NUMERICAL ANALYSES OF THE DAMAGED DIKES IN THE 1995 HYOGO-KEN NANBU EARTHQUAKE

Osamu MATSUO[1], Mitsu OKAMURA[2], Ryosuke UZUOKA[3] And Masaya MIHARA[4]

SUMMARY

River dikes were severely damaged during the 1995 Hyogo-ken Nanbu earthquake. The purpose of this study is to clarify the mechanism of the damage through the numerical analyses. The effective stress analysis was applied to a set of embankment sections along the Yodo-gawa River that were damaged and non-damaged during the 1995 Hyogo-ken Nanbu earthquake. As a result, the numerical method qualitatively reproduced a difference in damage at both sites. The results showed that the foundation sandy soils would have liquefied at both sites. The large settlement at the damaged site was due to the lateral spreading of the sandy soils underlying the embankment. Although the foundation sandy soils liquefied at the non-damaged site, the crest settlement was much smaller than that at the damaged site. Effects of the cut-off wall and the surface cover layer on the embankment settlement appeared to be small. A cohesive soil layer in the sandy layers was considered to have played an important role in reducing the crest settlement. The modeling of deformation characteristic for a cohesive layer becomes important for the accurate prediction of the embankment settlement.

INTRODUCTION

Soil embankments such as river dikes, road embankments and earth dams have been frequently damaged during past major earthquakes. Previous case studies [e.g. Sasaki, 1998] have shown that such embankment damages were destructive in most cases when the underlying saturated sandy soils liquefied. Reduced scale model tests have been performed to understand the seismic behavior of soil embankments on liquefiable foundation soils. These reduced scale model tests include shaking table tests [e.g. Koga and Matsuo, 1990] and more recently dynamic centrifuge tests [e.g. Koseki et al., 1994]. The factors, which affect the seismic behavior of the embankment, have been investigated through the experimental results.

The effective stress analysis technologies may be one of the promising tools that can predict the behavior of real scale embankments. The authors have examined the dynamic response FE analysis that incorporates a cyclic elasto-plastic constitutive model for sand and Biot's two phase mixture theory. The numerical method has been applied to the dynamic centrifuge tests [Matsuo et al., 1997] and the damaged river dikes during 1993 Hokkaido Nansei-oki earthquake [Matsuo et al., 1998]. They concluded that the numerical method reproduced the permanent embankment settlement ranging from about twice to a half of the measured ones for the quite well documented case records.

The river dikes along the Yodo-gawa River were severely damaged in 1995 Hyogo-ken Nanbu earthquake [Matsuo, 1996]. The settlement at Torishima (kilopost 0.2-2.2 km) exceeded 2 m for a length of 1.4 km, with a maximum settlement of 3 m. Kilopost means the distance from the river mouth. On the other hand, the settlement at Takami (kilopost 3.0-4.2 km) was hardly observed. The local difference in damage between near sties have been observed in past damaged embankment [Sasaki, 1998]. The factors, which caused the local difference in damage, should be examined to assess the seismic safety of embankments. The detailed soil investigation at Torishima and Takami were conducted after the earthquake.

[1] *Earthquake Disaster Prevention Research Center, Public Works Research Institute, Ministry of Construction, Tsukuba, Japa*
[2] *Earthquake Disaster Prevention Research Cntr, Public Works Research Inst, Tsukuba, Japan E-mail: okamura@pwri.go.jp*
[3] *Earthquake Disaster Mitigation Research Center, RIKEN, Miki, JapanE-mail: uzuoka@miki.riken.go.jp*
[4] *Dept. of Civil Engineering, Hazama Corporation, Tokyo, Japan E-mail: mihara@hazama.co.jp*

The purpose of this study is to clarify the mechanism of the damage by the effective stress method which has been verified by the authors. First, the numerical method was applied to a set of embankment sections that were damaged in Torishima and non-damaged in Takami. Secondly, the parametric studies were carried out to evaluate the effect of the following features on the crest settlement; 1) the cut-off wall at the toe of embankment, 2) the cohesive soil layer in the sandy layer, 3) the surface cover layer. The factors leading to a difference in damage between two sites were discussed through the numerical analyses.

DAMAGED RIVER DIKES

Figure 1 shows the epicenter of 1995 Hyogo-ken Nanbu earthquake, the measured horizontal maximum acceleration at ground surface [Shibata et al., 1996] and the area where the dikes along Yodo-gawa River were damaged. The damaged dikes are located within 40 km from the epicenter. According to the observed acceleration records near the damaged dikes, the maximum surface acceleration at the site was considered to exceed 200 Gal. Major damaged dikes were located near the river mouth. Figure 2 shows the damaged area and the two sites for the analysis along the Yodo-gawa River. The Torishima Dike (kilopost 0.2-2.2 km) in the left levee was severely damaged as shown in Photograph 1 [Matsuo, 1996]. The cross section as illustrated in Figure 3 was observed during the investigation by excavation of the damaged section [Sasaki, 1998]. The core of the dike was a soil embankment, and the river side surface was protected by concrete parapet wall. The concrete parapet at the top of the levees tilted and settled. The parapet slid about 8 m into the river. The embankment also severely settled and seemed to fall into the foundation as shown in Figure 3. The settlement exceeded 2 m on the kilopost 0.5-1.9 km, with a maximum of 3 m. Sand boils along the fissures were observed on the ground surface along the dike. Lateral movements and heaving of the ground surface occurred in the residential area. These facts suggest that soil liquefaction triggered the damage.

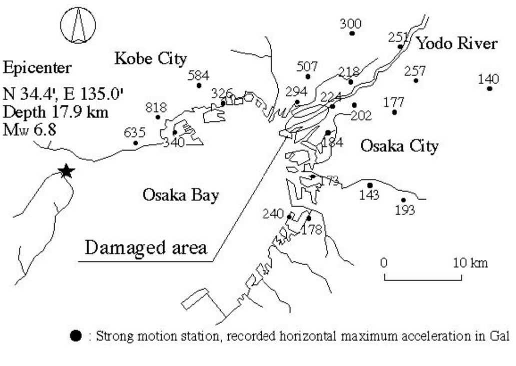
We carried out the numerical simulation for two sites as shown in Figure 2. The No. 1 site is located on the kilopost 1.4 km in Torishima, and the embankment damage was destructive and the settlement of the crest was about 2.7 m. On the other hand, the No. 2 site is located on the kilopost 3.0 km in Takami, and the major damage of the embankment was not observed. Although both sites were about 1.6 km away, their damages at both sites were very contrary with each other. Figure 4 shows the soil profiles and the cross sections of the embankment at both sites. These figures were drawn based on the boring data obtained after the earthquake. The soil profiles are basically similar each other. The dike is underlain by the Holocene deposit with a thickness of about 30 m. The deposit consists of, from the top, the upper sandy layer (As2), clay layer (Ac) and the lower sandy layer (As1). The As2 layer is also divided into the upper sublayer (As2-2) and the lower sublayer (As2-1). The soil of As2-2 layer is finer than that of As2-1 layer judging from SPT N-value and fine content at both sites. In addition, the height of the embankment is almost same at both sites. However, the following different features between two sites are observed.

1) The cohesive layer of about 2 m in thickness is sandwiched in the As2 layer at only No. 2 site.

2) The different types of cut-off walls are installed at the toe of embankment. The bending stiffness of sheet pile at No. 1 site was about ten times larger than RC (Reinforced Concrete) wall at No. 2 site.

3) The surface cover layer with the width of about 40 m was made at river side of No. 2 site. The surface ground elevation of river side at No. 2 site is about 4 m higher than that at No. 1 site.

The effect of these different features on the deformation of embankment is discussed through numerical parametric studies.



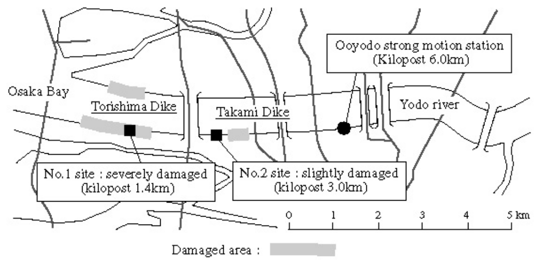


Figure 1: Location of Yodo-gawa river **Figure 2: Location of damaged dikes**
[Reproduced from Shibata et al., 1996 with modification]

Photograph 1: Damaged to the Torishima Dike [Matsuo, 1996]

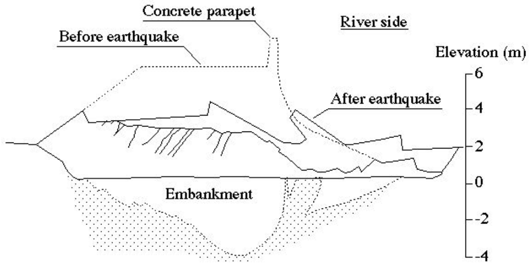


Figure 3: Cross section of the Torishima Dike [Sasaki, 1998]

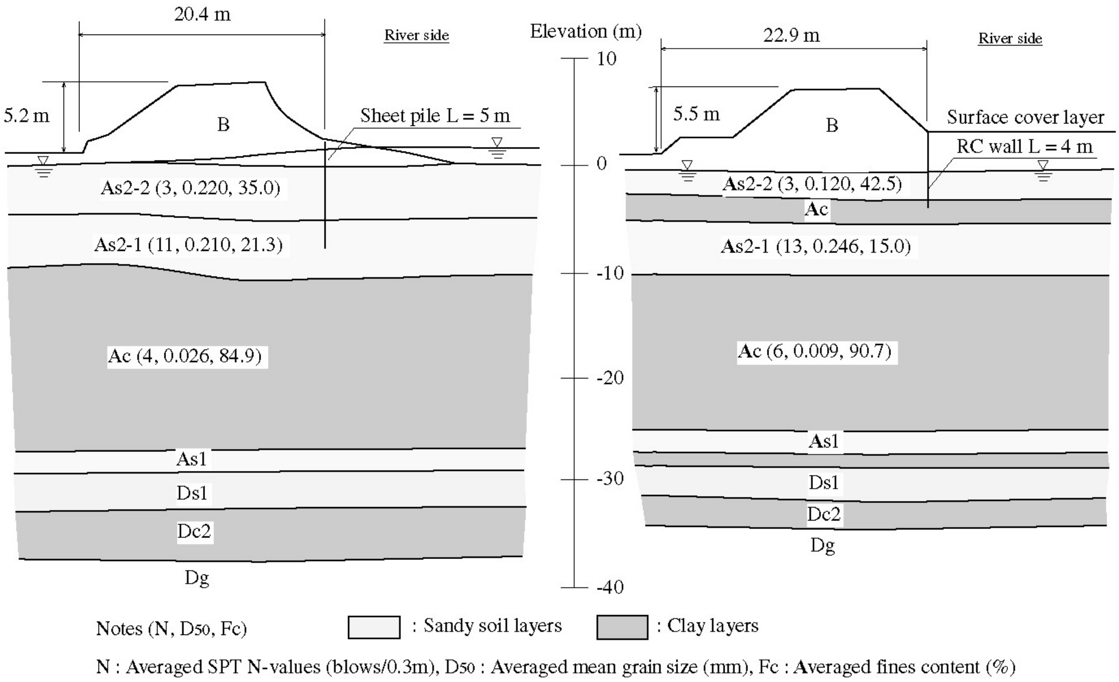


Figure 4: Soil profiles and embankment configurations

NUMERICAL METHOD

Governing Equations

In this study, the governing equations of for the coupling problems between soil skeleton and pore water were obtained with the two phase mixture theory [Biot, 1962]. Using a u-p (displacement of the solid phase - pore water pressure) formulation [Zienkiewicz and Bettes, 1982], a simple and practical numerical method for the two-dimensional liquefaction analysis was formulated. The finite element method (FEM) has been usually used for the spatial discretization of the governing equations. In this study, however, the finite element method (FEM) was used for the spatial discretization of the equilibrium equation, while the finite difference method (FDM) was used for the spatial discretization of the pore water pressure in the continuity equation [Akai and Tamura, 1978]. The accuracy of the proposed numerical method was verified by Oka et al. [1994] through a comparison of numerical results and analytical solutions for transient response of saturated porous solids. As details of this method were given in Oka et al. [1994], only a brief description of the method is given below. The governing equations are formulated by the following assumptions; 1) the infinitesimal strain, 2) the smooth distribution of porosity in the soil, 3) the small relative acceleration of the fluid phase to that of the solid phase compared with the acceleration of the solid phase, 4) incompressible grain particles in the soil. The equilibrium equation for the mixture is derived as follows:

$$\rho \ddot{u}_i^s = \sigma_{ij,j} + \rho b_i \tag{1}$$

where is the overall density, $\ddot{u}_i^s$ is the acceleration of the solid, σ_{ij} is the total stress tensor and bi is the body force. The continuity equation is derived as follows:

$$\rho^f \ddot{\varepsilon}_{ii}^s - p_{,ii} - \frac{\gamma_w}{k} \dot{\varepsilon}_{ii}^s + \frac{n\gamma_w}{kK^f} \dot{p} = 0 \tag{2}$$

where f is the density of the fluid, p is the pore water pressure, w is the unit weight of the fluid, k is the coefficient of permeability, ε_{ii}^s is the volumetric strain of the solid, n is porosity and Kf is the bulk modulus of the fluid.

Constitutive Models

The constitutive equation used for sand is a cyclic elasto-plastic model [Oka et al., 1992, Tateishi et al., 1995]. The constitutive equation is formulated by the following assumptions; 1) the infinitesimal strain, 2) the elasto-plastic theory, 3) the non-associated flow rule, 4) the concept of the overconsolidated boundary surface, 5) the non-linear kinematic hardening rule. The performance of the constitutive model was verified by Tateishi et al. [1995]. The model succeeded in reproducing the experimental results well under various stress conditions, such as isotropic and anisotropic consolidated conditions, with and without the initial shear stress conditions, principal stress axis rotation, etc.

The constitutive equation used for clay is a cyclic elasto-viscoplastic model [Oka, 1992]. The model is based on almost the same assumption of the elasto-plastic model for sand, except the flow rule of the model that is different from the model for sand. The model adopts the viscoplastic flow rule that can take the rate dependency of cohesive soil into account.

NUMERICAL CONDITIONS

We applied the cyclic elasto-plastic model for sand to the embankment (B layer in Figure 4) and sandy soil layers (As1, As2 and Ds1 in Figure 4), and the cyclic elasto-viscoplastic model for clay to the cohesive layers (Ac and Dc2 in Figure 4). The model parameters are divided to two categories, basic parameters and fitting parameters. The basic parameters, void ratio, swelling index, initial shear modulus etc., were directly determined by the in-situ tests and laboratory tests using undisturbed samples obtained from each soil layer. The physical property tests, undrained monotonic and cyclic triaxial tests were carried out after the earthquake. The fitting parameters were determined by adjusting technique. For the sandy soils, the parameter values were selected by trial and error in order to describe the liquefaction strength curve. For the cohesive soils, the parameters were estimated in order to describe the dynamic deformation characteristics considering a loading rate. The liquefaction strength curves and the dynamic deformation characteristics were obtained by the undrained cyclic triaxial tests using the undisturbed samples from each soil layer. The liquefaction strength curves obtained by the laboratory tests and simulated by the cyclic elasto-plastic model are shown in Figure 5 for As2 layers at both sites. The averaged strength of experimental data was used for setting the model strength at No. 1 site because the experimental strength was dispersed. As a result, the simulated liquefaction strength of both As2 layers at both sites was almost same. The cyclic shear stress ratio, required to cause 5% axial strain in 20 cycles, was about 0.20 at both sites.

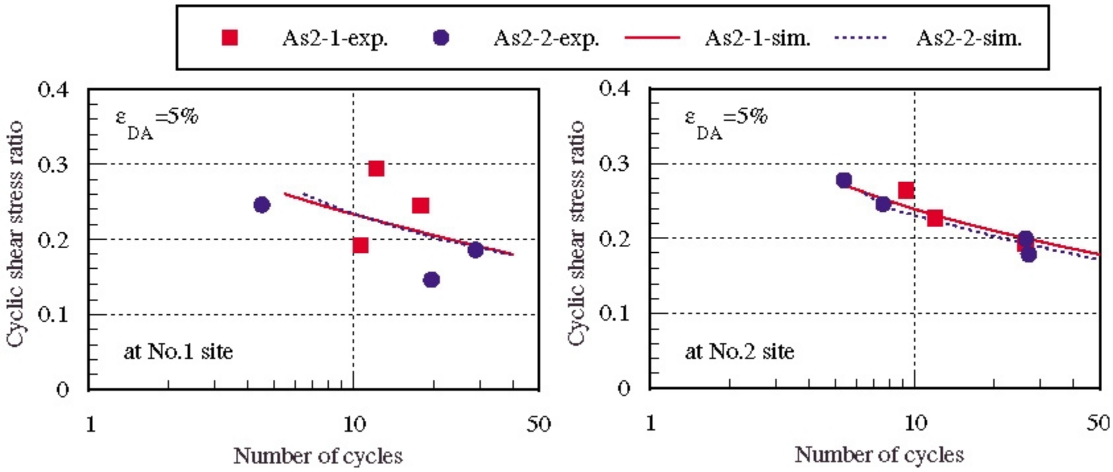


Figure 5: Liquefaction strength curves

Figure 6 shows (a) the calculated input motion for both sites and (b) the recorded motion at Ooyodo strong motion observation site as shown in Figure 2. The input motion was estimated at top of Dg layer which was assumed to be the common base layer, because Dg layer existed at No. 1, No. 2 and Ooyodo site respectively. The input motion at the base was deconvoluted from the transversal component of the recorded acceleration on the ground surface at Ooyodo site by using one-dimensional multiple reflection model. The input motion was used at the bottom of Dc2 layer that was treated as a rigid base of the FEM model. The time integration step of 0.001 was adopted, and the simulation was performed for 40 seconds. The Rayleigh damping proportional to initial stiffness was used to reproduce the damping in high frequency domain.

The FEM models for both sites were made from the soil profiles and embankment configuration as shown in Figure 4. The cut-off walls were modeled by beam elements, but the concrete parapet covering the embankment were not modeled. The initial stress state was computed by the static elasto-perfectly-plastic analysis, in which the Drucker-Prager type failure surface model was employed.

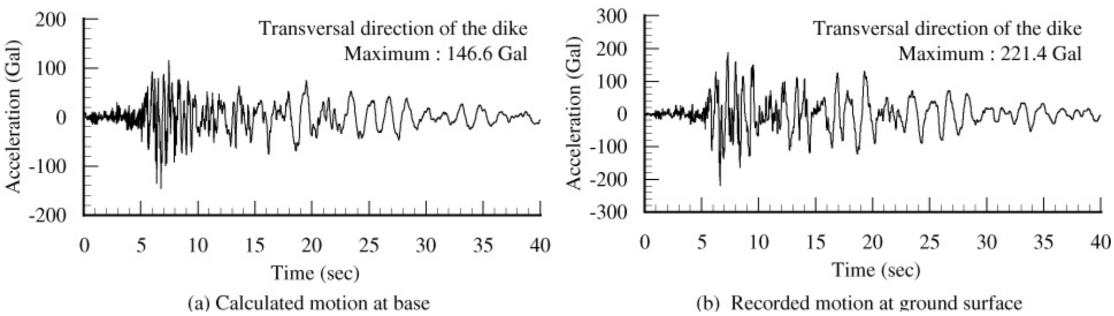


Figure 6: Input motions

NUMERICAL RESULTS

Figure 7 shows the distribution of excess pore water pressure ratio after the earthquake. Liquefaction occurred in the sandy layers of As2-1 and As2-2 at both sites. At No. 1 site, a part of As2-1 and As2-2 layers completely liquefy except for the area beneath the embankment. This behavior beneath the embankment was observed in the reduced model tests [Koga and Matsuo, 1990]. The soil beneath the embankment behaves under the stress-constrained boundary condition, because the liquefied ground cannot sustain the horizontal stress. After the ground at both sides of the embankment liquefied, the deviatoric stress in the soil beneath the embankment remained, and led the soil state to failure. As a result, the soil could not reach zero effective stress state. The non-liquefied zone also appeared under both toes of the embankment and in As2-1 layer. This behavior was due to the generation of large shear strain in As2-1 layer. In contrast with No. 1 site, As2-1 and As2-2 layers beneath the embankment at No. 2 site almost liquefied, because the sandwiched cohesive layers of Ac constrained the lateral deformation.

Figure 8 shows the deformed configuration after the earthquake and the amount of displacement at four representative nodes for each model. The scale of deformation was the same as that of the FEM mesh and the arrows in the figure showed the direction of the magnitude of displacement. The simulation reproduced a large settlement of about 2.9 m at the crest, which agreed with the measured settlement of about 2.7. The large settlement was due to the lateral spreading of the liquefied soils underlying the embankment at the damaged site. The cut-off walls at both sites moved with soil movement because the penetration depth into the cohesive layer was not sufficient. Although the major damage was not observed at No. 2 site, the simulated settlement was about 68 cm, which was much smaller than that at No. 1 site. Thus, the simulation could reproduce the qualitative difference in damage between No. 1 site and No. 2 site.

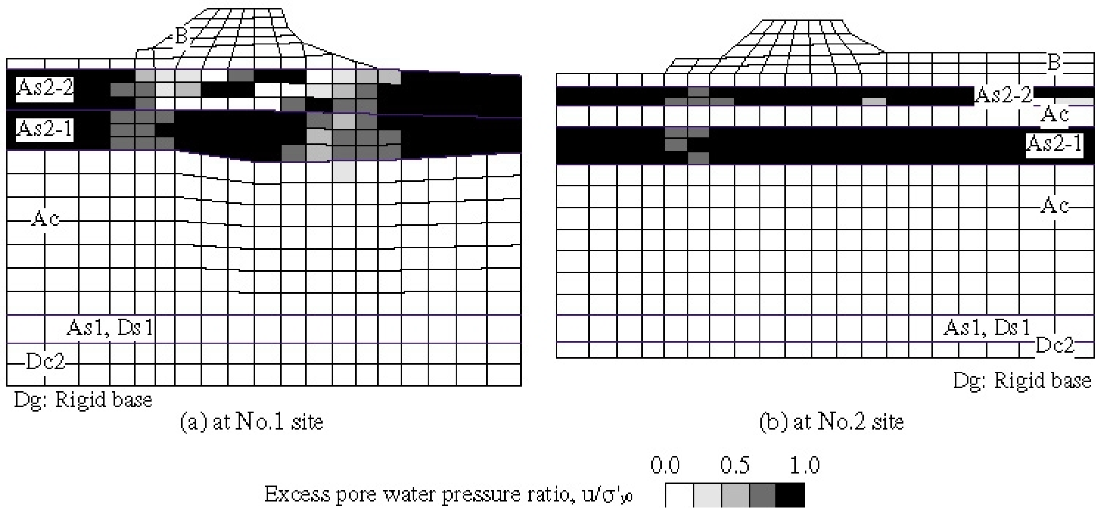


Figure 8: Distribution of excess pore water pressure ratio around the dike

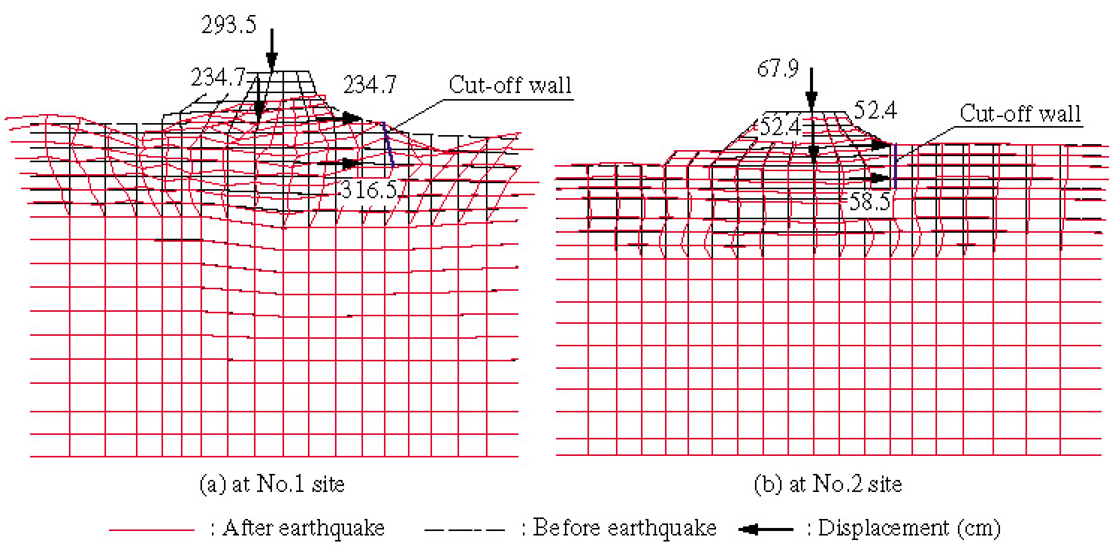


Figure 9: Deformed configuration around the dike

The simulation overestimates the observed crest settlement at No. 2 site. However, the embankment itself hardly deformed in contrast to the failed embankment at No. 1 site. Although the sandy soil layers (As2-1 and As2-2) almost liquefied, the sandwiched cohesive soil layer deformed slightly as shown in Figure 9. One of the factors affecting the less damage at No. 2 site was considered to be the existence of the sandwiched cohesive soil layer.

FACTORS CONTROLLING DAMAGE EXTENT

The factors leading to a difference in damage between two sites are discussed through parametric studies. The four additional numerical cases as shown in Table 1 were performed.

Table 1: Parametric cases

Case No.	Cut-off wall	Sandwiched clay	Surface cover layer	Notes
Case 0	-	-	-	Original case
Case 1	None	-	-	Without cut-off wall
Case 2	-	None	-	Without sandwiched Ac
Case 3	-	-	None	Without surface cover layer
Case 4	-	None	None	Case 2 + Case 3
None: not modeled, - : modeled, Case 2,3 and 4 only for No.2 site				

The simulated settlements at the center of the crest for both sites are summarized in Figure 10. As a result, the following tendencies were obtained from parametric studies.

1) The settlement in Case 1 without cut-off wall became large for both cases. The existence of different type cut-off walls was not significant for the difference in damage.

2) The Large settlement was generated in Case 2 where the sandwiched clay layer was replaced to sandy layer. The existence of cohesive layer in As2 layer would affect the less damage at No. 2 site.

3) The settlement slightly increased in Case 3 removing the surface cover layer at river side. The surface cover layer was not very crucial for the difference in damage between two sites.

4) However, in Case 4 removing the surface cover layer in addition to Case 2, the increment of settlement was larger than that in Case 3. The surface cover layer affected the crest settlement in the case without the sandwiched clay layer in As2 layer.

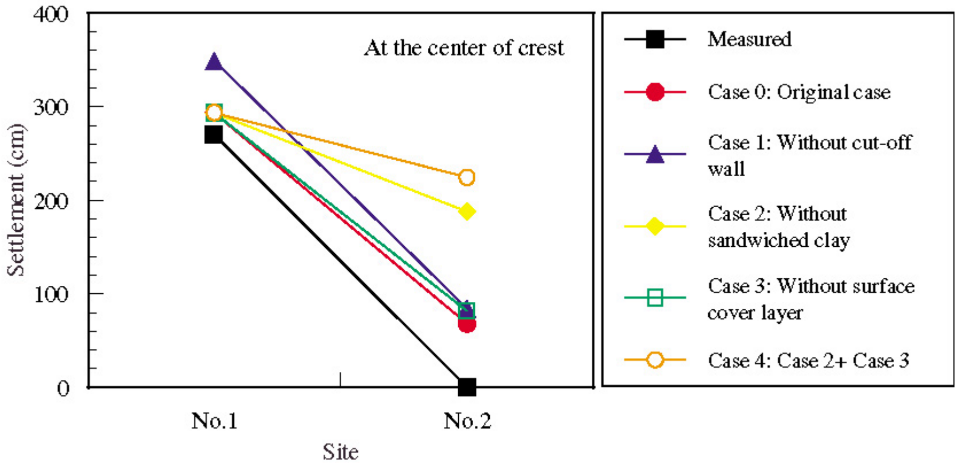


Figure 10: Crest settlement in parametric cases

CONCLUSIONS

The factors leading to a difference in damage between two sites along the Yodo-gawa River during the 1995 Hyogo-ken Nanbu earthquake were discussed through the numerical analyses. The numerical method was the dynamic response FE analysis that incorporated a cyclic elasto-plastic constitutive model for sand, a cyclic elasto-viscoplastic model for clay, and Biot's two phase mixture theory. The following results were obtained.

The numerical method qualitatively reproduced a difference in damage at both sites. The numerical results showed that the foundation sandy soils would have liquefied at both sites. The large settlement at the damaged site was due to the lateral spreading of the sandy soils underlying the embankment. Although the foundation sandy soils liquefied at the non-damaged site, the crest settlement was much smaller than that at the damaged site. The effects of the cut-off wall and the surface cover layer on the embankment settlement appeared to be small. The sandwiched cohesive soil in the sandy layers was considered to have played an important role in reducing the crest settlement. The cohesive soil layer interfered the sandy soil beneath the embankment to spread out laterally, thus minimized the settlement of overlying soil. The modeling of deformation characteristic for a cohesive layer becomes important for the accurate prediction of the embankment settlement.

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REFERENCES

Akai, K. and Tamura, T. (1978), “Study of two-dimensional consolidation accompanied by an elasto-plastic constitutive equation”, Proc. of JSCE, 269, pp98-104. (in Japanese)

Biot, M. A. (1962), “Mechanics of deformation and acoustic propagation in porous media”, Journal of Applied Physics, 33, 4, pp1482-1498.

Koga, Y. and Matsuo, O. (1990), “Shaking table tests of embankment resting on liquefiable sandy ground”, Soils and Foundations, 30, 4, pp162-174.

Koseki, J., Koga, Y. and Takahashi, A. (1994), “Liquefaction of sandy ground and settlement of embankments”, Proc. of Int. Conf. CENTRIFUGE’94, Singapore, pp215-220.

Matsuo, O. (1996), “Damage to river dike”, Soils and Foundations, SPECIAL ISSUE on Geotechnical Aspects of the January 17 1995 Hyogoken-Nambu Earthquake, pp235-240.

Matsuo, O., Nishi, K. and Uzuoka, R. (1997), “Numerical simulation of seismic failure of embankments founded on liquefied soil”, Proc. of 9th Int. Conf. on Computer Methods and Advances in Geomechanics, Wuhan, pp1807-1812.

Matsuo, O, Shimazu, T., Uzuoka, R. and Mihara, M. (1998), “Analytical study on deformation of the damaged dikes in the 1993 Hokkaido Nansei-oki earthquake”, Proc. of 10th Earthquake Engineering Symposium, 2, pp1545-1550. (in Japanese)

Oka, F. (1992), “A cyclic elasto-viscoplastic constitutive model for clay based on the non-linear hardening rule”, Proc. 4th Int. Sym. on Numerical Models in Geomechanics, Swansea, pp105-114.

Oka, F., Yashima, A., Kato, M. and Sekiguchi, K. (1992), “A constitutive model for sand based on the non-linear kinematic hardening rule and its application”, Proc. 10th World Conf. on Earthquake Engineering, Barcelona, pp2529-2534.

Oka, F., Yashima, A., Shibata, T., Kato, M., and Uzuoka, R. (1994), “FEM-FDM coupled liquefaction analysis of a porous soil using an elasto-plastic model”, Applied Scientific Research, 52, pp209-245.

Tateishi, A., Taguchi, Y., Oka, F. and Yashima, A. (1995), “A cyclic elasto-plastic model for sand and its application under various stress conditions”, Proc. 1st Int. Sym. on Earthquake Geotechnical Engineering, IS-TOKYO’95, pp399-404.

Sasaki, Y. (1998), “Seismic damage to river dikes”, Proc. Sym. on flow characteristics and permanent deformation of soil structures during earthquake, The Japanese Geotechnical Society, pp293-298. (in Japanese)

Shibata, T., Oka, F. and Ozawa, Y. (1996), “Characteristics of ground deformation due to liquefaction”, Soils and Foundations, SPECIAL ISSUE on Geotechnical Aspects of the January 17 1995 Hyogoken-Nambu Earthquake, pp65-79.

Zienkiewicz, O. C. and Bettes, P. (1982), “Soils and other saturated media under transient, dynamic conditions. General formulation and the validity of various simplifying assumptions”, Soil Mechanics-Transient and Cyclic Loads, New York, John Wiley & Sons, pp1-16.