Three-phase seepage-deformation coupled analysis for railway embankment damaged in 2004 Niigata-ken Chuetsu earthquake

Takaki Matsumaru i) and Ryosuke Uzuoka ii)

i) Research Associate, The University of Tokyo, 7-3-1 Hongo, Bunkyo-ku, Tokyo 113-8656, Japan.
ii) Professor, The University of Tokushima, 2-1 Minamijyouesanjima-cho, Tokushima 770-8506, Japan

ABSTRACT

In order to damage of embankments in 2004 Niigata-ken Chuetsu earthquake and to investigate the influence of the rainfall to the seismic behaviors of embankments, a series of numerical simulations were performed. In this simulation, the three-phase (soil, water, and air) coupled analysis was adopted for taking into account the behavior of unsaturated soil. It is shown that the quantitative reproduction of the deformation of the embankment could be made by applying the rainfall in seepage analysis before dynamic response analysis and that the influence of the heavy rainfall on the dynamic behavior was not small.

Keywords: unsaturated soil, rainfall, embankment, dynamic response analysis

1. INTRODUCTION

During the 2004 Niigata-ken Chuetsu earthquake occurred in Japan, a number of road and railway embankments collapsed in mountainous regions. It appears that the main reason of these damages was the increase of the degree of saturation before the earthquake, caused by the high rainfall intensity including heavy rainfall or typhoon. Therefore, it is important to investigate the influence of rainfall on the seismic resistance of embankments by numerical technique.

The coupled dynamic analysis based on the finite element method seems to be suitable for this problem because the reduction of the effective stress caused by the increase of pore water pressure during shaking can be considered in this method. Two-phase (soil and water) coupled analysis is one of the most simple and useful method in practical problems, and there are some studies successful in reproducing the seismic behavior of embankments affected by seepage water or rainfall (e.g., Uzuoka et al., 2005). However, recent studies by laboratory tests have suggested that unsaturated soil could be liquefied during earthquakes though unsaturated soil has the higher resistance compared to saturated soil due to the effect of pore air pressure (e.g., Tsukamoto et al., 2002 and Okamura and Soga., 2005). Usually, the water line in the embankments is low and most regions were in unsaturated conditions except the embankments located at a riverside or a seaside. In the two-phase coupled analysis, it is difficult to consider unsaturated soil, so three-phase (soil, water, and air) coupled analysis would be more suitable for evaluating the seismic resistance of embankments affected by seepage water or rainfall. However, there are a small number of researches applying three-phase coupled analysis focusing on the dynamic behavior of embankments affected by rainfall. (e.g., Mori et al., 2011 and Higo et al., 2013).

The authors have succeeded in conducting the three-phase coupled analysis to reproduce the behavior of the shaking table tests of the embankment affected by the rainfall or seepage water and showed the advantage of the three-phase coupled analysis (Matsumaru and Uzuoka, 2014a and 2014b). In this paper, numerical analysis with a focus on the dynamic behavior of the selected embankment damaged by this earthquake was performed with this three-phase coupled analytical method. In order to describe the behavior of embankment materials affected by cyclic loading precisely, the unsaturated cyclic triaxial tests were conducted and the material parameters of the constitutive model were determined appropriately.

2 NUMERICAL METHOD

2.1 Density and stress of three-phase porous media

The overall density of mixture \( \rho \) can be expressed as

\[
\rho = \rho^s + \rho^w + \rho^a
\]

\[
\rho^s = (1-n)\rho^{sR} + n\rho^{sR} + \left(1-s^w\right)\rho^{aR}
\]

where \( \rho^s \), \( \rho^w \) and \( \rho^a \) are the partial densities of soil skeleton, pore water and air respectively, \( \rho^{sR} \), \( \rho^{wR} \) and \( \rho^{aR} \) are the real densities of each phase, \( n \) is the porosity and \( s^w \) is the degree of water saturation.
The skeleton stress which means the effective stress of unsaturated soil is expressed as
\[ \sigma = \sigma' + \sigma^w = \sigma' - p^w \mathbf{I} + s^w (p^w - p^w) \mathbf{I} \] (2)
where \( \sigma' \) is the skeleton stress tensor (e.g. Gallipoli et al. 2003) defined as positive in extension, \( p^w \) is the pore water pressure and \( p^w \) is the pore air pressure. These pressures are defined as positive in compression.

2.2 Soil water characteristic curve
The SWCC (soil water characteristic curve) is assumed as a logistic function (Sugii et al. 2002), derived by
\[ s^w = \left( s^w_s - s^w_r \right) \exp \left( a + p^p + b_{lg} \right) \] (3)
where \( s^w_s \) is the saturated (maximum) degree of saturation, \( s^w_r \) is the residual (minimum) degree of saturation and \( s^w_w \) is the effective water saturation. The relationship between \( s^w_w \) and suction \( p^p \) is assumed as a logistic function with the material parameters \( a_{lg} \), \( b_{lg} \) and \( c_{lg} \). The logistic SWCC is a continuous function at \( p^p = 0 \); therefore the convergence in the iterative numerical scheme can be achieved.

The permeability coefficient of water and air are assumed to be dependent on the effective water saturation as
\[ k^w = k^w_s \left( s^w_w \right)^{\alpha_p} \quad k^p = k^p_s \left( 1 - s^w_w \right)^{\alpha_p} \] (4)
where \( k^w_s \) is the saturated (maximum) coefficient of water permeability, \( k^p_s \) is the dry (maximum) coefficient of air permeability, \( \alpha_p \) and \( \eta_p \) are the material parameters.

2.3 Constitutive equation for skeleton stress
A simplified constitutive model for saturated sandy soil is used for unsaturated soil with using a skeleton stress in place of an effective stress of saturated soil. Assuming that plastic deformation occurs only when the deviatoric stress ratio changes, the yield function is assumed as
\[ f = \sqrt{3} \sqrt{2 \left[ \mathbf{h} - \mathbf{a} \right]} - k = 0 \] (5)
where \( \mathbf{h} \) is the stress ratio and \( k \) is the material parameter which defines the elastic region. The kinematic hardening parameter (back stress) \( \mathbf{a} \), and its nonlinear evolution rule (Armstrong and Frederick, 1966) is assumed as
\[ \dot{\mathbf{a}} = \frac{2}{3} \mathbf{b} \dot{\mathbf{e}}^p - \mathbf{a} \dot{\mathbf{e}}^p \quad \dot{\mathbf{e}}^p = \left[ \mathbf{k}^p \right] \] (6)
where \( a \) and \( b \) are the material parameters, and \( \dot{\mathbf{e}}^p \) is the plastic deviatoric strain rate tensor.

Using the following equation, the hardening parameter \( \mathbf{a} \) is reduced according to the value of the equivalent plastic strain \( \dot{\mathbf{e}}^{p(e)} \) which is reset when the direction of loading changes, assumed as
\[ a = a_0 - \frac{(a_0 - a_1)}{1 + (a_0 - a_1) \exp (-C_f \tau^{p(e)})} \] (7)
where \( a_0 \) is the initial value of the material parameter \( a \), \( a_1 \) is the lower limit value of \( a \) and \( C_f \) is the parameter which controls the amount of the reduction \( a \). In order to describe plastic strain rate more precisely, the non-associated flow rule (e.g. Oka et al. 1999) was adopted, assumed as
\[ \dot{\mathbf{e}}^p = \dot{\mathbf{e}}^{p(e)} = \dot{\mathbf{e}}^p \quad \dot{\mathbf{e}}^p = \frac{\dot{\mathbf{e}}^{p(e)} + \dot{\mathbf{e}}^{p(a)}}{2} \] (8)
where \( \dot{\mathbf{e}}^{p(e)} \) and \( \dot{\mathbf{e}}^{p(a)} \) are the material parameters depending on the situations of stress and strain. From the equation (8), the plastic deviatoric strain rate and the plastic volumetric strain are derived as
\[ \dot{\mathbf{e}}^p = 2\dot{\mathbf{e}}^{p(e)} = \gamma \dot{\mathbf{e}}^p = (3\dot{\mathbf{e}}_1 + 2\dot{\mathbf{e}}_2) \dot{\mathbf{e}}_p = D \gamma \dot{\mathbf{e}}^p \] (9)
where \( \dot{\gamma} \) is the hardening coefficient and \( D \) is the coefficient of dilatancy. In order to change the volumetric strain rate depending on the suction \( p^p \) of unsaturated soil, we propose an equation which describes the relationship between the coefficient of dilatancy and the suction \( p^p \), assumed as
\[ D = D_0 + \left( D_0 - D_0 \right) \exp \left( -p^p / p^p_{ref} \right) \quad \left( p^p > 0 \right) \] (10)
\[ D = D_0 \quad \left( p^p \leq 0 \right) \]
where \( D_0 \) is the initial value of the coefficient of dilatancy \( D \), \( D_0 \) is the lower limit value of \( D \) and the \( p^p_{ref} \) is the parameter which controls the amount of the reduction \( D \). The equation (10) means that the plastic volumetric strain tends not to occur if the suction \( p^p \) is large.

With the non-associated flow rule, the plastic potential function is assumed as
\[ g = \sqrt{3} \sqrt{2 \left[ \mathbf{h} - \mathbf{a} \right]} + M_m \ln \left( \frac{p'_{\mathbf{a}}}{p'_{\mathbf{a}}^0} \right) \] (11)
where \( M_m \) is the critical state stress ratio, and \( p'_{\mathbf{a}} \) is \( p' \) when \( \left[ \mathbf{h} - \mathbf{a} \right] = 0 \).

Finally the elastic bulk modules are assumed as
\[ K^e = -K^e p' \quad G^e = -G^e p' \] (12)
where \( K^e \) is the elastic bulk modulus, \( G^e \) is the elastic shear modulus, \( K^e \) and \( G^e \) are the dimensionless elastic modules, respectively.
2.4 Constitutive equation for skeleton stress

The governing equations are derived by Uzuoka and Borja (2012) with following assumptions. 1) The conditions are isothermal, 2) the soil particles are incompressible, 3) the mass exchange among phases can be neglected, and 4) the material time derivative of relative velocities and advection terms of pore fluids to the soil skeleton can be neglected.

The momentum balance equation of the overall three phase material is derived as

\[
\rho \mathbf{a}^t = \nabla \mathbf{\sigma} - \left\{ s^w \rho^w + (1 - s^w) \rho^a \right\} \mathbf{I} + \rho \mathbf{b} \tag{13}
\]

where \( \mathbf{a}^t \) is the acceleration of solid skeleton and \( \mathbf{b} \) is the gravity acceleration vector. The mass and momentum balance equations of the pore water and air are derived as

\[
\left( n s^w \frac{\rho^w}{K^w} - n \rho^a \right) \frac{D s^w}{Dt} + n \rho^w \frac{D s^w}{Dt} + s^w \rho^w \frac{\nabla \mathbf{v}}{\rho^w} + \nabla \left( \frac{n s^w}{g} \rho^w \mathbf{g} \right) = \mathbf{s} \tag{14}
\]

\[
\left( \frac{n(1 - s^w)}{\Theta R} - n \rho^a \right) \frac{D \rho^a}{Dt} + n \rho^w \frac{D \rho^a}{Dt} + (1 - s^w) \frac{\nabla \mathbf{v}}{\rho^a} + \nabla \left( \frac{n(1 - s^w)}{g} \rho^a \mathbf{g} \right) = \mathbf{s} \tag{15}
\]

where \( K^w \) is the bulk modulus of the pore water, \( c \) is the specific water capacity, \( \Theta \) is the absolute temperature, \( R \) is the specific gas constant of air, \( \mathbf{v} \) is the velocity of soil skeleton and \( g \) is the acceleration of gravity.

Weak forms of the equations (13)–(15) are implemented in a finite element formulation. Newmark implicit scheme is used for time integration. The primary variables are the second-order material time derivative of displacement of soil skeleton \( \mathbf{a}^t \), pore water pressure \( p^w \) and pore air pressure \( p^a \). The weak forms are linearized and solved by Newton-Raphson method iteratively at each time step.

In the finite element formulation, Galerkin method and isoparametric 8-node elements are used. The soil skeleton displacement and the fluid pressures are approximated at 8 nodes and 4 nodes respectively to avoid volumetric locking.

3 CONDITIONS OF SIMULATIONS

3.1 Outline of damaged embankment

For conducting numerical simulations, we choose a collapsed embankment which supported a railway track (Morishima et al. 2005). Fig. 1 shows the illustration of the collapsed embankment. The damaged area had a length of about 90 m and a height of 2 to 7 m. The amount of collapsed soil was estimated to be about 9,900 m³.

This embankment is located in a valley eroded by the Shinano River. The bedrock is medium-grained sandstone, and siltstone is piled on top of the bedrock. The soil for the embankment was gravelly sand.

Fig. 1. Cross-section of collapsed embankment.

3.2 Analytical model

Fig. 2 shows the finite element modeling of the embankment. The embankment and bedrock were divided in square eight-node elements. The soil displacement at the bottom boundary is fixed in all directions and the lateral boundaries are vertical rollers. As the boundary conditions of the pore water pressure, the bottom and lateral boundaries are impermeable. At a part on the surface of the embankment and the ground, the quantity of flow at each node was according to the observed data of rainfall, though this boundary becomes permeable if the pore water pressure reached zero. As pore air pressure, the right and bottom boundaries are impermeable, while air drainage is allowed at the surface of the embankment and the ground. The surface at the left boundary is supposed to be on the same level as the water level of the Shinano River, so the boundary of the pore water pressure at this place is permeable with zero water pressure.

Fig. 2. Finite element modeling of embankment

3.3 Material parameters

The embankment was modeled by the proposed elasto-plastic model and the ground was by the elastic model. The parameters of the embankment material were determined from unsaturated cyclic triaxial tests.

The physical properties of the embankment material were as follows; the particle density \( G_s \) was 2.629; 50% diameter on the grain size diagram \( D_{50} \) 0.134mm; the uniformity coefficient \( U_c \), 11.65; and the fine fraction content \( F_c \); 23.3 %. The initial dry density was about 1.29 g/cm³. For the triaxial tests, the degree of water saturation was from about 48.0 % to 89.8 % according to the air pressure controlled during the isotropic
consolidation process. The pore water pressure was almost zero after the consolidation and the pore air pressure increased with the decrease in water saturation. The net stress was about 98 kPa for all specimens and the mean skeleton stress varied with the initial suction dependent on initial water saturation.

The cyclic shear was applied to the specimen under undrained air and water conditions. The input axial strain was eleven cycles of the triangular waves with multi step amplitudes whose single amplitudes were 0.1, 0.2, 0.5, 1.0, 1.5, and 2.0. The frequency of the sinusoidal wave was 0.001 Hz. This loading rate is slow enough to achieve an equilibrium condition between air and water pressure.

Fig. 3 shows the time histories of pore water pressure of experimental and simulated results. The pore water pressure increased during cyclic undrained shear. The mean skeleton stress attained zero in Case 3, which means that the specimen liquefied completely. On the other hand, the strength and stiffness was maintained during the cyclic shear in Case 1 and Case 2. In the simulated results, the model well reproduced the overall tendency of the test results of each case.

3.4 Procedure of analyses

We investigated the seismic behavior by performing the dynamic response analysis after conducting the seepage analysis in order to determine the water content in the embankment just before the earthquake. The seepage analysis was carried out in two phases. In the first phase, the steady state of the collapsed embankment subjected to the annual rainfall was determined. In the second phase, the degree of saturation of the embankment immediately before the earthquake was estimated by considering to what extent the actual time history of the precipitation affects that estimated from the steady state determined in the first phase. For the first phase, the time history of the rainfall is shown in Fig. 4 (a); 19 mm/day once every three days and 0 mm on the other days. This is based upon the annual average precipitation in Ojiya city, which is close to the relevant area. In order to calculate the steady state of the degree of saturation, the time duration of the precipitation for the first phase was set at three years, which was obtained from previous trial analyses. For the second phase, the time history of the rainfall intensity with the accumulated rainfall is shown in Fig. 4 (b): this is the observed time history of the precipitation from 1 July to 22 October. The accumulated rainfall observed in this period showed over 1,000 mm. The average rainfall calculated from the accumulated rainfall recorded in this period is about 9.1 mm/day, which is close to 1.5 times larger than the annual rainfall, 6.3 mm/h.

Fig. 5 shows the estimated ground motion employed in the dynamic analysis. The maximum acceleration of the motion was 665 gal.

![Time history of ground motion.](image)

We conducted three cases of simulations. In Case 1, the dynamic response analysis was performed without conducting seepage analysis for the second phase. In Case 2, the seepage analysis of the first phase was conducted before the dynamic response analysis. In order to investigate the influence of the rainfall on the seismic behavior, the simulation of Case 1 was performed for comparison. In Case 3, it was assumed that the rainfall poured on the slope surface of the bedrock above the embankment seeped into the embankment at its boundary with the bedrock, as shown in the Fig. 3. Because the slope surface above the top of the embankment is covered by concrete in order to avoid erosion or seepage, it seemed that the rainfall would be gathered on the boundary between the embankment and the slope. For this reason, we conducted the simulation of Case 3 for comparison.

For seepage analysis, the time increment is 1 day for the first phase and 1 hour for the second phase. Rainfall was poured into the nodes on the surface of the embankment and bedrock. For the dynamic analysis, the coefficients in Newmark implicit time integration are 0.6 and 0.3025. The time increment is 0.002 seconds for all cases. In the dynamic analyses, the
viscos damping of the solid skeleton is considered. The damping coefficient is 0.002.

4 RESULTS AND DISCUSSIONS

4.1 Distribution of degree of saturation

Fig. 6 shows the distributions of the degree of saturation in the three cases before shaking, obtained from the seepage analyses. In Case 1, where only the seepage analysis for the first phase was conducted, a large value of the degree of saturation was observed at the toe of the embankment. In the result of the simulation in Case 2, where the seepage analysis of the second phase was also conducted, the zone where a large value of water content in observed spread more widely around the toe than in Case 1. It seemed that the rainfall observed before the earthquake had a large influence in terms of the seepage into the embankments. In the simulation of Case 3, where the rainfall dropped on the upper slope was gathered and poured into the embankment around its boundary with the ground, the degree of saturation around the top showed a larger value though the distribution of the saturation at the toe of the embankment was the same as the result in Case 1.

![Distributions of degree of saturation before earthquake](image)

(a) Case 1  (b) Case 2  (c) Case 3

Fig. 6. Distributions of degree of saturation before earthquake.

4.2 Deformation of embankment

Fig. 7 shows the time histories of the vertical displacement at n174, located in the middle of the slope of the embankment, and at n485 at the top. In all cases, at both points, the displacement was increased after 5 second, because the large motion was inputted at this time. Compared to the displacement obtained in the simulation of Case 1, the displacement at n174 of Case 2 was larger. In Case 2, the degree of saturation showed a larger value from the toe to the middle of the embankment as shown in Fig. 7, so this increase of the water saturation would cause the larger displacement. On the other hand, the displacement at n485 obtained in Case 2 was almost the same as that in Case 1. Around the top of the embankment, the degree of the saturation was not increased by the rainfall before the earthquake, so the displacement at the top would not be increased.

In the simulation of Case 3, the displacement obtained at n485 especially showed a larger value than in Case 1 and Case 2. Around the top of the embankment, the degree of saturation was increased, so the larger displacement would be obtained at the top. The maximum value of the measured settlement at the top of the embankment was about 7.0m mentioned in 3.1. The displacement obtained in the simulation of Case 3 almost coincided with this value.

![Time histories of vertical displacement](image)

(a) n174  (b) n485

Fig. 7. Time histories of vertical displacement.

4.3 Behavior of embankment

Now, we focused on the differences of the seismic behavior among three cases. Fig. 8 shows the time history of the mean skeleton stress reduction ratio (MSSRR) at the element e79 obtained in the simulation of Case 1 and Case 3. In Case 1, the maximum value of the MSSRR became about 0.8 momentarily and showed 0.5 at the end without reaching 1.0, so the liquefaction did not occur at this element. On the other hand, in Case 3, this element would liquefy at about 8 second because the MSSRR reached 1.0. The same tendency as in Case 3 was also observed in the result of Case 2 though leaving out the figure. These results indicated that the behavior of the element depends on the initial degree of saturation.

![Time history of MSSRR at e79](image)

(a) Case 1  (b) Case 3

Fig. 8. Time history of MSSRR at e79.

Fig. 9 shows the stress-strain relationship at e79, located under the middle of the embankment, as show in Fig. 2. The stress-strain relationship shows the relationships between the strain difference and the shear stress. Using the horizontal strain $\varepsilon_x$ and vertical strain $\varepsilon_y$, the strain difference is defined as $\varepsilon_y-\varepsilon_x$. In Case 1, the strain difference was not increased since the shear stress was maintained during shaking. On the other hand, in Case 3, the shear stress was decreased during shaking because the liquefaction would occur during earthquakes as shown in Fig. 8. For this reason, the strain difference was increased largely. The strain
difference showed a plus value, which means that the element was compressed in the vertical direction and stretched in the horizontal direction. These differences of element behaviors in both cases would affect the scale of deformation. The increase of the strain difference as shown in Fig. 9 (b) was also observed under the top of the embankment only in Case 3.

![Stress-strain relationship at σ79.](image)

Fig. 9. Stress-strain relationship at σ79.

5 CONCLUSIONS

In order to reproduce the damage of the embankment in 2004 Niigata-ken Chuetsu earthquake and to investigate the influence of rainfall to the seismic behavior, the numerical analysis with the three-phase (soil, water, and air) coupled analysis based on porous media theory was conducted. In this simulation, in order to investigate the influence of the amount of the rainfall on the seismic behavior of the embankment, a series of numerical simulations were conducted by changing the conditions of the rainfall before the earthquake. Furthermore, the parametric studies were conducted to explain the effectiveness of the drainages or impermeable conditions on the seismic behavior. As a result, we achieved the following conclusions:

1. The unsaturated seepage analysis was performed for reproducing the distributions of the degree of saturation just prior to the earthquake. As a result, the degree of saturation was increased in the embankment around its boundary with the ground. Compared to the result obtained from the simulation only in consideration of the annual rainfall, the degree of saturation was increased more widely by applying the rainfall just before the earthquake.

2. The dynamic response analysis was conducted in order to reproduce the damage of the embankment after the seepage analysis. By the increase of the water content due to the heavy rainfall just before the earthquake, the dynamic response analysis showed the larger deformation of the embankment, compared to that calculated by the analysis with the water content obtained from the annual rainfall. Furthermore, the displacement calculated by the simulation almost coincided with the maximum observed residual settlement at the top of the embankment.

3. From the behavior of the element in the embankment, it was revealed that the liquefaction would occur during shaking. The occurrence of the liquefaction would cause the large strain in a wider region when heavy rainfall was assumed in the seepage analysis. On the other hand, the increase of the mean skeleton stress reduction ratio was smaller in the case with only the annual rainfall. The amount of the rainfall before the earthquake affected largely the seismic behavior of the embankment.

REFERENCES


