

Kita-Uebaru natural rock slope failure and its back analysis

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Received: 12 January 2009 / Accepted: 6 February 2010 / Published online: 10 March 2010
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Abstract A large landslide occurred in Kita-Uebaru (or Asato) area of Nakagusuku village in Okinawa Island (Japan) on 10 June 2006 after a rainy period of about 9 days. The total rainfall was 126 mm from June 8 till the time of the landslide this period. This landslide destroyed several buildings and roads, and the total travel distance of the landslide was about 110 m. In this article, the authors were concerned with the initiation conditions of Kita-Uebaru landslide and post-failure motions of the landslide body. The observations made in the landslide area, structural geology analyses and outcomes of geotechnical investigations are described first. Then, possible causes of the landslide are investigated through back analyses using as inputs the geological structure and the strength properties of planes of discontinuities involved in the sliding processes. The final part of the article is concerned with the simulation of post-failure motions of the landslide body. The results of the investigations and back analyses indicate that the failure plane was bi-planar and the heavy torrential rainy period for about 3 days was the main cause of initiation of the landslide. The mechanical model presented in this article was capable of capturing the overall features of the landslide body following the initiation of the failure.

Keywords Landslide · Rainfall · Post-failure motion · Limiting equilibrium analysis

Introduction

The potential for instability of natural and man-made slopes depends on geological, hydrogeological, geographical, geotechnical, environmental and seismological factors (Aydan et al. 1992, 2006, 2008, 2009a). Large landslides are often complex, and they may pass from a slump to a flow (Ulusay et al. 2007). Therefore, the estimation of landslides and their post-failure motions is of great importance to investigate the regional landslide risk and its impact on structures. The problem is essentially complex due to dynamic variations of the characteristics of the landslide body as well as the geometry of the basal slip plane. Most of landslides in Ryukyu Islands are associated with the Shimajiri mudstone formation. The natural slope landslides are generally caused by the heavy rainfalls. However, high-slope cuts in this formation also induced landslides. This formation is generally overlaid by the Ryukyu limestone formation. While rock slopes of Ryukyu limestone formation are generally stable, rockfalls may be caused by the erosion of underlying soft Shimajiri mudstone formation resulting in overhanging slopes. Similar failures may also be induced along the steep sea cliffs of Ryukyu Islands. A large landslide occurred in the Kita-Uebaru (or Asato) area of Nakagusuku village in Okinawa Island on 10 June 2006 after a rainy period from 8 to 10 June 2006 (Fig. 1). The total rainfall was 126 mm during this period according to the data from Goya meteorological station of the Japan Meteorological Agency (JMA) (2006). The landslide destroyed several buildings and Prefectural Road No. 35, and the total travel distance of the landslide was about 110 m (Gibo et al. 2006).

The authors first describe the observations made in the landslide area and the structural geology analyses. Then, geotechnical investigations performed on in situ outcrops

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Fig. 1 An aerial view of landslide area (from Okinawa Prefectural Government)



of failure planes involved in the landslide and the results of laboratory tests on collected rock samples are presented. A series of back analyses were applied to these samples to investigate the possible causes of the landslide with the considerations of geological structure and strength properties of planes of discontinuities involved in the landslide. In the final part of the article, a mechanical model for the simulation of post-failure motions of the landslide body is presented, and it is applied to the landslide with the purpose of simulating the post-failure geometry and its evolution in time.

Geology and meteorological observations

Ryukyu Islands are aligned parallel to the Ryukyu Trough. The tectonic movements associated with the formation of the Ryukyu Trough greatly affect the structural geology of the islands. The general trend of anticlines is NE-SW, and the area is disturbed by two conjugate faults whose strikes are NE-SW dipping to SE.

The inclination of bedding plane varies from place to place. The observations by the authors in the middle of the landslide zone indicated that bedding planes dip towards SE with an inclination ranging between 8 and 12 degrees. This observation is different from that made by Gibo et al. (2006). However, the data from Japan Geology Survey (2006) are in accordance with the observations of the authors. Figure 2 shows lower hemisphere stereo projection of the average orientations of discontinuities such as faults, bedding plane and natural slope.

The rock of the landslide area mainly consists of the Shimajiri mudstone group. Although the Shimajiri mudstone group is regarded as a mudstone group, it contains porous weakly cemented sandstone bands and siltstone. Furthermore, Shimajiri mudstone is prone to rapid

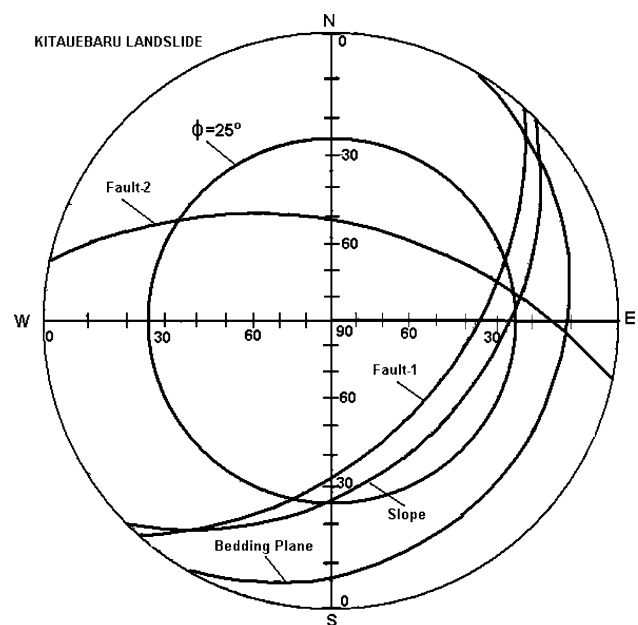


Fig. 2 Lower hemisphere equal angle stereo projection of discontinuities with a friction angle cone of 25 degrees

weathering due to its sensitivity to water absorption and desorption.

The landslide occurred in Kita-Uebaru area of Nakagusuku village in Okinawa Island on 10 June 2006 after a rainy period from 8 to 10 June 2006. The total rainfall was 126 mm during this period (JMA) (Fig. 3). This landslide destroyed several buildings and roads including Route 35 (Prefectural Road), and the travel distance of the landslide was about 110 m (Gibo et al. 2006).

First cracks were observed at 10 a.m. on 10 June 2006, while the main failure occurred at 4 p.m. on the same day. The prefectural road No. 35 was displaced for more than 100 m (Fig. 4). The main landslide surface is constituted by SE30-56/43-44 fault, while NE2-12/48-52 fault is another

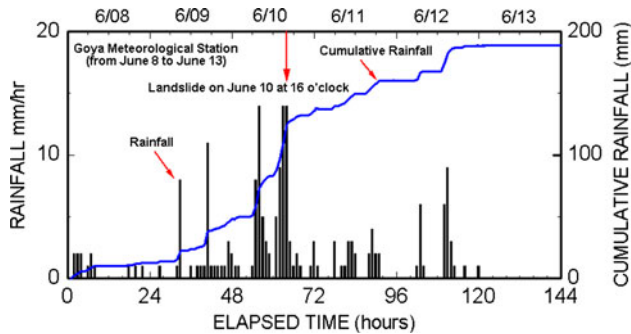


Fig. 3 Variation of rainfall in between 8 June 2006 and 13 June 2006

secondary bounding surface for the landslide zone. The secondary landslide associated with this fault. Although it is generally difficult to obtain the exact geometry of the landslide body, the landslide of Kita-Uebaru body slid over a sliding surface consisted of the fault plane (SE45/45) in the upper section and a relatively flat failure plane in the lower part, which is the bedding plane in view of observations by the authors (Fig. 5).

The Civil and Architectural Engineering Division of Okinawa Prefectural Government installed several instruments to monitor the behavior of the mobilized landslide body. The instrumentation involves the monitoring of water level, surfacial ground deformation through wire sensors and internal ground deformation by pipe-strain gauges (Fig. 4). Although some ground deformations occurred in relation to rainfalls, it is presently stable.

Geotechnical investigations

Compression tests were carried out on mudstone and sandstone samples from the Kita-Uebaru landslide area (Table 1). The sandstone has a very low strength ranging between 300 and 340 kPa, and it is very porous. The mudstone samples have much higher strength ranging between 1.2 and 3.6 MPa under natural water content conditions. This result is quite similar to previous test results by the authors on mudstone soaked in water for about 10 years.

Several institutes carried out several shear tests on undisturbed and disturbed samples of Shimajiri mudstone (Gibo et al. 2006). Peak cohesion ranged between 70 and 100 kPa, while the peak friction angle of undisturbed samples ranged between 23 and 27 degrees. Residual cohesion was nil, and residual friction angle was 11 degrees. Test results by Yamaguchi University indicated that peak friction ranged between 15 and 29 degrees, while the residual friction angle varied between 9 and 13 degrees.

The authors also performed some tilting tests on bedding planes by using dry samples on site, and tests indicated that

the friction angle ranged between 34 and 38 degrees. Tilting tests on saw-cut surfaces in laboratory ranged between 24 and 28 degrees (Table 2).

The Shimajiri mudstone is an impermeable rock with an average hydraulic conductivity of 2×10^{-6} cm/s. The highly permeable Ryukyu limestone with a hydraulic conductivity of 3.5×10^{-1} cm/s forms the aquifer with a thickness of between 10 and 70 m over Shimajiri mudstone. The effective porosity of the Ryukyu limestone is estimated to range between 10 and 15 percent (Tokashiki and Aydan 2003). Tectonic movements accompanied by several faults have formed underground conduits, and groundwater flows along these conduits.

Back analyses

As the landslide occurred after a period of heavy rainfalls, the variation of water pressure is most likely to be one of the main causes of the landslide of Nakagusuku Kita-Uebaru. As mentioned in the previous section, the landslide body geometry shown in Fig. 5 was selected on the basis of our own in situ observations as well as reports by Gibo et al. (2006) and the Civil Engineering and Architectural Division of Okinawa Prefectural Government (2006) and Japan Geographical Surveying Institute (2006). The friction of bedding plane of Shimajiri mudstone is more than 34 degrees under dry state obtained from tilting tests in situ. Since the mechanical properties of Shimajiri mudstone decrease with the increase of water content, the friction angle may decrease with water content besides the effective stress decrease due to pore water pressure. Both bedding and fault surfaces probably have no cohesion. Simple (Fellenius type), Bishop, Spencer and Aydan limit equilibrium methods (i.e., Aydan et al. 1996, 1997) are used for static stability analyses. In the analyses, the cohesion was assumed to be nil. First, the landslide body was assumed to be fully saturated, while the friction angle varied between 25° and 35°. Results of computations are shown in Fig. 6a.

Both simple and Bishop methods imply that the landslide body is at the critical state for the friction angle under dry state. However, Aydan and Spencer methods imply that the friction angle should decrease in order to cause the failure.

Next, the pore pressure was increased while the friction angle was kept at 25°. The safety factors computed are shown in Fig. 6b. As expected, the simple method and Bishop method indicate that the pore water pressure coefficient should be 0.7 to initiate the landslide, while the method of Aydan requires that the water pressure coefficient should be 0.93, which is the most likely situation for a rainy period of 3 days. Although it may be still speculative, the reduction of friction angle and increase of water pressure

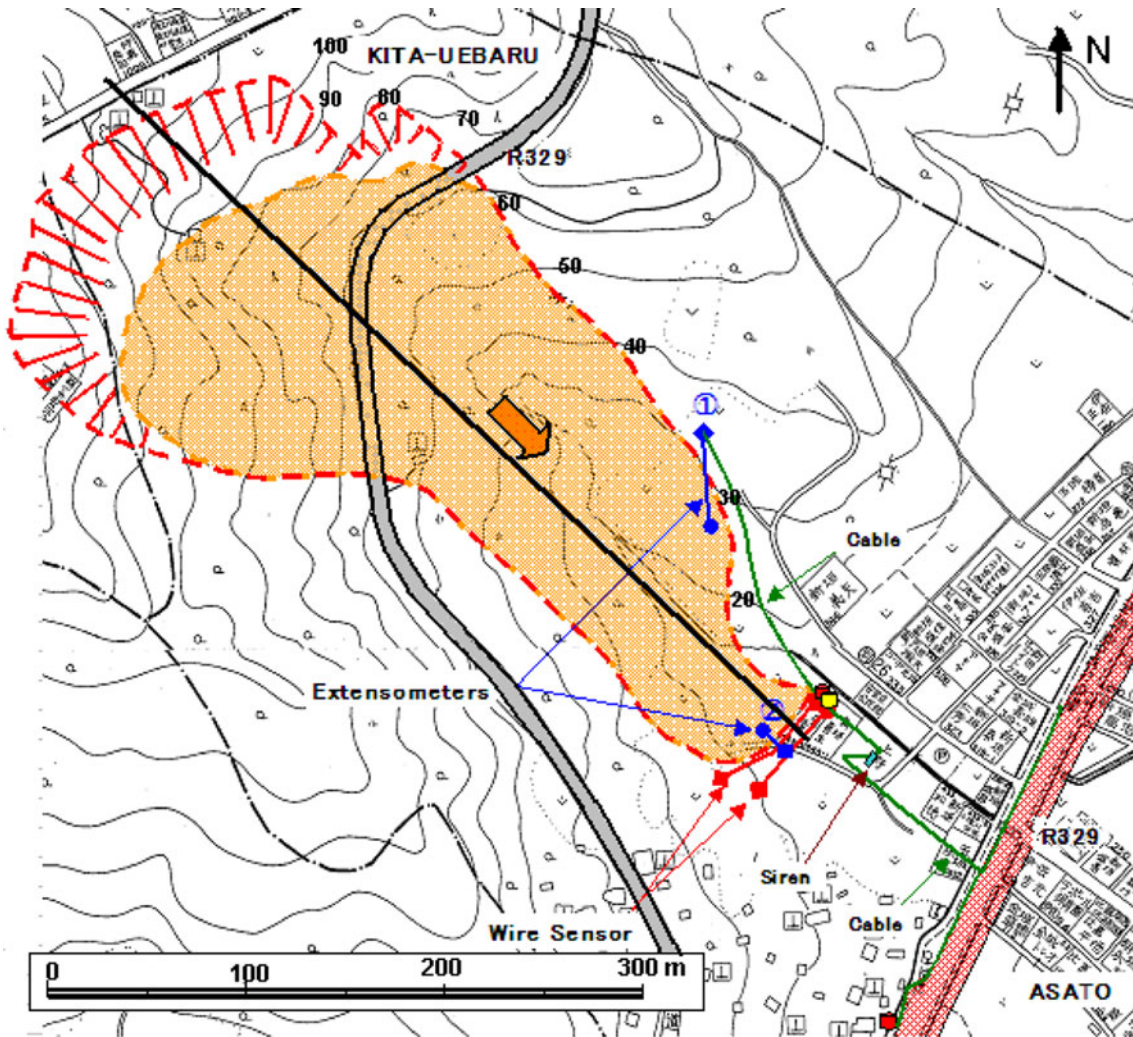


Fig. 4 Topographical map of landslide area (modified from the original drawing by Civil Engineering and Architectural Division of Okinawa Prefectural Government in Japanese)

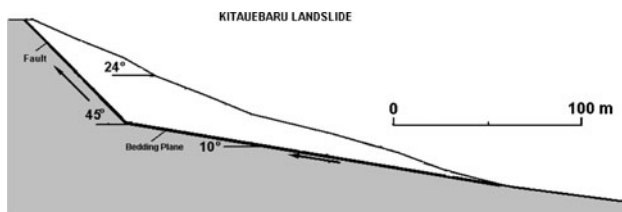


Fig. 5 Geometry of the landslide body along the line shown in Fig. 4

Table 2 Frictional properties of discontinuities

Interface	Friction angle
Sandstone to sandstone (sawcut)	34–38
Mudstone to mudstone	24–28
Sandstone to mudstone	32–36
Mudstone natural	28

Table 1 Physical and mechanical properties of Shimajiri mudstone group

Sample name	Unit weight (kN/m ³)	Water content (%)	Uniaxial strength (kPa)
Sandstone S1	17.57	23.4	340
Sandstone S2	22.64	28.7	343
Mudstone S1	18.51	12.0	1,240
Mudstone S2	14.68	10.2	3,640

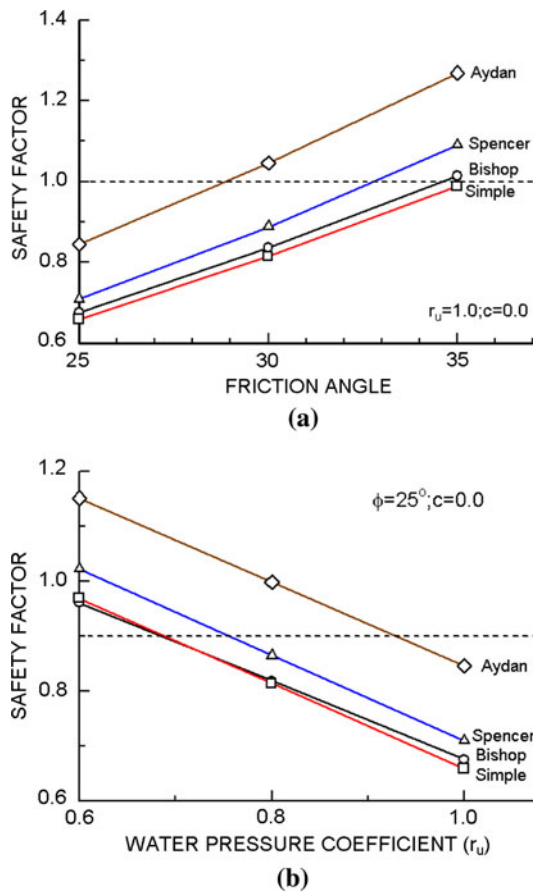


Fig. 6 Computed safety factors by different methods for different conditions. (a) Variation of computed safety factors as a function of friction angle. (b) Variation of computed safety factors as a function of pore water coefficient

coefficient should be the main factors to cause this landslide.

Recently, Nakamura et al. (2006) computed that the slopes of Shimajiri mudstone would be unstable for a residual friction angle of 13.8 degrees for fully saturated conditions. However, this value may only be valid for static stability condition against sliding.

Estimation of post-failure motions

The so-called sledge model is one of commonly used methods to estimate the travel distance of landslides (i.e., Voight 1988), and it was advanced to include the effects of static and dynamic pore pressure, earthquake forces together, with the use of Bingham-type yield criterion (Aydan and Ulusay 2002; Aydan 2003; Aydan et al. 2005, 2006). If the travel distance is short, the estimations may be somewhat different than the actual situation. In this article, an advanced model is proposed to consider a general slip

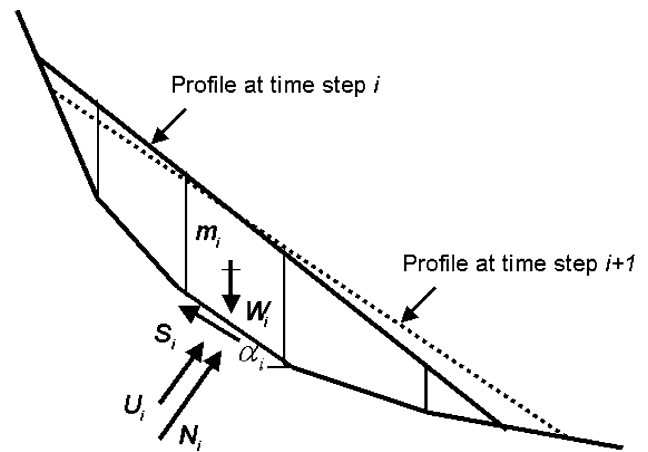


Fig. 7 Illustration of force system for a typical slice of landslide body

surface, and it is applied to the particular case of Kita-Uebaru landslide. The computational results are compared with actual landslide geometry.

Mechanical model for estimating post-failure motions

Let us consider a landslide body consisting of N number of blocks sliding on a slip surface as shown in Fig. 7. Furthermore, the slope surface is assumed to remain planar. If inter-slice forces are assumed to be nil as assumed in the simple sliding model (Fellenius type), one may write the following equation of motion for the sliding body:

$$\sum_{i=1}^2 (S_i - T_i) = \bar{m} \frac{d^2s}{dt^2} \tag{1}$$

where \bar{m}, s, t, S_i and T_i are total mass, travel distance, time, shear force and shear resistance, respectively. Shear force and shear resistance may be given in the following forms together with Bingham-type yield criterion:

$$S_i = W_i \sin \alpha_i, T_i = c_i A_i + (N_i - U_i) \tan \varphi_i + \eta W_i \left(\frac{ds_i}{dt} \right)^b \tag{2}$$

where $W_i, A_i, N_i, U_i, \alpha_i, c_i, \varphi_i, \eta$ and b are weight, basal area, normal force, uplift pore water force, basal inclination, cohesion, friction angle of slice i , Bingham type viscosity and empirical coefficient, respectively. If normal force and pore water uplift force related to the weight of each block as given below,

$$N_i = W_i \cos \alpha_i, U_i = r_u W_i \tag{3}$$

one can easily derive the following equation with the use of Eqs. 1–3:

$$\frac{d^2s}{dt^2} + \eta \left(\frac{ds}{dt} \right)^b - B(t) = 0 \quad (4)$$

where

$$B(t) = \frac{g}{\bar{m}} \left(\sum_{i=1}^2 m_i (\sin \alpha_i - (\cos \alpha_i - r_u) \tan \varphi_i) + \frac{c_i A_i}{g} \right) \quad (5)$$

In the derivation of Eq. 4, the viscous resistance of shear plane of each block is related to the overall viscous resistance in the following form:

$$\eta \bar{m} g \left(\frac{ds}{dt} \right)^b = \sum_{i=1}^2 \eta m_i g \left(\frac{ds_i}{dt} \right)^b \quad (6)$$

Equation 4 can be solved for the following initial conditions together with the definition of the geometry of basal slip plane.

$$\text{At time } t = t_o, s = s_o \text{ and } v = v_o \quad (7)$$

There may be different forms of constitutive laws for the slip surface (i.e., Aydan et al. 2006, 2008; Aydan and Ulusay 2002). The simplest model to implement would be elastic-brittle plastic. If this model is adopted, the cohesion will exist at the start of motion, and it will disappear thereafter. Therefore, cohesion component introduced in Eq. 5 may be taken as nil as soon as the motion starts. Thereafter, the shear resistance will consist of mainly frictional component together with some viscous resistance.

Numerical solution procedure

The geometry of the moving and resisting masses are incrementally computed, and resulting dynamic force conditions are modified simultaneously by assuming that no dilation of sliding mass occurs during motion. In other words, the volume of the landslide mass is preserved during motion. In the computation, the total mass and the mass of each block are first computed. Then, the incremental displacement of the mass center is computed, and the mass of each block is modified so that it is preserved by assuming that the slope surface remains planar. Following the updating of the geometry of blocks, the acceleration, velocity and travel distance of the the total mass are computed. This procedure is continued until the velocity of the landslide body becomes nil or the assumptions are violated.

Comparisons and discussions

An analysis for the post-failure motion of the landslide body of Kita-Uebaru using the model is described in this sub-section. The material properties and the geometry of

the sliding body were the same as those used in the analyses of previous section. The sliding surface consisted of the fault plane and bedding plane emanating from the toe of the slope as illustrated in Fig. 5. As the inclination of the natural ground in the sliding direction was almost equal to the bedding plane inclination, the inclination of the failure surface was assumed to be the same as that of the bedding plane. The value of the pore pressure coefficient was assumed to be 1.0 by considering the results of the analyses presented in previous section. The friction angle was taken as 25 degrees. Figure 8 shows the evolution of motion of the toe of the sliding mass with time. The computed displacement was 109.87 m. This value is very close to that reported by Gibo et al. (2006). Figure 9 shows the deformed geometry of the landslide body at selected time steps. As can be seen from Fig. 9, gradual flattening of the landslide body takes place and the final struture is very close to the actual post-failure geometry. It is also important to notice that the residual friction angle of the landslide body used in the analysis is much higher than the one computed by Gibo et al. (2006) and Nakamura et al. (2006). In other words, the static back analyses based on

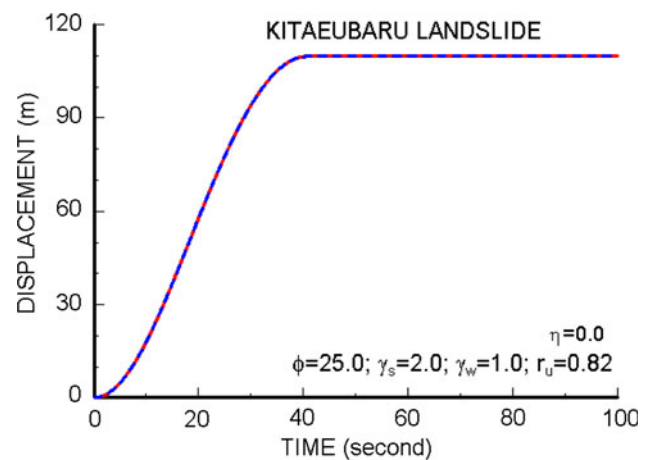


Fig. 8 Displacement of Kita-Uebaru landslide body during sliding

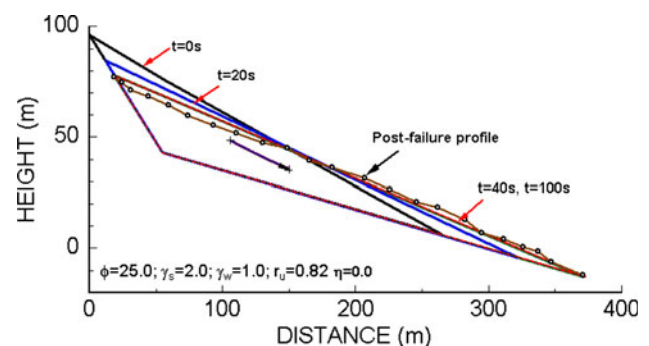


Fig. 9 Deformed configuration of Kita-Uebaru landslide body at different time steps

post-failure configurations yield extremely small residual strength properties as the dynamic characteristics of the landslide body are not taken into account. It is very important to consider the dynamics of post-failure motions for estimating actual residual strength properties of the landslide body and their possible movement in space. Furthermore, the impacts of post-failure motions can be of particularly great importance in urbanized areas (i.e., Ulusay et al. 2007; Aydan et al. 2009b; Aydan 2009; Aydan and Shimizu 1993).

Conclusions

The Kitauebaru landslide was caused by heavy rainfall. Besides the gravitational effects, there is no doubt that the increase of pore water pressure is the main mechanism to trigger this landslide. Since mechanical properties of Shimajiri mudstone are strongly influenced by the water content, the reduction of cohesion and friction coefficient may be expected. Since the inferred landslide surface consists of bedding plane and an existing normal fault surface, the cohesion should be nil and friction angle would be the main source of shear strength. The stability analyses indicate that the friction angle must be less than 35° , and the pore pressure coefficient should be more than 0.6. The pre-existing discontinuities such as faults strongly contributed to the initiation of this landslide. Therefore, it is suggested to take into account the structural geological features for the landslide risk assessment studies.

The authors presented a simple and efficient method for modeling the post-failure motions of landslides. In this method, the pore pressure variation, gravitational effects are considered and a Bingham-type yield criterion is adopted. The method proposed was capable of simulating the post-failure configuration and the time evolution of the landslide body. Furthermore, post-failure analyses implied that the actual strength properties of ground may be much higher than those inferred from simple static residual strength analyses.

Acknowledgments The authors would like to sincerely acknowledge the Civil and Architectural Engineering Division of Okinawa Prefectural Government for the information on the Kita-Uebaru landslide and aerial photographs and topographic map of the landslide body.

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