京都大学防災研究所年報 第43号 B-1 平成12年4月
Annuals of Disas. Prev. Res. Inst., Kyoto Univ., No. 43 B-1, 2000

A Modified Geotechnical Simulation Model for the Areal Prediction of Landslide Motion

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Synopsis

Based on the previous work of geotechnical simulation on the motion of landslides conducted by Sassa (1988), a modification is made on the shear resistance item and the geotechnical simulation model is improved. The landslides are considered to be with two-layer structure, that is, debris layer and sliding zone. After sliding for a certain distance, the soil at sliding zone will reach its steady state, and the shear resistance will mobilize to the minimum value. In the sliding process, the thickness of the debris layer always changes its thickness, and results in the change of the apparent friction angle. According to this consideration, change model of apparent friction coefficient is built, and is applied to Sassa's simulation model. Finally, an application study is carried out on the Sumikawa landslide occurred in Akita Prefecture in 1997, and the simulation results of areal prediction shows a good correspondence to the deposit distribution of the actual landslide.

Keywords: mobilized apparent friction coefficient, thickness change, accumulation possibility of excess pore pressure, undrained loading, steady state

1. Introduction

As a natural phenomenon, landslide always causes tragic consequence, including untold numbers of deaths/injures and huge economic losses. Especially, in recent years, with the residential and industrial developments expanding into unstable hillside area under the pressures of growing population, the losses due to landslides are apparently growing. For prevention of the landslide disasters, areal prediction of landslide motion becomes more and more important. Landslide motion prediction is considered to be a composite work; its development is always based on the understanding of mechanism of landslide motion. Hungr (1995) classified the available simulation models for landslide motion to two categories: lumped mass models idealizing the motion of a slide as a single point (Koerner, 1976; Perla et al., 1980; Hutchinson, 1986), and models based on continuum mechanics (Sassa, 1988). The friction of the sliding surface takes the most important role in both categories models. The most famous lumped mass model, perhaps the first model to interpret the landslide motion is the "sled model", which was originally proposed by Heim, 1932 (introduced by Korner, 1980). In this model, it is assumed that all energy loss during landslide motion is caused by friction. The average friction during the motion has been called as the equivalent coefficient of friction μe (Hsu, 1975), the average coefficient of friction f (Scheidegger, 1973), and the average friction angle has been called as the apparent friction angle ϕ_a (Sassa, 1988), and travel angle α (Cruden & Varnes, 1994).

Sassa (1985) has put forward that the apparent friction angle is chiefly a combination result of the real internal friction angle ϕ_a and the pore pressure during motion. He suggested that the apparent friction angle ϕ_a should be approximately expressed by Equation (1).

$$\tan\phi_a = \frac{\sigma - u}{\sigma}\tan\phi_m \quad (1)$$

ϕ_a: apparent friction angle,
ϕ_m: internal friction angle during motion,
σ: normal stress,
u: pore pressure.

In the same direction as Sassa, Hutchinson (1986) proposed a consolidated model for motion of flow slides. When a landslide mass having an excess pore pressure moves on an impermeable and oneroded ground, the excess pore pressure gradually decreases due to consolidation of the mass, while the friction during motion increases.

Based on the concept of the apparent friction angle, Sassa (1988) presented a geotechnical model for the motion of landslides. This is a quasi-three-dimensional frictional model. Its motion equation was deduced from equilibrium of forces acting on a soil column, and the continuum equation was deduced from fluid dynamics. Through fixing a frame (column of Fig.1), the motion of the sliding mass was examined. It is convenient to be understood, and has been widely used in the landslide simulation for areal prediction of landslides (Zhang and Sassa, 1997; Hong, 1997).

Hungr (1995) developed a continuum model to simulate the rapid slides, debris flows, and avalanches. His model is based on a Lagrangian solution of the equations of motion and allows the selection of a variety of material rheologies, which can vary along the slide path or within the sliding mass. It also allows for the internal rigidity of relatively coherent slide debris moving on a thin liquefied basal layer.

Varnes (1978) classified landslides in five types: fall, topple, slide, spread and flow. The two types of slide and flow are the research subjects in this paper. They are the landslide types with sliding surface. This paper will improve Sassa's model (1988) with some consideration on the apparent friction angle. It is assumed that the sliding mass composes of two layers. That is, a relatively coherent slide debris layer moving on a thin sliding zone (like Hungr's model). For the sliding zone, it can liquefy or not, depending on its structure, saturated condition, grain crushing susceptibility and so on. A basic assumption is that it will reach its steady state after sliding for a certain distance. This phenomenon is well observed in our current researches by means of

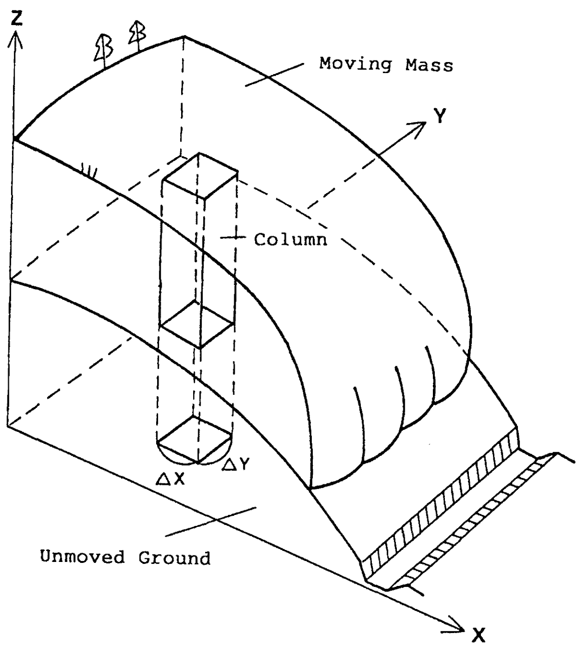

Fig. 1 A moving landslide and a column in it (from Sassa, 1988)

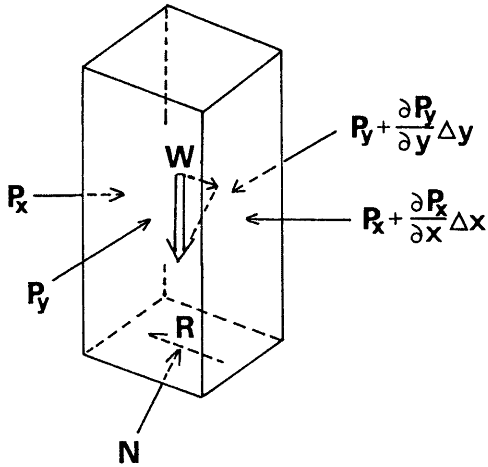

Fig. 2 Forces acting on a column (from Sassa, 1988)

undrained ring shear apparatus (Sassa, 1998; Wang, 1998; Wang et al., 2000). For the upper debris layer, the thickness will change during the motion like fluid. It is a common phenomenon that the thickness of landslide mass becomes thinner during landslide motion, and finally landslide stops and deposits. This means that the normal stress acting on the sliding zone will decrease. It then results in the corresponding increasing of the apparent friction coefficient.

In the other hand, the undrained degree in the sliding path will be determined according to the drained condition in the sliding path. This consideration is similar with Hutchinson (1986) and Sassa (1988). In Sassa (1988), it is presented a sketch model to show the pore pressure generation possibility in three types of slope conditions. Here, we use a parameter, namely, "accumulation possibility of excess pore pressure", to quantify the drained and saturated condition in the sliding path of landslide.

The organization of this paper is as follows. The fundamental equation in the model for the motion of landslides deduced by Sassa (1988) will be first introduced. The "change model of apparent friction coefficient" will be then presented, focusing on the effect of the thickness change on the apparent friction coefficient. This will be followed by an application in the Sumikawa landslide, which occurred on May 11, 1997 in Akita Prefecture Japan. The reason to use this

$$\frac{\partial M}{\partial t}+\frac{\partial}{\partial x}(u_0M)+\frac{\partial}{\partial y}(v_0M)=gh\frac{\tan\alpha}{q+1}-kgh\frac{\partial h}{\partial x}-\frac{g}{(q+1)^{1/2}}\cdot\frac{u_0}{(u_0^2+v_0^2+w_0^2)^{1/2}}\{h_c(q+1)+h\tan\phi\} \qquad (2\text{-}1)$$

$$\frac{\partial N}{\partial t}+\frac{\partial}{\partial x}(u_0N)+\frac{\partial}{\partial y}(v_0N)=gh\frac{\tan\beta}{q+1}-kgh\frac{\partial h}{\partial y}-\frac{g}{(q+1)^{1/2}}\cdot\frac{v_0}{(u_0^2+v_0^2+w_0^2)^{1/2}}\{h_c(q+1)+h\tan\phi\} \qquad (2\text{-}2)$$

$$\frac{\partial h}{\partial t}+\frac{\partial M}{\partial x}+\frac{\partial N}{\partial y}=0 \qquad (3)$$

h: depth of the sliding mass,
M, *N*: Discharge in x and y direction per unit width, respectively ($M=u_0h, N=v_0h$),
k: lateral pressure ratio,
$\tan\phi$: apparent friction coefficient of the soil in the sliding zone,
hc: cohesion head (defined as cohesion $c=\rho g h_c$, it is zero after sliding for a long distance. ρ: density),
$\tan\alpha$, $\tan\beta$: inclination of the intersection between the original slope surface and the x-z plane, y-z plane, respectively,
$q=\tan^2\alpha+\tan^2\beta$, $w_0=-(u_0\tan\alpha+v_0\tan\beta)$
(from Sassa, 1988)

landslide as an example is that this landslide is very complicated, and after landslide occurred, most of the sliding mass in the upper part moved for a small distance, and the sliding mass in the front part loaded on the torrent deposit in the Sumikawa landslide, and moved down along the Akagawa river for about 3.3 km (Yanagisawa and Umemura, 1999).

2. The fundamental equation

The fundamental equation was deduced by Sassa (1988). Fig. 1 illustrates a moving landslide and a column in it. Fig. 2 represents a column and forces acting on it. Through examining the motion of a sliding mass with a frame fixed in a space, the equation of motion (Eq. (2-1, 2-2)) and equation of continuity (Eq. (3)) were obtained.

3. A supposed change model of apparent friction coefficient

The change of the apparent friction coefficient in the sliding zone is based on a two-layer structure model of the sliding mass. Fig.3 illustrates a concept model of a sliding mass composing of two layers. The debris layer (upper layer) changes its thickness during sliding process. The sliding zone will reach its steady state after slide for a certain sliding distance.

Fig. 4 presents the change of apparent friction coefficient in the sliding zone (lower layer) in a diagram of stress path. The shear behavior of soils in the sliding zone can be commonly categorized into three types.

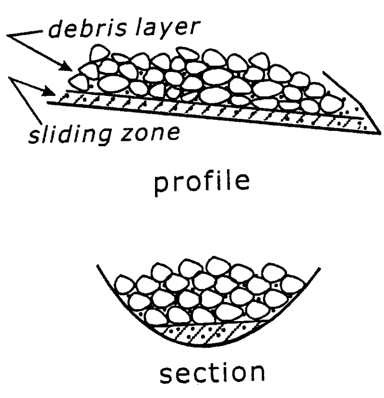

Fig. 3 Illustration of two-layer structure of landslide mass

A-type: no pore pressure built-up type. A typical case is dry soil layer landslide. The shear resistance at the steady state in this type is the drained residual strength, while the apparent friction angle is the residual friction angle.

B-type: completely liquefied type. This is an ideal saturated undrained condition, always corresponding to the rapid long run-out landslides. In this case, the pore pressure can be built-up as large as possible (Δu_{ss}), and the dissipation speed of the pore pressure at the sliding zone is so small that the shearing process in the sliding zone can be treated as in the undrained condition.

C-type: medium type. Many actual landslides belong to this type. It is likely to name this type as the "partially saturated and drained type". In this type, the built-up pore pressure depends on the comparison of generation rate and dissipation rate of pore pressure.

We can say, A-type is an extreme case of the C-type, when the dissipation speed of pore pressure is much greater than the generation speed of the pore pressure, or in a dry landslide case; while B-type is an extreme case when the motion of sliding under a completely undrained condition.

To quantify the different types in the traveling path of an actual landslide, a parameter *Bss*, named as "accumulation possibility of excess pore pressure", is proposed. This parameter is mainly decided by the properties of the soils in the sliding zone, existing situation of groundwater, drainage condition and so on. This parameter is an empirical index, and a suggestion range of *Bss* value is presented in Table 1.

Table 1 Suggest value range for *Bss*.

Type and Bss	Description
A-type *Bss* = 0-0.1	unsaturated sliding mass moving on dry surface;
B-type *Bss* = 0.9-1.0	saturated or unsaturated sliding mass moving on fully saturated surface; Saturated sliding mass moving on impermeable surface (concrete channel)
C-type *Bss* = 0.1-0.9	saturated sliding mass moving on dry and permeable surface (dry sand ground)

The other two important parameters in Fig.4, τ_{ss}, shear resistance at the steady state in the completely undrained condition and ϕ, the effective residual friction angle, can be measured conveniently in

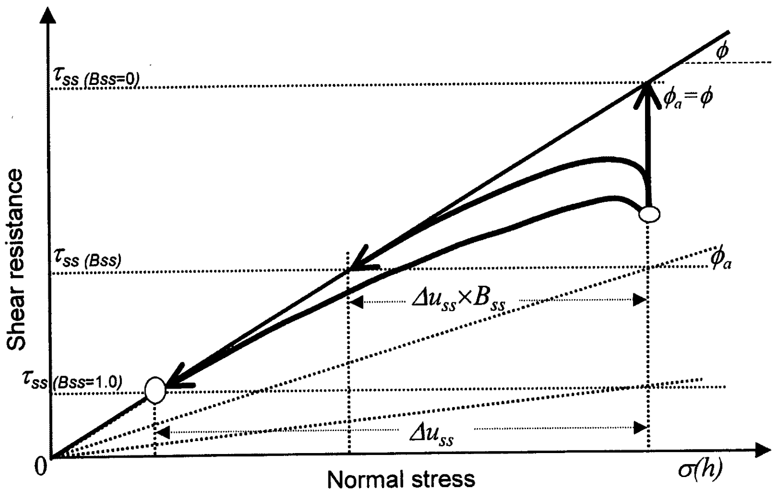

Fig.4 The change model of apparent friction coefficient

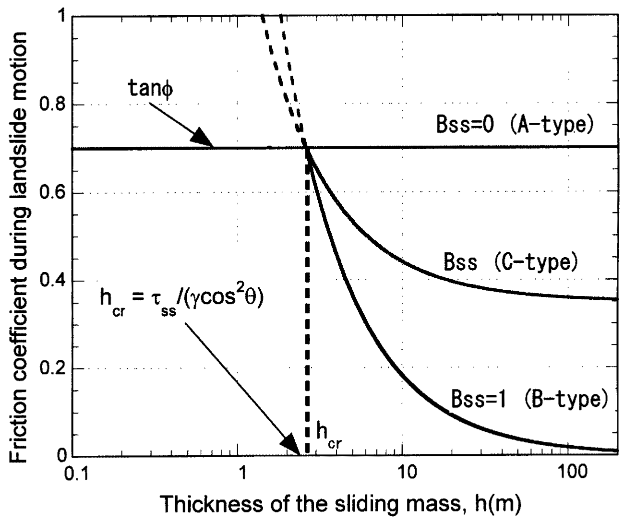

Fig. 5 The change of friction coefficient during landslide motion with thickness of sliding mass and *Bss*

laboratory. In the Disaster Prevention Research University, Kyoto University, we measure these two parameters by means of the undrained ring-shear apparatus (Sassa, 1997; 1998). Because there is no limit in the shear displacement and it can keep the shear box under the undrained condition, the ring-shear apparatus is an effective tool to monitor the stress path of soils, and to measure the shear resistance in the steady state.

After the measurement of τ_{ss} and ϕ, and an estimation of the B_{ss} value, the shear resistance at a certain state (h, B_{ss}) can be obtained by Eq. (4), while normal stress can be obtained by Eq. (5), respectively.

$$\tau_{ss}(h, B_{ss}) = \tau_{ss} + (\sigma(h)\tan\phi - \tau_{ss})(1 - B_{ss}) \tag{4}$$

$$\sigma(h) = \gamma h \cos^2\theta \tag{5}$$

where, θ: the slope angle; γ: the natural unit weight of the debris soil layer; h: the thickness of the sliding mass.

Then, the apparent friction coefficient $\tan\phi a$ can be obtained by Eq. (6). Fig. 5 shows the change of the apparent friction coefficient during landslide motion with the change of thickness and Bss.

$$\tan\phi_a = \frac{\tau_{ss}(h, B_{ss})}{\sigma(h)} \tag{6}$$

In Fig. 5, three curves of different Bss representing the three types of landslides (A, B, C) cross at "D" point. From the right side to the left, the apparent friction coefficient increases with the decrease of the thickness. The apparent friction coefficient is smaller when the value of Bss is larger. While, when it goes beyond the "D" point to the left, the apparent friction coefficient becomes greater than $\tan\phi$. It means that, after the upper debris layer becomes thinner than a critical thickness, h_{cr} (defined by Eq. (7)), suction will take effect. In actual case, it is almost impossible to keep suction in the sliding zone, so we assume that the apparent friction coefficient equals to $\tan\phi$ when h is smaller than h_{cr}, as presented in Eq. (8).

$$h_{cr} = \tau_{ss} / \gamma\cos^2\theta \tag{7}$$

$$\tan\phi_a = \tan\phi \text{, when } h \le h_{cr} \tag{8}$$

Considering the change of friction coefficient during motion resulted from the thickness change of debris layer, this model can simulate the motion and deposit process of landslides effectively.

4. Application to the Sumikawa landslide

It is important for prediction of landslide motion to express how far does it move, and how widely does it spread (Sassa 1998). After changing the fundament equations to the forms of difference equations, Sassa wrote a computer program and carried out areal simulation on landslide motion of the Ontake debris avalanche.

The modification of friction coefficient item is made on Sassa's program, and a simulation is conducted on the Sumikawa landslide in this paper.

The Sumikawa landslide occurred at the Sumikawa Spa, Hachimantai, Akita Prefecture on May 11, 1997. The failed sliding soil mass destroyed the buildings of the Sumikawa Spa completely at first, then the debris at the tongue of the lower part flowed into the Akagawa river, and caused severe damage in the downstream area. The Akagawa Spa, located 1.2 km downstream, was also destroyed completely. Finally, the debris flow reached a checkdam 3.3 km downstream. This landslide caused no casualties, because a few days ago, signs of an impending landslide were noticed by professionals, and all the people were evacuated by the order of the prefecture government (Hoshino and Asai, 1997; Tohno et al. 1997; Yanagisawa and Umemura 1999).

The geology of this area was summarized by Yanagisawa and Umemura (1999). The bedrock of Neogene Green tuff is covered by Sumikawa tuff and Akita-Yakiyama lava. Solfataric soils are present around the Sumikawa Spa.

As described previously, for landslide motion simulation, three sets of data are necessary: (1) the thickness data of the sliding mass; (2) topographic data of the sliding surface; and (3) physical and mechanical parameters data of the sliding mass, sliding zone and the traveling path. In this simulation, the thickness of the sliding mass, the contour data of the sliding surface, and the topographic data in this area were taken from the "land condition map of the landslide disasters in Hachimantai-Sumikawa in 1997" (Geography Survey Institute, 1998). While, the mechanical property parameters were based on the works of Yanagisawa and Umemura (1999) and ourselves.

Fig. 6 shows the distribution map of the landslide (Chiba, 1997). The upper part of the landslide stopped after slided for 20 to 40 m, while the sliding mass at the tongue of the lower part slided for a very long distance after it rushed into the Sumikawa river, and loaded on the alluvial deposits. The sliding character at the lower part shows an apparent fluidization. Fig. 7 is a bird's eye view of the Akagawa river, which was taken on May 12, 1997, the next day after the landslide occurred. It shows a larger area than Fig. 6. In the middle of the Fig.7, National route 341 crossed the Akagawa river. In the downstream, there is a checkdam. The

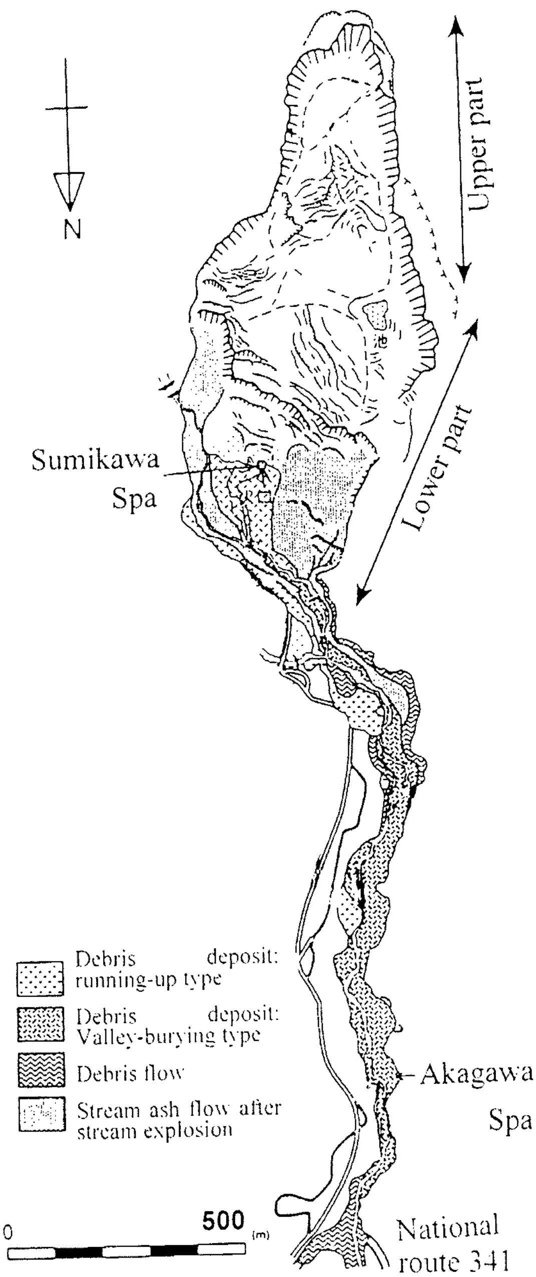

Fig.6 Distribution map of the Sumikawa landslide (modified from Chiba, 1998)

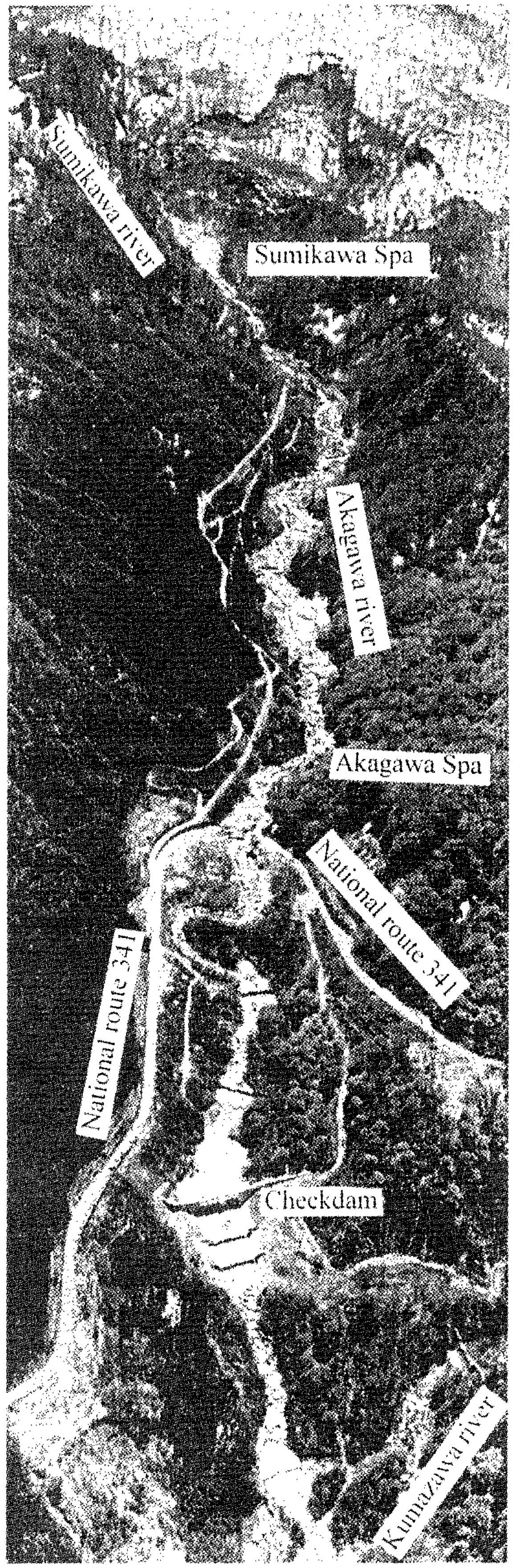

Fig.7 Bird's eye view of the Akagawa river on May 12, 1997 (modified from Chiba, 1998)

landslide mass changed to debris flow and damaged the National route 341, crossed the checkdam. It was even found that there was debris deposit at the down stream of the joint point of Akagawa river and the Kumazawa river.

According to the intensive investigation in the Sumikawa landslide conducted by the Forestry Agency of Japanese government, and Japanese Geography Survey Institute, the topography of the sliding surface and the thickness of the original landslide mass was obtained as shown in Fig.8.

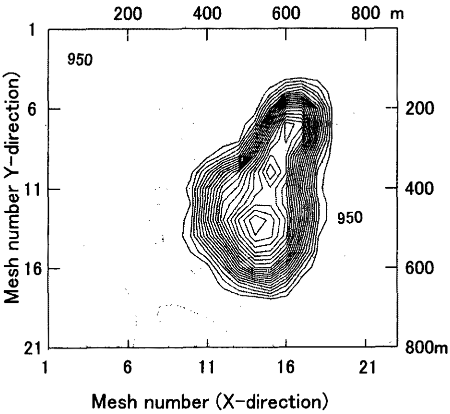

Fig.8 Thickness of the sliding mass

Yanagisawa and Umemura (1999) conducted saturated undrained box-shear test on the sliding surface clay in the main block area, and the effective shear parameters ϕ'=13.5° was obtained. We conducted a ring shear test on the alluvial deposit in the Sumikawa river. It was obtained that the shear resistance at the steady state is about 6 kPa and the effective residual friction angle is 38.6° (Fig.9). These parameters of the sliding surface clay and the river deposits were used in the simulation on the motion of the Sumikawa landslide.

Figs. 10 (a, b, c, d) shows the result of simulation at t = 100, 500, 1000 and 1919 second. The solid contour lines show the thickness of the sliding mass, while the difference between two contour lines is 2 m. From Fig.10 (a) to Fig.10 (d), the sliding process of the sliding mass can be followed, and when t= 1919 second, the sliding mass stopped, when there are about 280,000 m^3 of debris flow out of this area.

5. Conclusions

An improvement was made on Sassa's geotechnical model for the motion of landslides. Based on the concept of two-layer structure of sliding mass of landslides, the upper layer of debris mass will change its thickness during the sliding process, while the soil in the sliding zone mobilizes its shear resistance to the steady state. This makes the simulation on the motion and deposition of landslide more effective. The application to the Sumikawa landslide showed a good correspondence with the actual landslide motion.

Acknowledgements

Computation time was provided by the Supercomputer Laboratory, Institute for Chemical Research, Kyoto University. The assistance from Associate Professor Hiroshi Fukuoka was deeply appreciated.

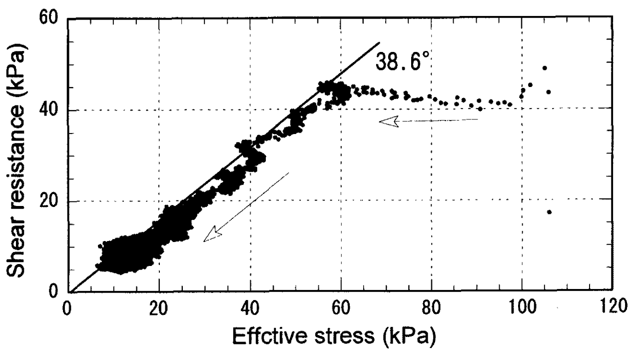

Fig.9 Result of ring shear test on the alluvial deposit in the Sumikawa river

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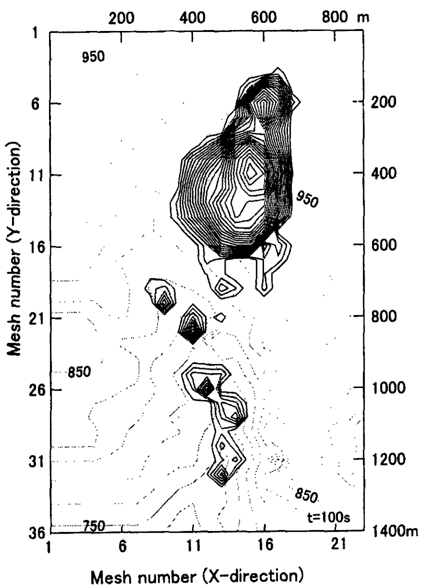

Fig. 10 (a)

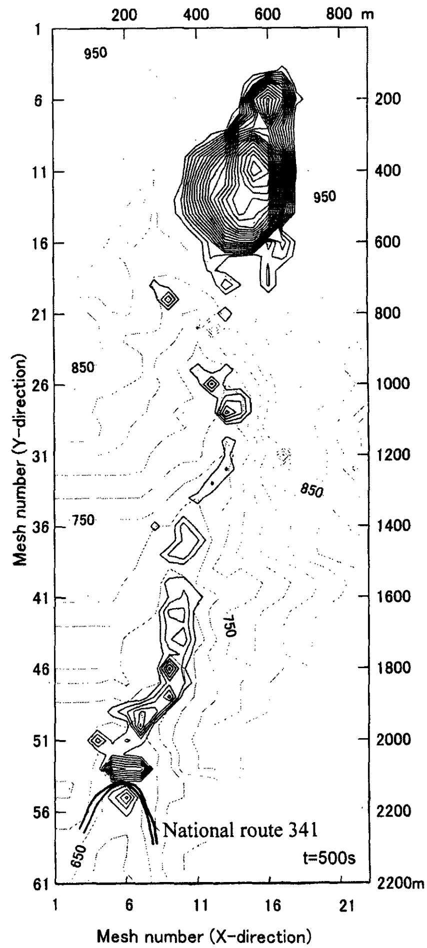

Fig. 10 (b)

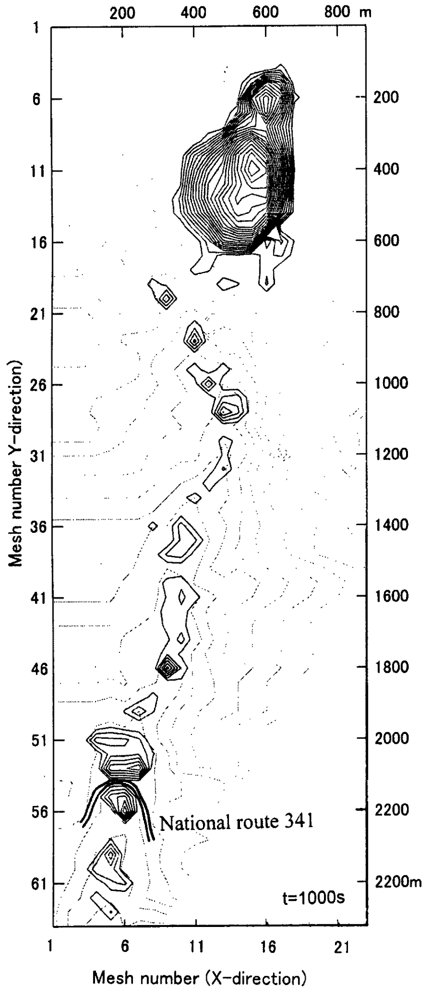

Fig. 10(c)

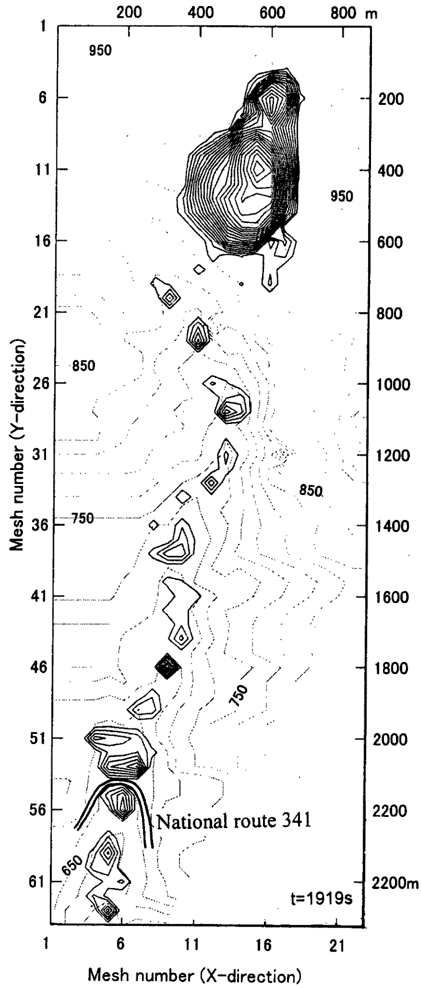

Fig. 10(d)

Fig. 10 Simulation results. The solid contour lines show the thickness of the debris layer
Each contour = 2m
(a): t=100s; (b) t=500s; (c) t= 1000s; (d) t=1919s.

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要 旨

本研究は佐々（1988）より提案された地すべり運動予測モデルにリングせん断試験結果より推定される定常状態せん断強度（τ_{ss}）と運動中土塊の厚さの変化より提案される運動中の摩擦係数変化モデルを導入し、1997 年秋田県澄川地すべりを例として、流動性地すべりの運動範囲の予測を行った．運動土塊の拡散に伴う摩擦係数の増大が的確に表現できるようになり、流動性地すべりの運動範囲のより正確な予測が可能になると思われる．

キーワード：摩擦係数の変化、土塊厚さの変化、過剰間隙水圧の蓄積能力、非排水載荷、定常状態