NUMERICAL SIMULATION FOR A LANDSLIDE DUE TO STRENGTH DEGRADATION OF WEATHERED ROCK INDUCED BY CYCLIC SHEARING DURING EARTHQUAKE

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ABSTRACT :

An analytical method is developed to predict a landslide induced by strong earthquake. In this study, a numerical simulation for one dip slope landslide occurred during 2004 Mid Niigata Prefecture Earthquake in Japan is conducted. First of all, utilizing measurements of ground investigation and laboratory tests, the mechanical properties of the slip surface is investigated to develop an appropriate constitutive model used in the analysis. The simulation is based on the 2-D dynamic elasto-plastic FEM with total stress formulations. A strain-softening behavior of weathered rock is taken into account, which is important to simulate such a catastrophic failure. As a result, the observed phenomena can be simulated by the analysis, and the proposed method is found be very effective.

KEYWORDS: Slope, Earthquake, Collapse, Dynamic, FEM, Strain-softening

1. INTRODUCTION

The dynamic elasto-plastic finite element method (FEM) has been confirmed to be very effective for the evaluation of the seismic stability of slope. Such a fact has been supported by many inspiring papers, e.g., Toki et al. (1985), Griffiths et al. (1988), Woodward et al. (1994), Ugai et al. (1996), Iai et al. (1999) and Uzuoka (2000). Constitutive models derived from rational mechanical theory are indispensable for obtaining analytical results with high accuracy. On the other hand, with models possessing a large number of independent parameters, it may be difficult to adequately determine all of the necessary input parameters. Wakai & Ugai (2004) proposed a simple but rational constitutive model that can be applied to the seismic design of slope. According to above observations, the joint use of G-γ, h-γ relationships and the c-φ shear strength parameterization is adopted in the model. They have suggested the applicability of the proposed model to the prediction of the residual deformation after the earthquake.

By the way, from an engineering point of view, it should be emphasized that the following two kinds of slope failure can be distinguished from each other definitely. The fist one is "**the large deformation**", where the sliding mass stops after the earthquake. In the case, the residual displacement often attracts attention for the seismic design of adjacent structures. The second one is "**the catastrophic failure**" so-called as "**the collapse**". In the case, the sliding mass continues moving even after the earthquake, as far as there are no obstacles on the way of moving. From a mechanical point of view, it means that the self weight of the sliding mass will become not to be supported at all after the earthquake, although it has been statically supported before. The degradation of the shear resistance along the slip surface during the earthquake may cause such a long distance traveling failure. To simulate those phenomena numerically, a constitutive model of soil with strain-softening characteristics needs to be introduced in the elasto-plastic finite element analysis.

In this study, an analytical method is developed to predict so-called "**collapse**" of slope induced by strong earthquake. The analysis is based on the 2-D dynamic elasto-plastic finite element method. A numerical simulation for one dip slope landslide occurred during The 2004 Mid Niigata Prefecture Earthquake in Japan is conducted. In the case, an upper block has slid along the bedding plane during the earthquake. A new constitutive model is proposed to consider the strain-softening characteristics of the slip surface, extended from the original model proposed by Wakai & Ugai (2004) before. The mechanical properties for the tuff seam located along the bedding plane are investigated by the cyclic direct shear tests of undisturbed block samples. As a result, the observed phenomena are simulated well by the analysis.

2. ANALYTICAL MODEL

2.1. Yokowatashi Landslide

Many landslides in mountain area occurred during The 2004 Mid Niigata Prefecture Earthquake in Japan. In this paper, one dip slope landslide along the Shinano River, Yokowatashi landslide, is simulated numerically. As seen in Figure 1(a), a part of the upper Shiroiwa layer and the surface earth with high trees remain as they were on the bedding plane at the far end of this picture. The remaining upper Shiroiwa layer of soft silt rock exposes its side face. The other part of the upper Shiroiwa layer which made up the opposite side of the slid area is visible on site. The portion of the upper Shiroiwa layer between them had covered the planer tectonic dip surface which is clearly seen in the picture, and it has slid more than 72m to the west toward the Shinano River.
The inclination of the bedding plane is approximately 22°. The thickness of the slid Shiroiwa block at the end is about 4 m and those of earth on the block ranges from 20 cm to 1 m. The height of upper Shiroiwa layer remaining at the other side is about 2.5 m with earth cover of 60 cm thick near the ridge of the slope. Figure 1(b) is a close picture showing soil sampling for the laboratory tests. A thin seam layer of 5-10 mm thick was sandwiched between the upper and lower Shiroiwa layers. The material of the seam is tuff. Both Shiroiwa layers were gray, weathered, and changed their color to brown up to about 8 cm inside from the boundary of the seam. The details of Yokowatashi landslide has already been reported by Onoue et al. (2006).

(a) Whole of collapse (b) Undisturbed block sample for laboratory tests

Figure 1 Yokowatashi landslide

2.2. Analytical Model

As aforementioned, the thickness of the slid soft rock plus earth with trees is thin at the one end, and it is rather thick at the other end. A two dimensional numerical analysis is focused on the cross section of the slid slope with its medium thickness. The finite element mesh of eight nodes element is shown in Figure 2. The time history of the response at **Point A** in the figure will be mentioned later. The upper and lower soft rock layers were assumed to be elastic material, and the sandwiched tuff seam was assumed to be elasto-plastic material having a thickness of 10 mm taking strain softening characteristics into consideration. The surface soil at the foot of the slope is sand and gravel spreading down to the Shinano, which was modeled as elasto-plastic material with no strain-softening.

The basic concept of the newly proposed model with strain-softening is the same as the simple cyclic loading model originally proposed by Wakai and Ugai (2004). In those models, the undrained shear strength τ_f with Mohr-Coulomb's c and φ is specified as the upper asymptotic line of the hyperbolic stress-strain curve. In addition, in the new model, the shear strength value τ_f was modified so as to be a simple decreasing function of the accumulated plastic strain γ_p to incorporate the strain softening characteristics (Wakai et al. 2005). Thus, the shear strength value during earthquake is given as,

$$\tau_f = \tau_{f0} + \frac{\tau_{fr} - \tau_{f0}}{A + \gamma^p}\gamma^p \tag{2.1}$$

where the initial strength is denoted as,

$$\tau_{f0} = c \cdot \cos\phi + \left(\frac{\sigma_1 + \sigma_3}{2}\right)_{initial} \times \sin\phi \tag{2.2}$$

In the new model, the shear stiffness ratio, G_0, was also assumed to decrease in proportion to the decrease of shear strength.

The constants of Rayleigh damping were assumed to be α=0.171 and β=0.00174 which are almost equivalent to a damping ratio of 3 % for a vibration period of 0.2 through 2.0 s. The material properties used in the analysis are summarized in Table 1.

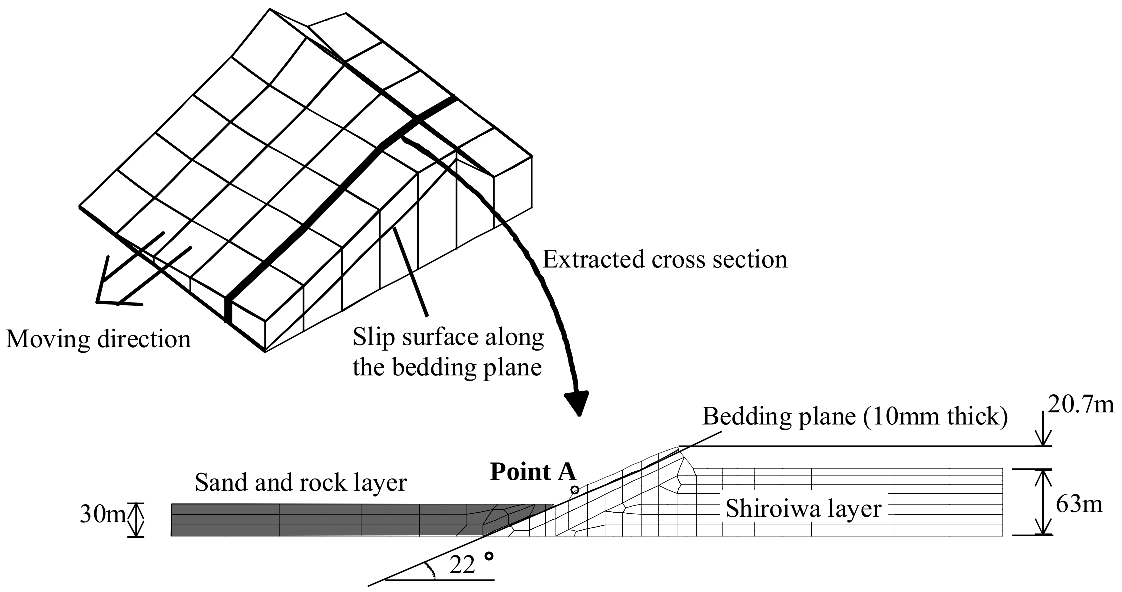


Figure 2 Two dimensional finite element meshes for the simulation

2.3.Modeling Based on Laboratory Test

Figure 3 shows an intact sample consisting of the upper and lower Shiroiwa layers and the tuff seam in between. The sample was subjected to the cyclic direct shear test under constant volume condition. Figure 4 compares the simulated hysteretic loop during cyclic loading by the proposed model to the observed one in each case. The horizontal axis in each diagram is described in strain. As for the figures of the tests, the strain value was estimated as the horizontal displacement divided by the thickness of the sandwiched layer of 10mm. As seen in the figures, it is shown that the increase of the number of cycle gradually decreases the mobilized shear stress during cyclic loading. Although they don’t perfectly agree, they are almost similar to each other in various confining pressures.

Table 1 Material parameters used in the analysis

	Layer	Shiroiwa silt rock	Tuff seam	Sand and gravel
Basic parameters	Young’s modulus $E(\mathrm{kN/m^2})$	100000	30000	30000
	Poisson’s ratio ν	0.3	0.3	0.3
	Cohesion $c(\mathrm{kN/m^2})$	—	24	0
	Internal friction angle ϕ(deg)	—	30.9	35
	Dilatancy angle ψ(deg)	—	0	0
	$b \cdot \gamma_{G_0}$	—	8.0	18.
	n	—	1.40	1.35
	Unit weight $\gamma(\mathrm{kN/m^3})$	20.0	18.0	18.0
Strain-softening parameters	Residual strength ratio τ_{fr}/τ_{f0}	—	0.30	—
	A	—	4.0	—

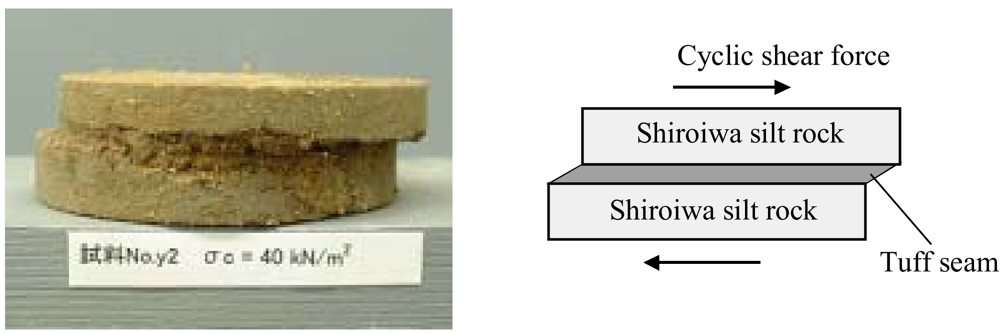
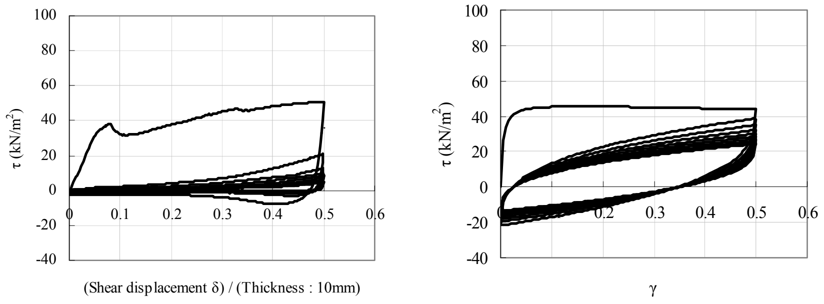
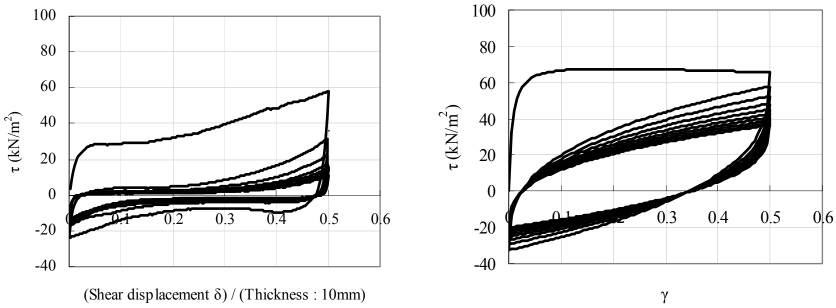


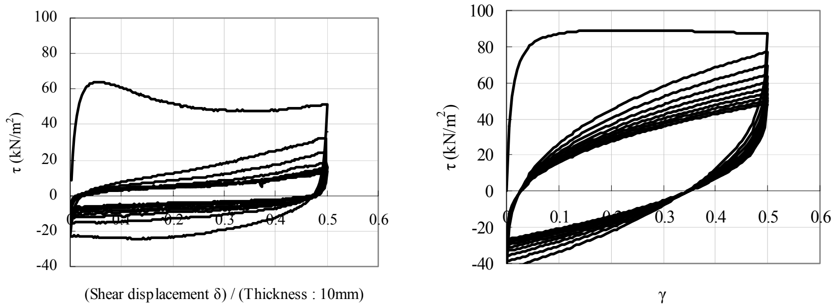
Figure 3 Test specimen consisting of upper and lower silt rocks with tuff seam in between



(a) Initial normal stress, σ =40kN/m^2



(b) Initial normal stress, σ =80kN/m^2



(c) Initial normal stress, σ =120kN/m^2

Figure 4 Comparison between observed (left column) and simulated (right column) hysteretic loops

3. ANALYTICAL RESULTS

3.1. Time Histories of Displacement

The acceleration record observed at Takezawa (EW) (Figure 5) was used as an input wave in the analyses. Two analytical cases were conducted to examine the influence of seismic intensity on the inducement of sliding. One is the case where the observed acceleration record is input as it is at the base of the analytical area (here, denoted as the **Full** case), and the other is the case where the acceleration amplitude is compressed to one half that of the observed wave is input (the **Half** case).
Figure 6 shows the time histories of horizontal displacement at the foot of the slope, namely **Point A**, in Figure 2. As seen in the figures, the slope did not suffer any big damages after earthquake in the **Half** case, while a so-called collapse occurred in the **Full** case. The horizontal displacement is 20 m at an elapsed time t=40 s and almost 65 m at t=50 s.

3.2. Global Deformation Mode

The residual deformation at 0, 10, 20, 30, 40 and 50 s after the beginning of the seismic motion in the

Full case is shown in Figure 7. The long distance movement of the upper Shiroiwa block along the bedding plane can be seen. Such a result agrees actual phenomena. However, you should notice that the formulations used in this study are based on the infinitesimal strain theory. The predicted velocity of the sliding mass may not have sufficient accuracy, and the large deformation effect needs to be considered to obtain a more accurate result for moving body.

3.3. Newly Proposed Index for Seismic Stability of Dip Slope

In this section, the three variables are defined to use. The one is the mobilizable shear resistance ΣR_f, which is the total of the undrained shear strength τ_f given by Eq.(2.1) along the slip surface. The second one is the sliding force ΣT, which is the total of the shear stress τ along the slip surface. And the last one is the sliding force due to self gravity of the upper sliding block ΣT_s, which does not include the inertia. Figure 8 shows the time histories of ΣR_f and ΣT in each case. It is found that ΣT gradually decreased and finally coincided with ΣR_f around t=35 s, which was just before a continuous moving of the upper block has started as shown in Figure 6.

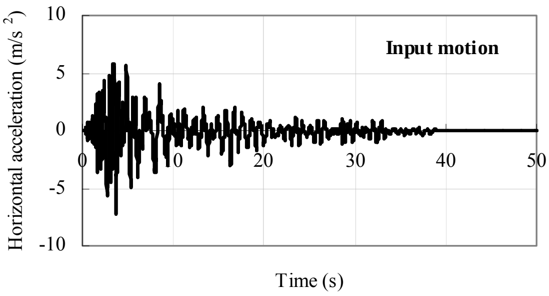


Figure 5 Input horizontal acceleration.

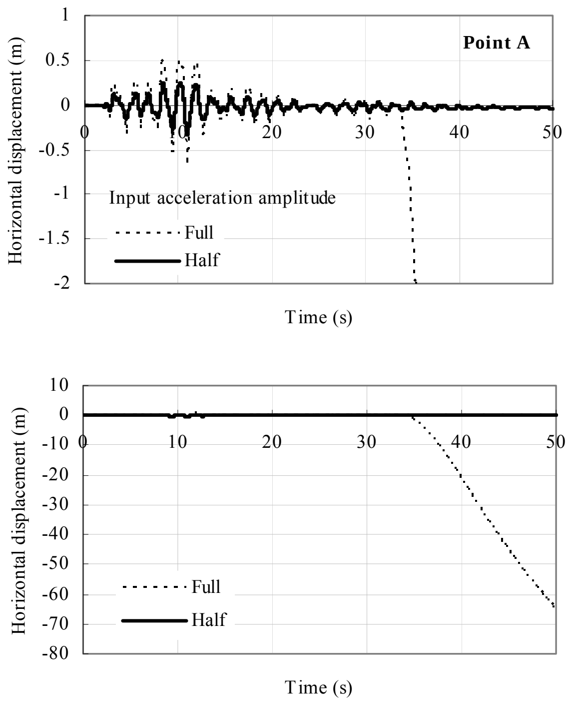


Figure 6 Time histories of horizontal displacement at **Point A**

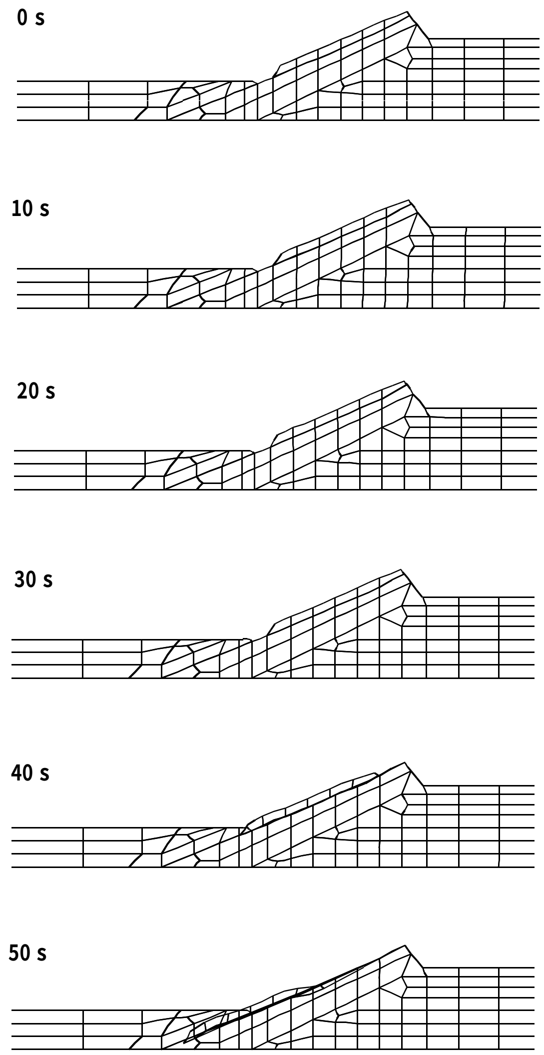
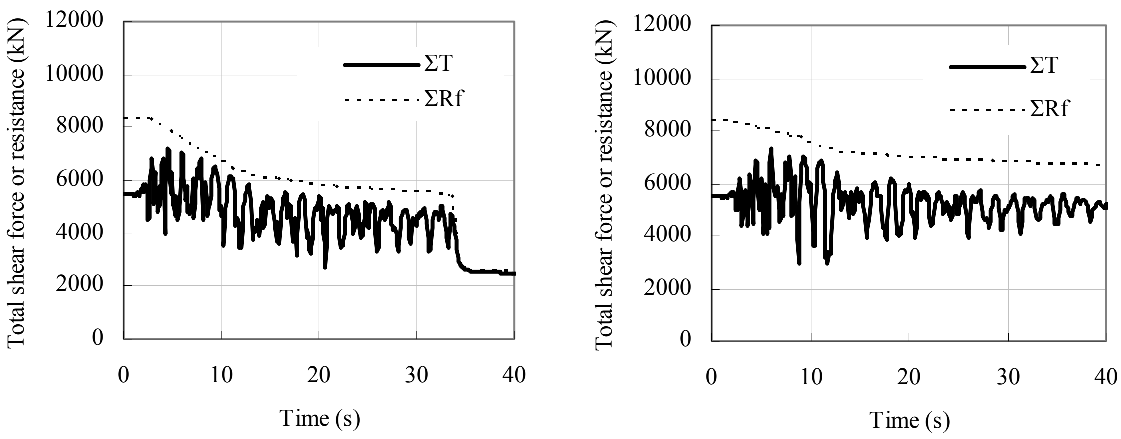


Figure 7 Global deformation mode during earthquake

According to this fact, the collapse of slope can be defined as the balance between ΣR_f and ΣT. To simplify the discussion, here we use ΣT_s as constant during earthquake, instead of ΣT. And the following index F_d is introduced to evaluate the seismic stability of such a dip slope.

$$F_d = \frac{\sum R_f}{\sum T_s} \tag{3.1}$$

Figure 9 shows the time histories of F_d in each case. The moment that F_d became 1.0 accords with the moment of collapse. The index is found to be useful for explanation of a dip slope landslide.



(a) Full case (b) Half case

Figure 8 Time histories of ΣR_f and ΣT in each case

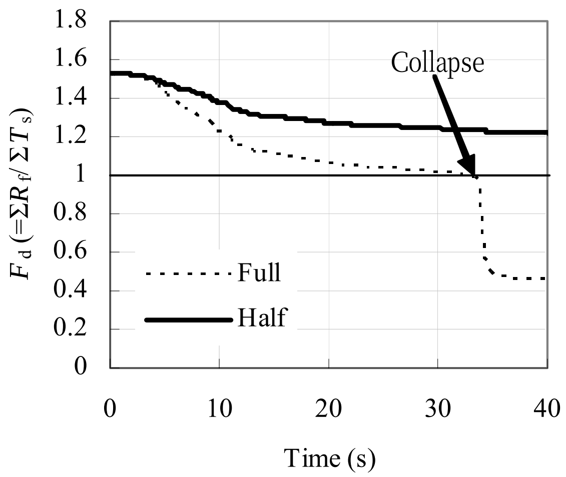


Figure 9 Time histories of F_d in each case

4.CONCLUSIONS

Throughout this study, the following conclusions have been obtained.

(1) An analytical method was developed to predict a dip slope landslide induced by strong earthquake. In the analysis, the strain-softening characteristics of the bedding plane were considered numerically. The verification of the proposed method was achieved by a numerical simulation of an actual dip slope collapse. It was concluded that the proposed method was effective for such a simulation.

(2) According to the fact such that the collapse of slope can be defined as the balance between the mobilizable shear resistance along the slip surface and the sliding force due to the self gravity of the sliding block, a new index F_d to evaluate the collapse was proposed. It was found to be useful for explanation of a dip slope landslide.

(3) The formulations used in this study were based on the infinitesimal strain theory. The predicted velocity of the sliding mass may not have sufficient accuracy, and the large deformation effect needs to be considered to obtain a more accurate result for moving body.

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