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Progressive failure of slope due to tunnel excavation and its numerical simulation

Rupture progressive de pente due à l'excavation de tunnel et sa simulation numérique

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ABSTRACT
In the paper, a case history was studied by FEM based on elasto-viscoplastic model with strain softening. In the case history, a natural slope failure occurred due to a tunnel excavation, in which no any evidence of rainfall or underground water movement could be observed. A finite element analysis based on an elasto-viscoplastic model is conducted to simulate the progressive failure of the slope. The mechanical behavior of the slopes, such as the development of shear strain, the deformation of the ground, the propagation of shear band and the progressive failure are discussed in detail.

RÉSUMÉ
Dans cet article, une étude de cas est traitée par la méthode des éléments finis basée sur un modèle elasto-viscoplastic avec ramollissement de contrainte. Dans cette étude de cas, une rupture normale de pente s'est produite à la suite d'une excavation de tunnel, dans laquelle aucune évidence de précipitations ou de mouvements souterrains d'eau n'a été observée. Une analyse par la méthode des éléments finis basée sur un modèle elasto-viscoplastic est conduite pour simuler la rupture progressive de la pente. Le comportement mécanique des pentes, tels que le développement de la contrainte de cisaillement, la déformation des matériaux, la propagation de la bande de cisaillement et la rupture progressive sont discutés en détail.

1 INTRODUCTION

In the case when pore water is not involved, it is known that progressive failure of cut slope is usually caused by the deterioration of the microstructure of geologic materials due to swelling and weathering during and/or after the cut of slope, and softening behavior of ground (Adachi et al., 1994). In dealing with long-term stability of a cut slope, time-dependent behaviors brought about by inherent viscosity of geomaterial should be considered. If the time dependency due to the deterioration of the strength of geomaterials is considered, then for boundary value problem, it is necessary to conduct a numerical analysis based on elasto-viscoplastic model (Adachi et al., 1990).

In the paper, a natural slope failure caused by tunnel excavation was investigated in detail. In the slope failure (Yashima et al., 2001), no any evidence of rainfall or underground water movement could be observed. The slope failed several days after the arriving entrance of the tunnel was cut through. Progressive failure was clearly observed. Therefore, the elasto-viscoplastic behavior of ground is thought to be the main reason of the slope failure. In the paper, a finite element analysis based on an elasto-viscoplastic model (Adachi et al., 1994) was conducted to simulate the progressive failure of the slope. The purpose of the research is to develop a numerical method for evaluating the progressive failure behavior of natural and cut slopes not only in qualitatively but also in quantitatively.

2 NUMERICAL SIMULATION OF PROGRESSIVE FAILURE IN A NATURAL SLOPE DUE TO TUNNEL EXCAVATION

The slope failure considered in the paper happened in 1997 in Yoshigi-Gun, Gifu Prefecture, Japan. The failure was ignited by an excavation of tunnel whose axial line is crossed obliquely with the slope, as shown in Figure 1. In this case, an embracing retaining wall and retaining reclaimed embankment shown in Figure 2 are usually constructed to resist against eccentric earth pressure acting on lining of tunnel at the entrance of the tunnel before the tunnel is excavated through. In field excavation, however, the engineer at the construction site found that the rock in the surrounding ground of the tunnel was quite good and then decided to excavate throughout the entrance without constructing the retaining reclaimed embankment, as shown in Photo 1. The retaining reclaimed embankment is usually made of reclaimed soil mixed with cement and has a quite strong strength to be able to resist an eccentric earth pressure from surrounding ground. Six days after the entrance of the tunnel were excavated, several cracks appeared in the first lining and then some rock bolts began to fall out, as shown in Photo 2. The cracks in shotcrete developed very quickly. One and half hours after the first crack was observed, the tunnel lining was completely destroyed due to the failure of the slope above the tunnel. Photo 3 shows the failed steel frame after the fallen soils were moved away.

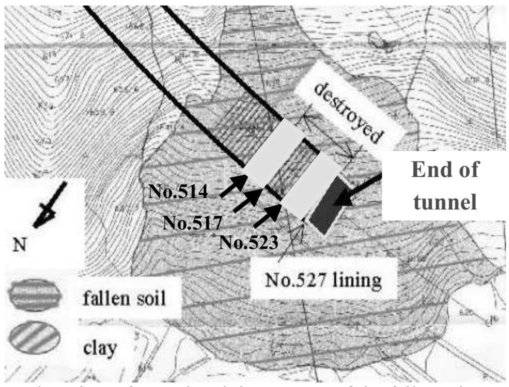


Figure 1 Plan view of tunnel and the contour of the failure slope

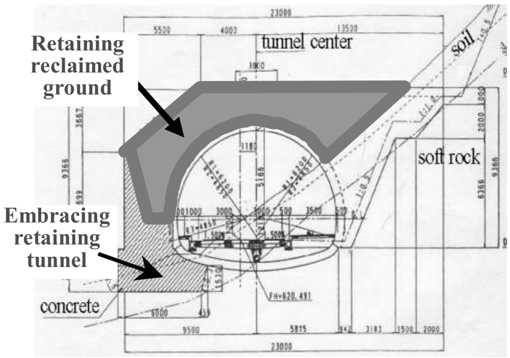


Figure 2 Sectional view of tunnel in original design

The topographic features of the arriving entrance of the tunnel obtained from aerial photographs and field survey are listed as follows:

- A trace of large-scale failure with 30mx30m along the riverside of the tunnel was observed. The failed soil moved towards the river.
- The failed slope located at the corner of the river, experienced erosion of water flow.
- A clear evidence of a thrust of the ground due to slope failure can be observed at the erosive slope.
- The trace of old landslide can be observed on the upper part of the slope.

The geological condition of the surrounding ground are as follow:

- The slope that failed is very deep at low part.
- Several clayed weak layers distributed within the ground and formed as distinct discontinuous faults.
- Thick layer of strong weathered rock distributed along the slope.

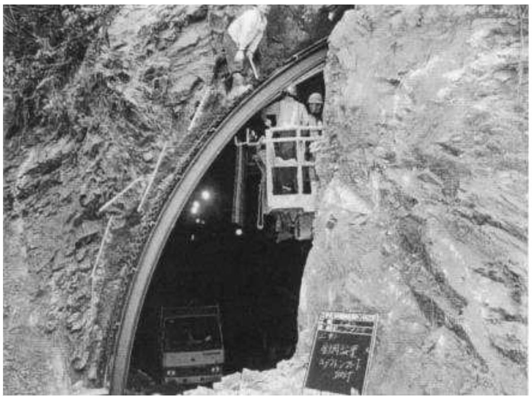


Photo 1 Situation immediately after the entrance was cut through

(a) Crack in the shotcrete near entrance

(b) Fallen of rockbolt

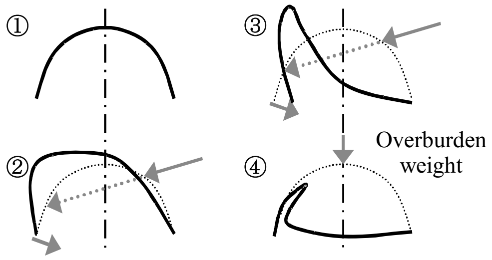
Photo 2 Omen of tunnel collapse

(a) View from outside

(b) View from inside

Photo 3 Failure of steel lining

Figure 3 shows the mechanism of the failure of steel lining. According to the weather condition, there was no any evidence of rainfall or underground water movement. It is, therefore, very clear that the failure did not related to any underground water or rainfall. The failure was caused by the excavation of the tunnel, which is obliquely crossed with the slope, with the absence of an retaining reclaimed ground formerly designed to resist the eccentric earth pressure acting on lining of the tunnel. After the excavation of the arriving entrance of the tunnel, large shear stresses generated within the ground. Due to the inherent creep behavior of soft rock and the strain softening characteristics, creep strains developed in the ground gradually. Local progressive failures occurred and finally lead to a total failure of the slope. Therefore, in simulating the progressive failure of the slope, an elasto-viscoplastic model with strain softening is adopted in the finite element method.



① Lining without the action of shear force
② Progress of deformation of lining due to shear force
③ Further deformation due to the moment at right foot of lining
④ Failure caused by overburden weight

Figure 3 Failure mechanism of steel lining

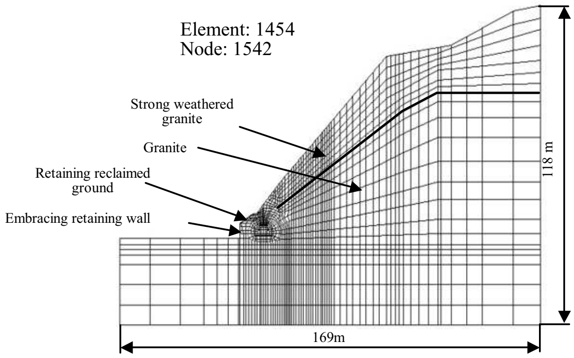


Figure 4 Finite element mesh

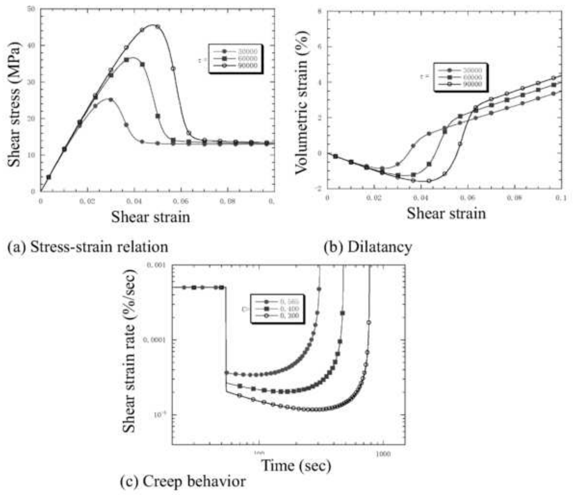


Figure 5 Stress-dilatancy relation and creep behavior of soft rock

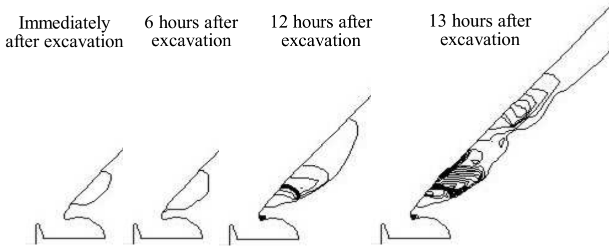
Figure 4 shows the finite element mesh in two-dimension (2D) FEM. Though the problem considered here is a typical 3D problem, a relatively simple 2D analysis was conducted. Because the problem is time dependent, the process of the tunnel construction should be considered in time sequential. In constructing the tunnel, rock bolts with lengths of 4 & 6 meters, steel lining and shotcrete were used, these materials were simulated with plastic beam elements. The calculation is divided into three stages. First stage is to calculate the initial stress field with gravitation under the boundary condition that two vertical side-faces were roller boundaries and fixed boundary at the bottom. The second stage of the calculation is the excavation of tunnel with a ratio of excavation 40%. The third stage of the calculation is the installation of lining followed by the excavation of residual 60% of the initial stress and then the time dependent behavior of the ground movement is calculated continuously.

As to the determination the parameters involved in the elasto-viscoplastic model, due to the difficulties of undisturbed sampling of the ground, the parameter M^*_f, which represents the residual strength of the ground, is assumed as the value of 1.2 η_0, where η_0 is the initial stress ratio of ground. Table 1 lists the values of the material parameters used in the calculation.

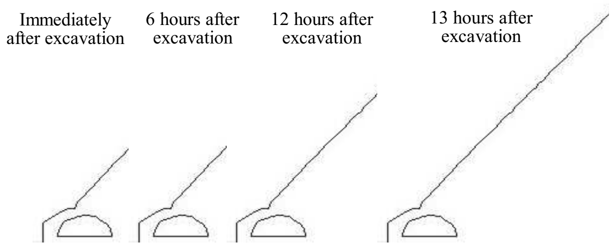
Table 1 Material parameters of ground

Parameters	Strong weathered granite	Granite	Retaining reclaimed ground	Embracing retain wall
E (MPa)	100	600	100	25000
ν	0.35	0.25	0.35	0.25
M^*_f	$1.20\times\eta_0$	∞	1.49	∞
G'	480			
b (MPa)	0.87			
σ_{mb} (MPa)	18.0			
$\overline{M}_m$	1.25			
τ	90000			
a	0.959			

Figure 5 shows the stress-strain-dilatancy relation and creep behavior of the soft rock whose values of material parameters are listed in Table 1. The ground shows a typical strain hardening and strain softening and shear dilatant. The creep failure behavior is also clearly described by the model (Adachi et. al., 1994). In the model, parameters τ and C are determined by curve-fitting method based on conventional triaxial compression and creep tests.



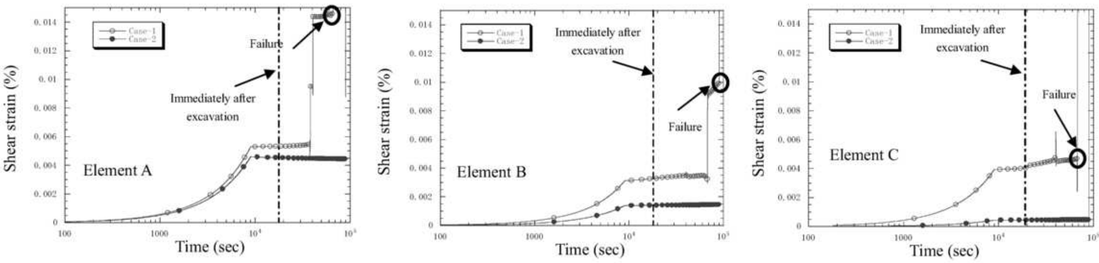
(a) Without retaining reclaimed ground



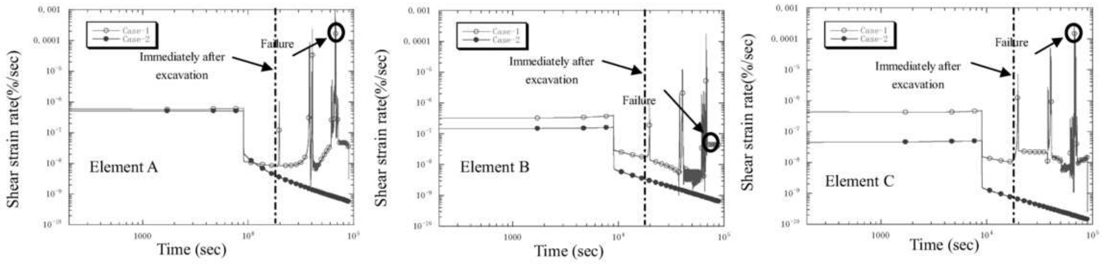
(b) With retaining reclaimed ground

Figure 6 Development of shear strain within slope

Figure 6 shows the comparison of the development of shear strain within the ground of the slope under different construction condition, that is, Case 1: without retaining reclaimed embankment and Case 2: with retaining reclaimed embankment. It is found that in Case 2, due to the retaining reclaimed embankment, the creep shear strain did not develop further after the completion of the tunnel excavation. On the other hand, in Case 1, the creep shear strain developed continuously after the tunnel excavation and finally reached a large strain (several hundred percentage, which means a completely failure of soft rock). The same tendency can be seen in Figure 7, in which a comparison of the development of failure zone in two different cases is shown. In Case 1, a progressive failure zone can be clearly identified, showing that

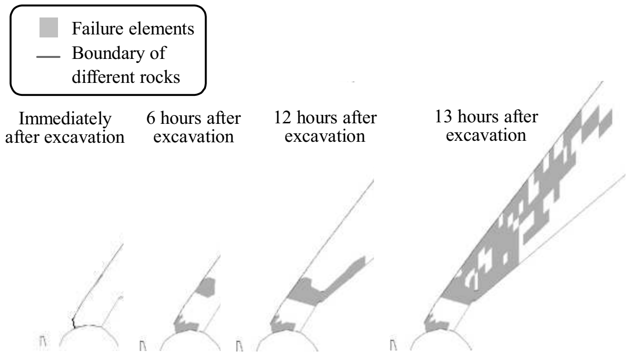


(a) Time history of shear strain

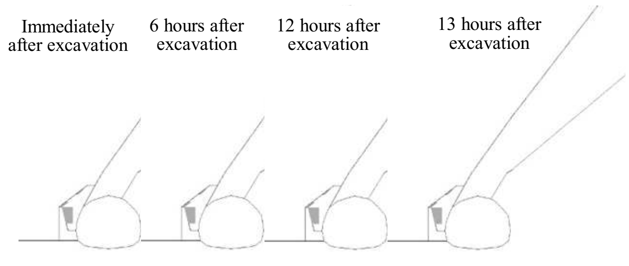


(b) Time history of shear strain rate

Figure 9 Time history of shear strain and shear strain rate within ground



(a) Without retaining reclaimed embankment



(b) With retaining reclaimed embankment

Figure 7 Development of failure area within slope

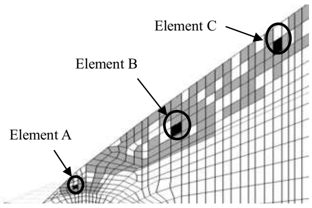


Figure 8 Position of the elements to be noticed

Photo 4 View of the recovery slope

the present finite element analysis based on an elasto-viscoplastic model can well simulate the behavior of progressive failure due to the inherent creep behavior of soft rock.

Figure 9 shows the time history of shear strains and shear strain rates of ground at different places as shown in Figure 8. In Figure 9a, creep strains at elements A, B and C are plotted in two different cases. It is found that in Case 2, the creep strain did not develop any more due to the resistance effect of the retaining reclaimed embankment. In Case 1, however, the creep strain developed slowly at the beginning and then increased abruptly at certain time and finally reached a failure state, which is defined as a state where the stress ratio of the ground undergone strain softening and reached the residual stress state. Figure 9b shows the creep strain rates of the ground. In Case 1, a typical creep failure curve can be obtained, being almost the same as the curve of element simulation plotted in Figure 5.

From Figures 7 and 9, a clear conclusion can be given, that is, the retaining reclaimed ground is definitely needed in present case history. If the retaining reclaimed embankment were constructed during the excavation of the tunnel, the larger-scale creep failure of the slope would not happen. Photo 4 shows the picture of the recovery slope. In order to keep a safe slope gradient, the slope was cut with 15 cutting berms, in which 240,000m^3 soil was moved away. It took 4 million US dollar to recover the slope!

3 CONCLUSIONS

In this paper, a large-scale slope failure due to tunnel excavation was studied. Based on the carefully topographic and geologic survey, the ignition of the slope failure can be regarded as the absence of a retaining embankment designed at the beginning. Based on the field survey, 2D finite element analysis was conducted and found that it is possible to simulate the progressive failure of the slope where there was no any evidence of rainfall or underground water movement. Due to the factor that the creep behavior of the ground was an inherent characteristic of the ground, the progressive failure of the slope should be considered with suitable elasto-viscoplastic model in the numerical calculation related to boundary valuable problems. The result of the numerical analyses showed that if the retaining reclaimed embankment were constructed during the excavation of the tunnel, the larger-scale creep failure of the slope would not happen.

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