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# NON-LINEAR ANALYSES OF DYNAMIC BEHAVIOR OF EMBANKMENT STRUCTURES CONSIDERING TENSILE FAILURE

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## SUMMARY

A number of embankments were damaged during earthquakes. Although many tensile cracks were frequently observed on the damaged embankment, few earthquake response analyses have considered the tensile failure in its constitutive model of soil. In this paper, an elasto-plastic constitutive model of soil is proposed in which the tensile failure is considered. A couple of non-linear dynamic FE analyses of embankment are performed using the proposed constitutive model; one is the simulation of static inclination experiment of embankment models and the other the analysis of the railway embankment structure damaged during the 1968 Tokachi-oki earthquake. The results of former analysis coincide well with those of experiment in terms of collapsing process at each inclination angles. The later analysis can simulate the actual situation; the newer part of the embankment is damaged and the older part is deformed little.

## INTRODUCTION

Embankments were frequently damaged during earthquakes. In Japan, however, only a seismic coefficient method is used for earthquake resistant designs of them. Though many seismic design codes have been revised in order to introduce near-source strong ground motions after 1994 Northridge earthquake and the 1995 Hyogo-ken nambu earthquake, those for embankment structure are not yet. It is because the deformations of embankment caused by strong shaking are difficult to be estimated. Investigations for estimating the deformations of embankment are necessary in order to develop a new seismic design method based on performance design.

Some simplified methods have been proposed to estimate the permanent deformations of embankments which are induced by seismic motions [e.g. Seed et al. 1966; Makdisi et al. 1977]. In these methods the embankment bodies are replaced by the block models based on the Newmark's procedure [Newmark 1965] or by the FE models with elasto-plastic constitutive model. The former methods cannot consider the deformations except sliding and have some difficulties for estimate the equivalent motion which is input to the sliding rigid block. The later methods are frequently adopted in these days [e.g. Griffiths et al. 1988;Ugai et al. 1996; Iai et al. 1999] because it can consider various failure modes. However many of those analyses do not consider specifically the tensile failure in its constitutive model.

Tensile cracks are frequently observed on the damaged embankments and the specimens of embankment model for experiments. Tensile failure should be related to the collapsing process of embankment and important for evaluating performance of seismic resistance. Toki et al. (1985) analyzed the sliding displacement of slope during earthquakes based on joint elements which can simulate tensile cracks. However the joint elements must be arranged along the potential sliding lines which are predicted in advance of analysis. Very few studies are reported in which seismic response analyses for embankments considering tensile failure are performed. In this paper, an elasto-plastic model considering tensile failure is proposed to simulate the damaged embankments with tensile cracks. The experimental inclination tests are simulated using the proposed constitutive model in order to examine its performance. The dynamic response analysis is performed about the collapsed railway embankment

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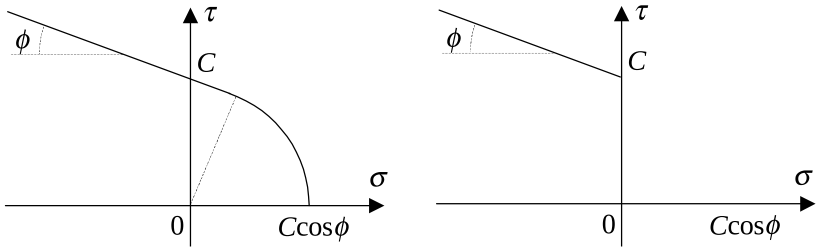
with tensile cracks during the 1968 Tokachi-oki earthquake. The result coincides well with the damaged state reported.

## CONSTITUTIVE MODEL FOR SOIL CONSIDERING TENSILE FAILURE

When tensile failure occur in the soil element, it should be considered that tensile cracks are generated and tensile stress becomes zero. Therefore, two different failure criteria are used before and after tensile failure. Before tensile failure the failure criteria follow Mohr - Coulomb's criteria in compressive range and have the circular caps in tensional range. After tensile failure it loses the circular cap as shown in Figure 1.




(a) Before tensile failure (b) After tensile failure

**Figure 1: Failure criteria for soil element**

Von Mises's model with a hemisphere cap in tensional range is used for the plastic potential function as shown in Figure 2. As for shear behavior the shear stress drops to the residual stress when failure occurs, as shown Figure 3. The residual stress is defined by the parameters called as "residual internal friction angle" and "residual cohesion".

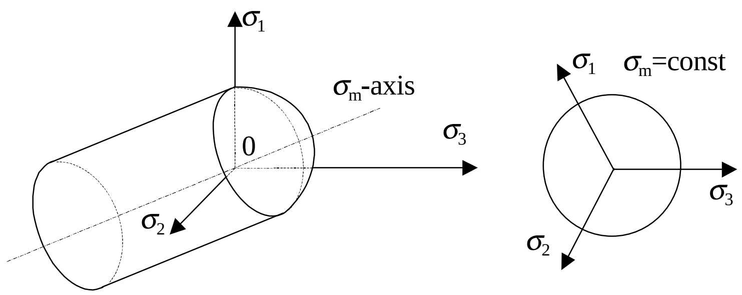



**Figure 2: Plastic potential for soil element**

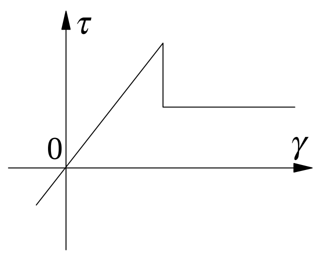



**Figure 3: Constitutive model**

## VERIFICATION FOR CONSTITUTIVE MODEL BY COMPARING WITH STATIC INCLINATION TEST OF EMBANKMENT MODEL

### 3.1 Summary of inclination test

Ito et al. (1982) performed the experiment of static inclination tests to investigate failure pattern of embankment during earthquakes. Figure 4 shows the embankment model used and Figure 5 shows the inclination angle defined in test. The tests were done for 4 times using specimens with the same shape and material. The inclination angle, $\theta$, were enlarged by a degree after investing the state of specimen.

Tensile cracks occurred at 34 to 35 degrees on the average of four tests. The collapsing inclination angle seems to be 36 to 37 degrees. The distributions of clacks are shown in Figure 6.

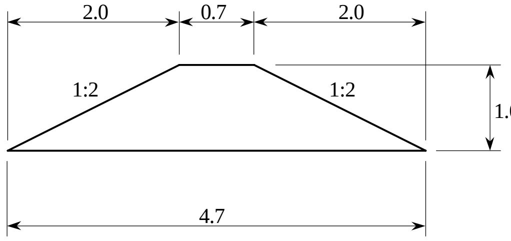



**Figure 4: Embankment model for inclination test**

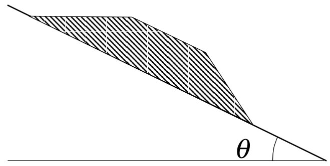



**Figure 5: Inclination angle**

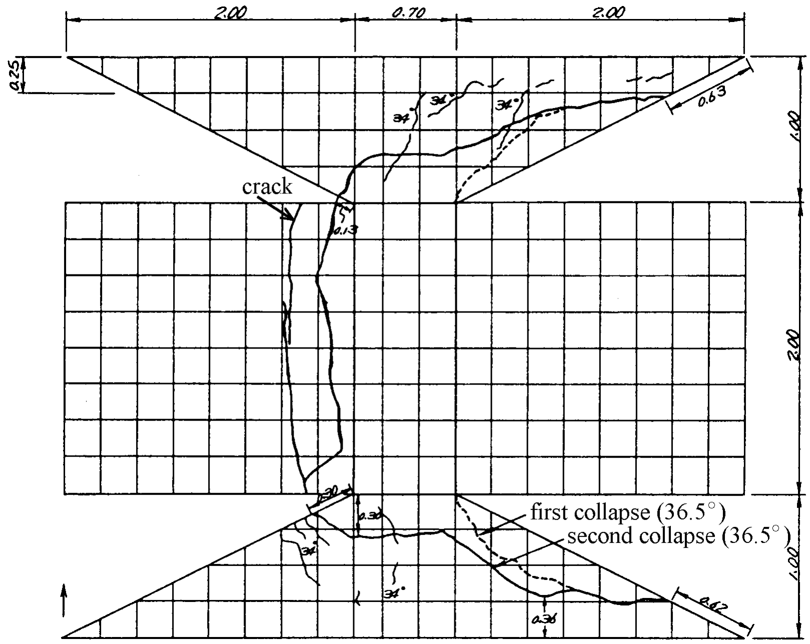



**Figure 6: Distribution of cracks [Ito et al. 1982A]**

## 3.2 Analytical condition

Dynamic response analysis is performed to simulate the experiment of static inclination test. Figure 7 shows FE mesh analyzed and Table 1 shows the properties of material which are estimated from Ito et al. (1982A, 1982B). The value of residual cohesion is set to 88 % of the initial cohesion.

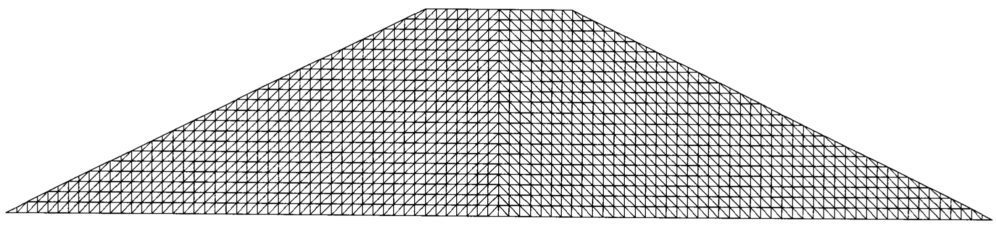


**Figure 7: FE mesh to simulate the inclination test**

Firstly self-weight analysis is performed to calculate the initial stress of the embankment. Secondly the equivalent horizontal and vertical accelerations which are occurred in relation to the inclination angle are input into the model. The time history of inclination angle is shown in Figure 8. Non-iterative time integration method [Sakai et al. 1996] is used for calculating non-linear response. Rayleigh damping coefficients are determined as 5% damping at 0.1Hz and 20Hz. The time interval, $\Delta t$, is 0.01 second.

**Table 1: Properties of material**

| | | |
|---|---|---|
| Unit mass density | : $\gamma_t$ | 1.4 t/m$^3$ |
| Poisson's ration | : $\nu$ | 0.3 |
| Shear velocity | : $V_s$ | 63 m/sec |
| Cohesion | : $c$ | 2.45 kN/m$^2$ |
| Internal friction angle | : $\phi$ | 36 degrees |
| Residual cohesion | : $c_{rd}$ | 2.16 kN/m$^2$ |
| Residual internal friction angle | : $\phi_{rd}$ | 36 degrees |

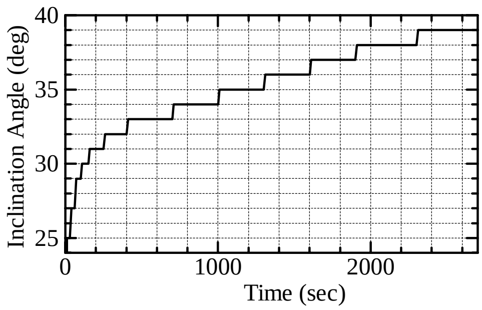


**Figure 8: Time history of inclination angle**

### 3.3 Results of analysis

Figure 9 shows the displacement at the right shoulder of the embankment. The collapse occurs at 1600 seconds when the inclination angle reaches 37 degrees as shown Figure 8. The collapsing angle calculated is the same as that obtained by the experiment. If the values of residual cohesion are changed to 100 % and 80 % of the initial cohesion, the collapsing angles become 39 and 35 degrees, respectively.

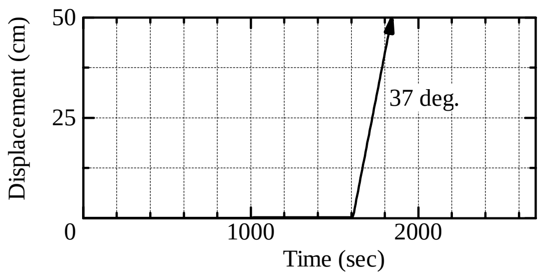
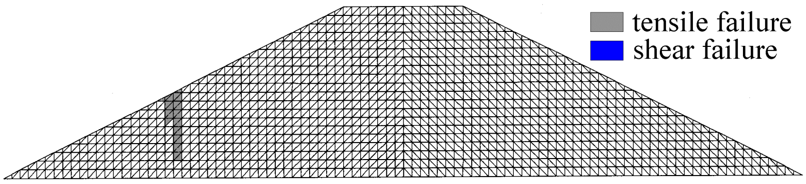


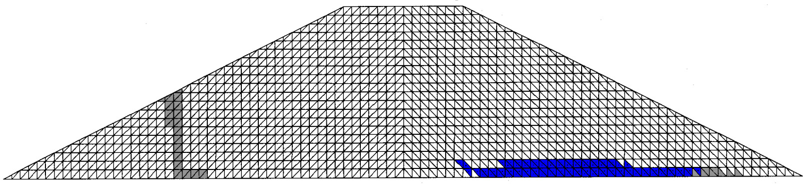
**Figure 9: Displacement at the right shoulder of the embankment**

The collapsing process of the embankment is shown in Figure 10 to compare with the experiment. Where, T and T' indicate the time from the beginning of the analysis and the time from the moment when the newest inclination angle is applied, respectively. Gray and black elements stands for the element in which the plastic deformation is induced at the moment. The deformations are drawn using the same scale with the size, namely the deformation is not exaggerated

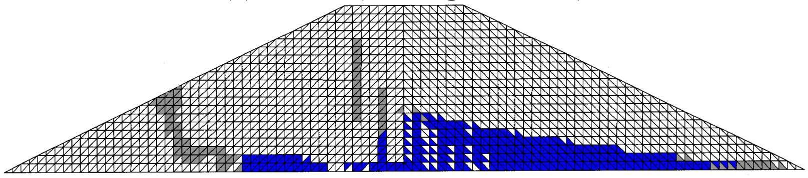
A series of vertical tensile crack is appeared in the middle of left slope when $\theta$ is 25 degrees, as shown in Figure 10(a). In the case of experiment, however, no crack is observed at the location through all inclination angles as shown in Figure 6. When $\theta$ is 34 degrees, as shown in Figure 10(c), a series of the vertical tensile failure in the center of the embankment is appeared and the similar tensile crack is also observed in the experimental result. The tensile failure run to the upper part of the embankment at $\theta$=35 degrees. The sliding failure occur from the toe of the slope to another side of shoulder of embankment at $\theta$=37 degrees. The similar phenomena are observed in the experiment at the same angle. The results of the numerical analysis coincide well with those of the experimental test.



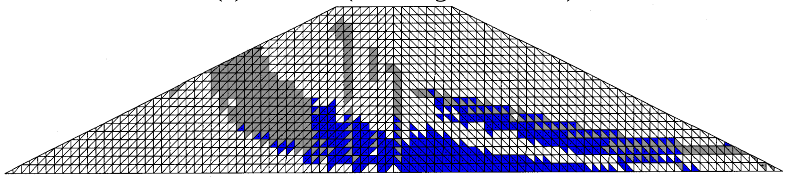

(a) T’=0sec ($\theta$=25deg.,T=10sec)



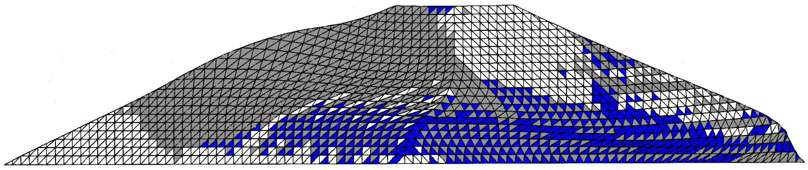
(b) T’=0sec ($\theta$=30deg.,T=110sec)



(c) T’=0sec ($\theta$=34deg.,T=710sec)



(d) T’=0sec ($\theta$=35deg.,T=1,010sec)



(e) T’=140sec ($\theta$=37deg.,T=1,750sec)

**Figure 10: Collapsing process of the embankment**

## NON-LINEAR DYNAMIC ANALYSIS FOR COLLAPSED RAILWAY EMBANKMENT DURING THE 1968 TOKACHI-OKI EARTHQUAKE

### 4.1 Summary of collapse of embankment

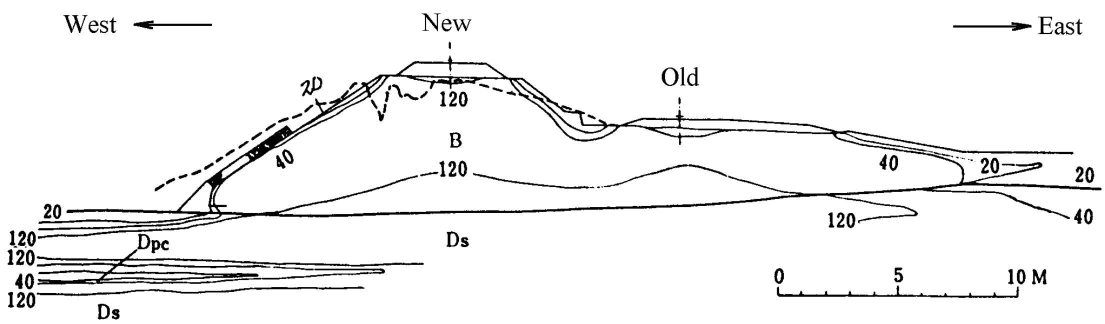



**Figure 11: Cross-section of the embankment analyzed [Ikehara 1972]**

A large number of embankments were seriously damaged during the 1968 Tokachi-oki earthquake. Response analysis is performed for an embankment which has open cracks running parallel to the axis of embankment and hard bearing ground. The embankment analyzed has the older and newer parts, constructed on diluvial deposit which consists of gravelly volcanic cohesive soil, located at the point of 687K070m between Ottomo and

Chibiki.  Figure 11 shows the cross-section of the embankment.  The contour lines in the figure show the Dutch-cone penetration resistance in kg/cm$^2$ and the broken line indicates the shape after the earthquake.

The damages are concentrated on the newer part of embankment; it is deformed toward the western direction and has tensile cracks on the top of slope.  On the contrary the older part has little deformation.

**4.2 Analytical condition**

FE analysis considering tensile failure is performed for this embankment.  Rayleigh damping coefficients are determined by the same condition used in the previous chapter.  Time interval $\Delta t$ is 0.001 second for the analysis.  Figure 12 shows the FE mesh analyzed.

The material of the embankments is fine-sandy volcanic cohesive soil.  The properties of material are listed in Table 2 which are determined from Ikehara (1973). Here the residual strengths under undrained condition are used because fine-sandy volcanic cohesive soil has a small coefficient of permeability [Uezawa et al. 1970].

The incident wave on the top of diluvial layers at Hachinohe harbor were identified using the computer program SHAKE [Schnabel 1972] from the accelerogram observed on ground surface [Tsuchida 1969].  Since the embankment is constructed on diluvial deposit, the EW component, the cross direction of the embankment, of the identified incident wave is used for the analysis.  The time history of input ground motion is shown in Figure 13.

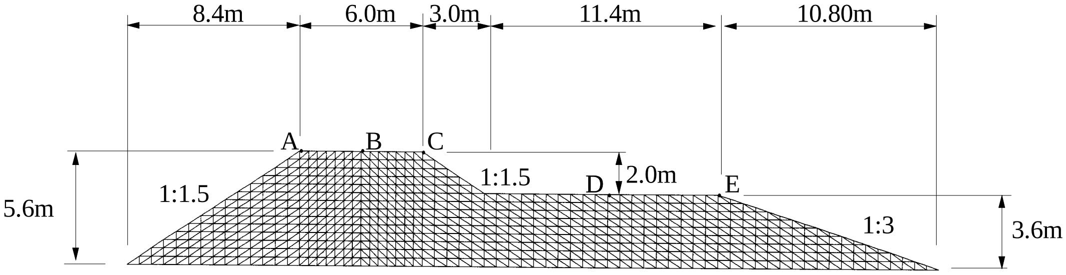



**Figure 12: FE model analyzed**

**Table 2: Properties of material used in the analysis**

| | | |
|---|---|---|
| Unit mass density | : $\gamma_t$ | 1.9 t/m$^3$ |
| Poisson's ration | : $\nu$ | 0.35 |
| Shear velocity | : $V_s$ | 200 m/sec |
| Cohesion | : $c$ | 3.92 kN/m$^2$ |
| Internal friction angle | : $\phi$ | 40 degrees |
| Residual cohesion | : $c_{rd}$ | 3.43 kN/m$^2$ |
| Residual internal friction angle | : $\phi_{rd}$ | 10 degrees |

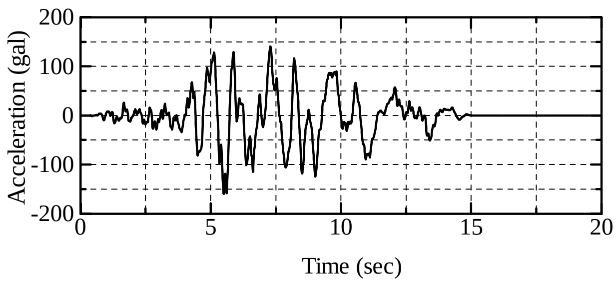



**Figure 13: Time history of input ground motion**

**4.3 Results of analysis**

Figure 14 shows the time histories of horizontal displacement at points A to E in Figure 12 and the collapsing process of the embankment is drawn in Figure 15.

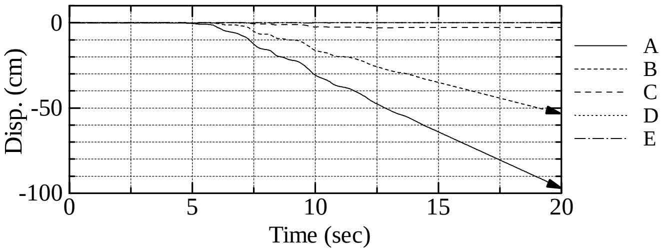
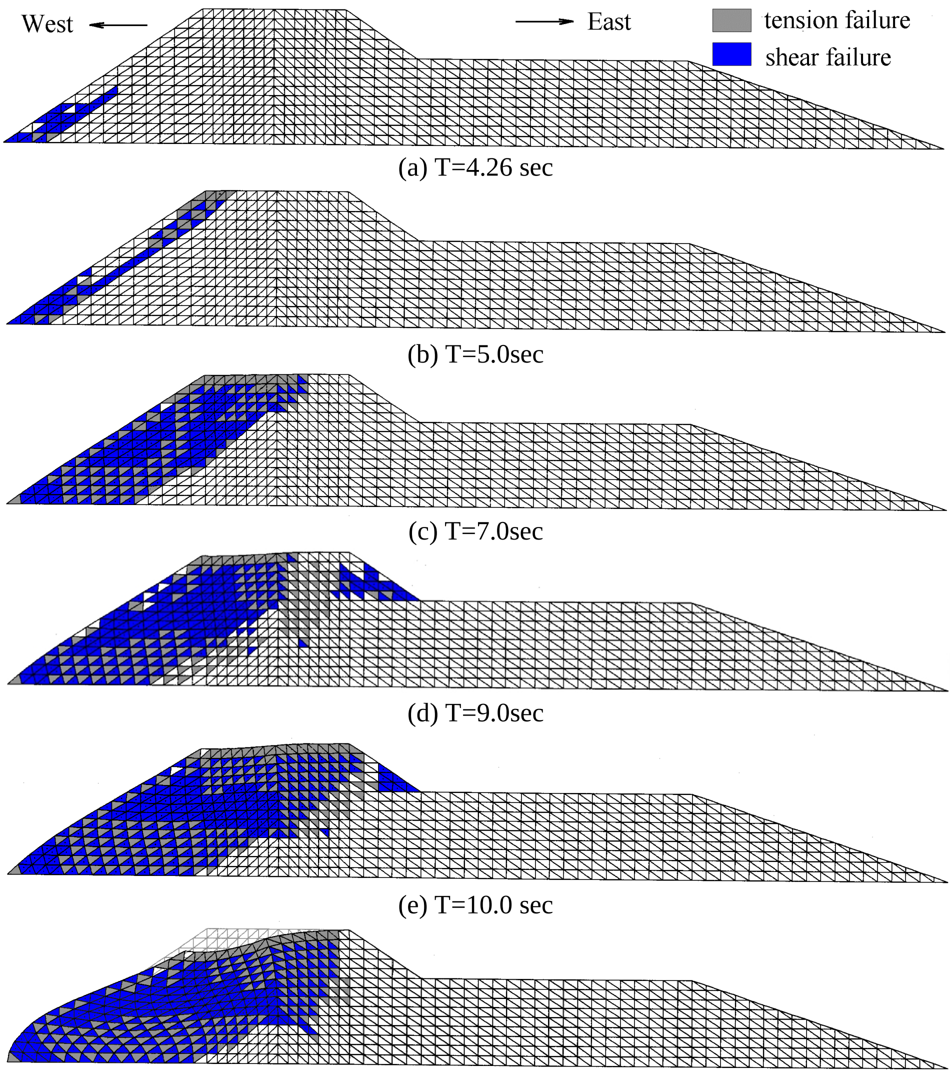



**Figure 14: Time histories of horizontal displacement**

As shown in Figure 14, the displacement at point A, the western top of the slope, and that at point B, the center of crown of the newer part, are gradually increasing after 15 seconds when the input ground motion become zero. This is because the embankment is collapsing with its own weight just after the earthquake. On the contrary, the displacements at the points D and E located on the old part are very small. The numerical analysis gives appropriate results for the damages of embankments.

Here the details of the collapsing process are discussed. Firstly the failure at the face of the western slope makes progress gradually from the toe to the top of the slope, as shown in Figure 15(a) and (b). It induces the tensile failure at the crown of newer part and forms the failure line which is shallow from and parallel to the surface of slope. Secondly the failure line develops toward the center of the embankment. The tensile failure spreads vertically and horizontally from the top of western slope as shown in Figure 15(c) and covers wide region of the center of the embankment as shown in Figure 15(d). The results qualitatively coincide well with the actual damage process since several rows of tensile cracks occur which are parallel to the embankment axis. However the displacement of point C, the eastern shoulder of newer crown is about 2cm in numerical analysis, where a large mount of deformation are practically observed after the earthquake. In the case where the residual internal friction angle and cohesion are set to half of values listed in Table 2, the older part has little deformation though the deformation at point C increases a little. In the cases where the amplitudes of input ground motion are varied, similar collapsing processes are obtained though the resulted deformations are changed.




(a) T=4.26 sec

(b) T=5.0sec

(c) T=7.0sec

(d) T=9.0sec

(e) T=10.0 sec

(f) T=20.0 sec

**Figure 15: Collapsing process of the embankment**

## CONCLUSION

(1) The elasto-plastic constitutive model of soil considering tensile failure is proposed to analyze earthquake response of earth structures such as embankment by FEM.

(2) The static inclination tests of embankments are simulated using the proposed constitutive model. The numerical results coincide well with the experiment in terms of the distribution of cracks and the inclination angle when the embankment collapses.

(3) The damaged embankment during the 1968 Tokachi-oki earthquake is simulated. The deformations of the embankment computed are similar with the observed ones.

(4) The numerical results show that the tensile failure is strongly related to the collapsing mechanism of earth structure under strong shaking.

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