based on the geometrical profile of each landslide. The laboratory triaxial tests were conducted on unsaturated soil samples retrieved from the sites of landslide. By conducting a series of tests with varying water contents, the residual shear strength ratio was inferred from the results of laboratory triaxial tests. Based on the fact that the values of the residual shear strength ratio back-calculated and laboratory-determined are compared fairly well, the chart correlating the sliding distance and run-out distance was drawn for each case of landslides, which is of practical use.

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