

Research Paper

Prediction of huge earthquake-induced deformation of in-service embankments using crushed mudstone as a soil material with slaking and proposal of countermeasures

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ARTICLE INFO

Keywords:

Mudstone
Slaking
Embankment
Seismic Analysis
Countermeasure

ABSTRACT

Evaluating the seismic resistance of embankments using crushed mudstone as a geomaterial is an urgent and crucial requirement. In this study, a ground investigation was conducted on the actual embankment. Based on the results, the seismic response of the embankment, simulating progressed slaking, was conducted by elastoplastic finite deformation analysis using two major types of earthquake motion: epicentral and subduction zone earthquakes. Based on the results of the geotechnical investigation, the embankment could be divided into three layers owing to differences in physical properties, and slaking progressed below the groundwater level in the embankment. The embankment did not exhibit large deformation during the epicentral earthquake owing to the short duration. For the subduction zone earthquake, the developed shear strain was from the two large acceleration groups and the subsequent smaller accelerations, resulting in large deformation. Seismic loading caused the gradual loss of the overconsolidation and decay of the structure which reduced the embankment strength. This analysis revealed that shear strain developed at the slope toe and the lower part of the embankment. Furthermore, the analysis after the earthquake was also conducted to examine whether or not countermeasure method is feasible for emergency restoration. The seismic resistance was greatly improved when a combination of ground improvement and replacement/counterweight fill methods were used to reinforce these areas, which is not only during but also after the earthquake. This study can contribute to the understanding of the seismic behavior of soil structures using materials undergoing internal deterioration and to the development of countermeasure methods for such structures.

1. Introduction

Matsuzaka et al. (2017) showed that embankments constructed with mudstone had the highest amount of damaged soil per disaster and the largest disaster scale among the embankment materials damaged from 1993 to 2004 in Japan. After 2004, embankments using crushed mudstone as a soil material (hereinafter mudstone embankment) have also failed during the 2007 Noto Peninsula earthquake (JSCE and JGS, 2007) and the 2009 Suruga earthquake (Takagi et al., 2010; Saito and Torimoto, 2011; Nakano and Sakai, 2016). Moreover, several studies have investigated the deformation and failure of soil structures due to slaking outside of Japan. For example, settlement and slippage have occurred in clay cores and the rockfill section of the Atatürk Dam in

Turkey (Cetin et al., 2000), the 2010 collapse of the Hattian landslide dam caused by the 2005 Kashmir earthquake (Sattar et al., 2010; Kiyota et al., 2011). As expressways can act as an emergency transportation route, they must possess a high degree of seismic resistance. In particular, the embankment, which is the foundation of an expressway, must be able to resist major seismic events such as the Nankai Trough Earthquake—one of the most notable earthquakes in Japan. The plate boundary in this region has experienced occurrences of $M_w \geq 8$ earthquakes (Ando 1975). Therefore, the reduced stability of mudstone embankments owing to slaking is a serious problem.

Slaking phenomenon, one of the causes of failure of mudstone embankments, is defined as “a phenomenon in which water immersion in mainly soft rock destroys the bonding strength, causing the rock to

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<https://doi.org/10.1016/j.compgeo.2024.106855>

Received 13 July 2024; Received in revised form 26 September 2024; Accepted 20 October 2024

Available online 5 November 2024

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become muddy or disintegrate” (Dictionary of Geotechnical Engineering Terms). Several studies have been conducted on the mechanism of mudstone slaking, including Nakano (1967, 1980), Nakano et al. (1998), Yatabe et al. (2000), Kikumoto et al. (2016), Sharma et al. (2017), and Sakai and Nakano (2019). These studies demonstrated the influence of clay minerals on the strength and interpreted slaking based on elastoplastic mechanics. Laboratory tests on crushed mudstone aggregate samples confirmed that the strength decreases with the progression of slaking to repeated drying and immersion cycles.

For the analysis of mudstone embankments, replication analyses have been conducted as in Bakir and Akiş (2005), Nakano and Sakai (2016). In other words, the analysis is a post-disaster validation analysis and the Type-C prediction of Lambe (1973) has been performed, and there have been few efforts on predictive analysis, which is a Type-A prediction. There were cases where the embankment does not collapse during the earthquake but collapses after the earthquake in the 2007 Noto Peninsula earthquake or the 2009 Suruga earthquake. Post-earthquake behavior is also important, since in cases where there is a groundwater level within the mudstone embankment, positive excess pore water pressure may be generated during the earthquake and the embankment may deform due to its dissipation. Various studies on slope stability are active for soil structures made of mudstone with Type-A evaluation, such as Zhang et al. (2017) and Luo et al. (2019). Other studies on predictive analysis have also been conducted, such as statistical analysis and Newmark method for landslide prediction (Ma and Xu, 2019), as well as liquefaction determination (Sukkarak et al., 2020). However, there are few examples of predictions made about not only during but also post-earthquake.

Seismic countermeasures are also important to maintain the function of an emergency transportation route in response to a large earthquake. Seismic measures for embankments have been studied; Yin et al. (2019) showed that a seismic hazard assessment based on probabilistic seismic hazard analysis, a fragility assessment based on incremental dynamic analysis and probabilistic seismic demand analysis, and a probability risk assessment based on a hazard curve differ with and without retaining wall. Additionally, Zhou et al. (2023) evaluated seismic fragility curves, focusing on a settlement of the embankment crest. These studies do not provide a detailed discussion of seismic mechanisms, and these also do not address post-earthquake.

In this study, detailed geotechnical investigations were conducted on a high mudstone embankment that had been in service for more than 15 years. Boring, sampling, and laboratory tests were conducted to understand the physical properties of the embankment. Mechanical tests were also conducted on the sampled embankment materials to understand their mechanical properties and reproduce their mechanical behavior using the elastoplastic constitutive model, the SYS Cam-clay model (Asaoka et al., 2000, 2002). Based on the results, the analytical conditions of the mudstone embankment were determined, and the seismic response was simulated to predict the behavior of the embankment under two types of seismic motion through numerical analysis. The differences and characteristics of the deformation behavior of the embankment under the two types of earthquakes and countermeasures were discussed.

A feature of this study is conducting a Type-A prediction focusing on predictive analysis rather than a Type-C prediction involving post-disaster validation analysis. In addition, a key feature of this study is to validate the parameters used in the analysis from both static and dynamic perspectives using measured data. The predicted seismic deformation behavior of the in-service embankment can reveal its weak points and help devise anti-seismic reinforcement construction methods, which is also important from an engineering perspective.

The numerical analysis was performed on GEOASIA (Noda et al., 2008) which is a soil-water coupled finite deformation analysis equipped with the SYS Cam-clay model. GEOASIA has been able to reproduce Type-C predictions for the embankment collapse near Makinohara SA on the Tomei Expressway in Japan, as well as the Noto Peninsula

earthquake in 2007 (Nakano et al., 2010; Sakai and Nakano, 2012). Therefore, it is an effective tool for analyzing the seismic response of embankments.

The same mudstone embankment and earthquakes were the targets of a research presented by Inukai and Nakano (2022) at an international conference. However, it conducted the analysis on a saturated mudstone embankment assuming a post-heavy rainfall scenario. In this study, we analyzed the same embankment considering the actual groundwater level. Furthermore, effective seismic countermeasure methods are analytical investigated, and the mechanism of their seismic improvement is also presented. These points differ from those of previous proceedings.

2. Features of the target embankment and layer classification based on a geotechnical survey

2.1. Overview of target embankment

Fig. 1 illustrates the cross section of the target embankment in the Chubu region, Japan. The embankment is located on an inclined ground and has a height of approximately 28 m. It has been in service for approximately 20 years. We conducted a detailed geotechnical investigation for 4 months after 15 years of service. The positions of the four boreholes drilled for the survey are indicated in Fig. 1 (as Bor. 1 to Bor. 4). The red dots represent the locations where the undisturbed samples described in Section 3 were collected. The highest and lowest groundwater levels during the investigation were observed using a self-recording water level gauge. The groundwater fluctuation at the middle of the embankment was significant, indicating that the area was subjected to repeated drying and immersion. Fig. 2 shows the embankment after excavation of the toe near Bor. 3. Although the excavated area was above the observed groundwater level, the grain size was different in the upper and lower sections of the photograph, confirming the slaking around the bottom of the embankment.

2.2. Target embankment features and layer classifications

Fig. 3 illustrates the N value and grain size of Bor. 1, with depth representative of the four boreholes. The gravel content was 60 % or more from the ground surface to a depth of approximately 10 m, whereas it was almost 50 % or less below the observed highest groundwater level at a depth of 12.5 m. However, below the observed highest groundwater level, the clay content was high. The gravel was mainly crushed mudstone with a large grain size. This difference in grain size indicates that grain refinement occurred not only at depths between the highest and lowest observed groundwater levels, but also below the lowest groundwater level. It suggests that slaking is progressed due to fluctuations in the effective stress with an increase/decrease in groundwater. The N value near the ground surface was marginally lower (approximately 10). It exceeded 50 near a depth of 10 m with a large amount of gravel, but it ranged from 10 to 20 below the groundwater level although this material was the same (more on this later) to the groundwater above. The N value below the groundwater level was significantly lower than that above the groundwater level.

In addition to these differences in physical properties, considering that different embankment materials were used from the surface to a depth of approximately 6 m, the mudstone embankment was divided into three layers, as illustrated on the right-hand side of Fig. 3. Hereinafter, the embankment materials used in this analysis are referred to as Layers 1, 2, and 3. It can be interpreted that the slaking of Layer 3 progressed because of grain refinement below the groundwater level, as explained above.

The slaking ratio test (JHS110) conducted before construction demonstrated a low slaking ratio of 6.6 % to 15.2 %. However, the slaking ratio of the samples in this study was 73.9 %, which is significantly different from the two results. Assuming that each sample was the

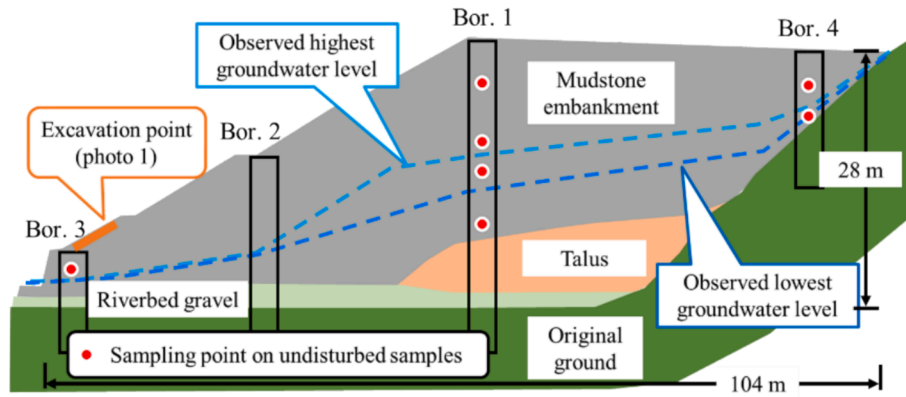


Fig. 1. Embankment cross section.

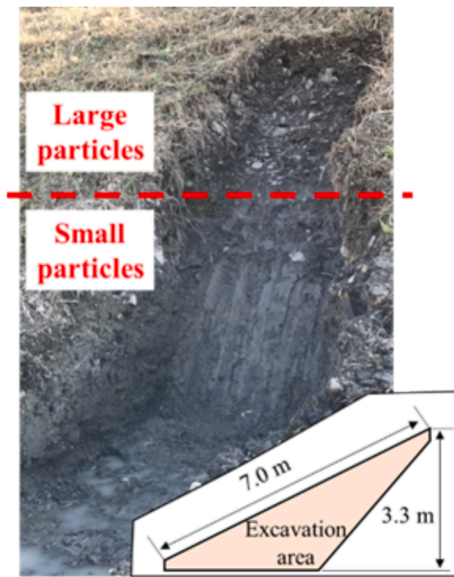


Fig. 2. View of excavation area.

same mudstone, the result implied that, even though the mudstone had a low slaking ratio during construction, the ratio changed due to the repeated drying–immersion cycles and fluctuation of the groundwater level over many years.

3. Mechanical properties of embankment materials by laboratory tests and parameter determination by elastoplastic constitutive model

3.1. Overview of the SYS Cam-clay model

The SYS Cam-clay model is an elastoplastic constitutive equation that represents a wide range of soil behaviors from naturally deposited clays to sands by introducing the concept of soil skeleton structure (structure, overconsolidation, and anisotropy) into the modified Cam-clay model (Roscoe and Burland, 1968). This model expresses anisotropy by introducing a term η^* , which was proposed by Sekiguchi and Ohta (1977), to the modified Cam-clay model. The mechanical behavior of overconsolidated and structured soils is expressed by introducing the concept of subloading surface (Hashiguchi, 1978, 1989; Asaoka et al., 1997) and a superloading surface (Asaoka et al., 2000, 2002), which are similar to the normal yield surface. These similarity centers are the origin ($q = p' = 0$).

The normal yield, subloading, and superloading surfaces are

presented in Appendix A. We express R^* as the similarity ratio of the normal-yield surface to the superloading surface; a smaller R^* indicates a higher degree of structure. $R^* = 1.0$ indicates complete structural degradation, and coincidence of the normal yield and superloading surfaces. Overconsolidation R is the reciprocal of the overconsolidation ratio (OCR), which increases to 1.0, as the OCR decreases (loss of overconsolidation). $R = 1.0$ indicates that the soil has become normally consolidated, superloading and subloading surfaces coincide. The difference in the rates of change of R^* and R for the unit plastic deformation can describe the behavior of clays and sands. More information on these evolution rules is provided in Appendix B. Decay of structure, i.e., an increase in R^* , denotes plastic compression, whereas a loss of consolidation, i.e., an increase in R , denotes plastic expansion.

The parameters of the elastoplastic and evolution rules and initial states can be determined by reproducing the mechanical behavior of the reconstituted and undisturbed specimens using the SYS Cam-clay model. This method was used in the present study.

3.2. Consolidated undrained triaxial compression tests and results reproduced by the SYS Cam-clay model

Based on the classification of the layers, separate borehole samples were extracted from each layer and then mixed and remolded after preparing them in a grain size of 425 μm or less. The reconstituted samples were pre-consolidated at 300 kPa for one week, and subsequently subjected to a consolidated undrained triaxial compression test. Subsequently, the SYS Cam-clay model was used to reproduce the experimental results for the reconstituted and undisturbed specimens.

The results of the consolidation undrained triaxial compression (hereinafter referred to as $\overline{CU}$) tests on the reconstituted samples and those reproduced using the constitutive model are depicted in Fig. 4. The reconstituted specimens typically had a lower soil structure than the undisturbed specimens. Therefore, the elastoplastic parameters of the modified Cam-clay model were determined by reproducing the mechanical behavior of the reconstituted specimen. The experimental results showed that Layer 1 has higher deviator stress than Layers 2 and 3, suggesting greater strength. Focusing on the results at 300 kPa confining pressure, both (a) and (b) show a hardening with plastic compression in the initial shear phase (axial strain less than 2 %). Moreover, the mechanical behaviors of the samples in Layers 2 and 3 were nearly identical; thus, they were determined as the same material. The simulation results coincided with the experimental results.

The numerical analysis can simulate the process of the embankment construction. (Details are provided in Section 4.2). Therefore, initial states were assumed before embankment construction as shown in the pink circle of Fig. C1. The initial states, which are specific volume v ($v = 1 + e$, e : void ratio), R , and R^* , of the embankment soil elements must be identified at the beginning of construction of the embankment. These

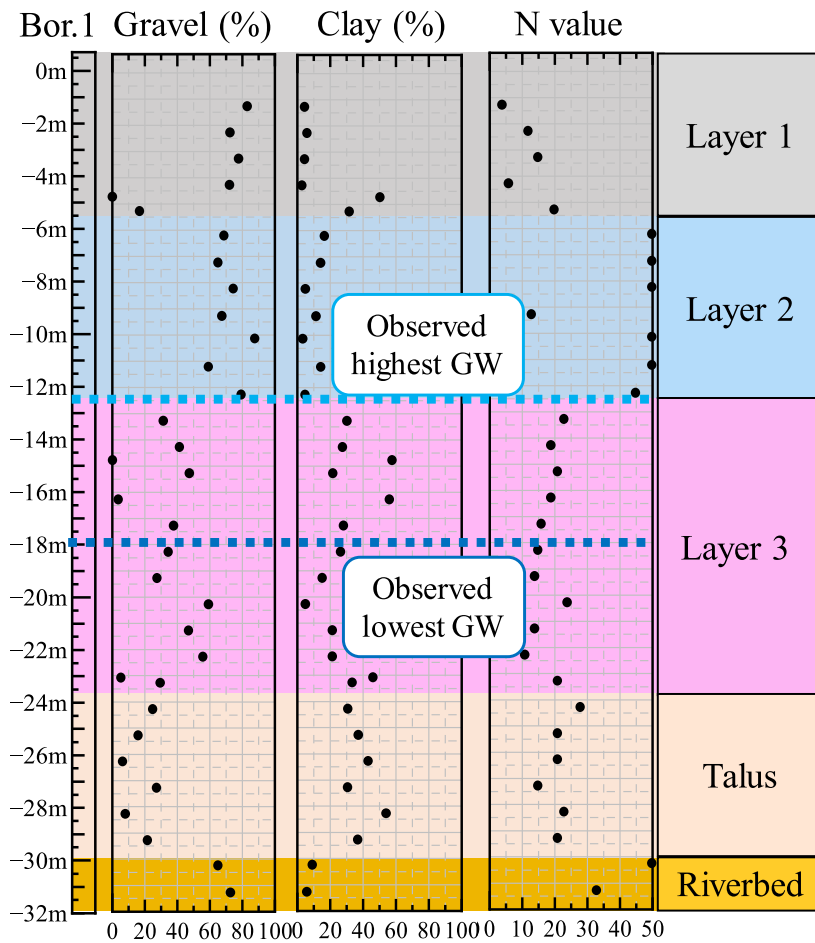


Fig. 3. Physical properties obtained by boring survey (Bor. 1).

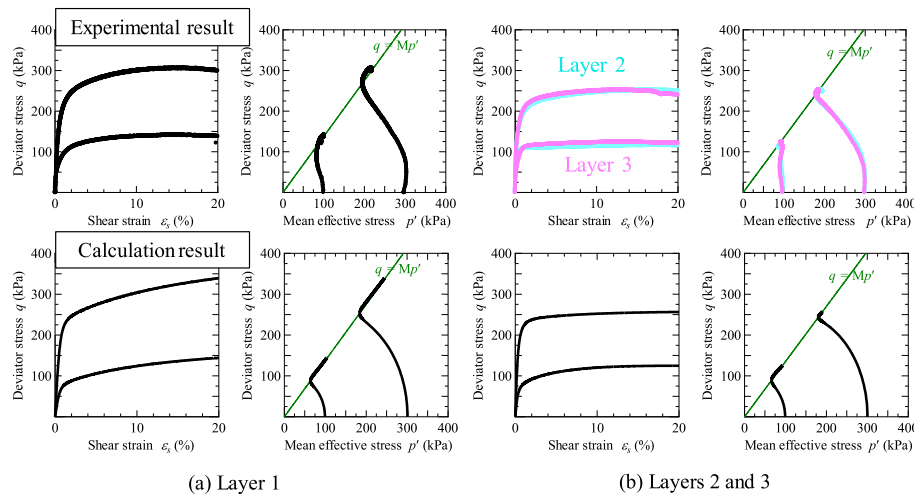


Fig. 4. Mechanical test results(upper) and calculation results(lower) of $\overline{CU}$ tests on reconstituted samples.

initial states were obtained by reproducing the $\overline{CU}$ test on an undisturbed sample. A stress history is given in the reproduction to identify the initial conditions of the embankment construction, but the details are given in Appendix C.

Fig. 5 presents the results of the $\overline{CU}$ tests on the undisturbed specimens in each layer and the shear behavior of the reproduced calculations. Compared with the mechanical behavior of the reconstituted specimens, the hardening behavior with plastic expansion (increase in q

with an increase in p' above the critical state line $q = Mp'$) was more remarkable in all layers. Comparing the results for Layers 2 and 3 at a confining pressure of 50 kPa, Layer 2 had a higher strength because the maximum deviator stresses were 600 (Layer 2) and 200 kPa (layer 3), respectively. Comparing Layers 2 and 3, it can be said that slaking is more progressed in Layer 3 because it contains more fine grains, according to the geologic column in Fig. 3, the slope excavation in Fig. 2, and the grain size distribution curve in Fig. 6. This comparison also

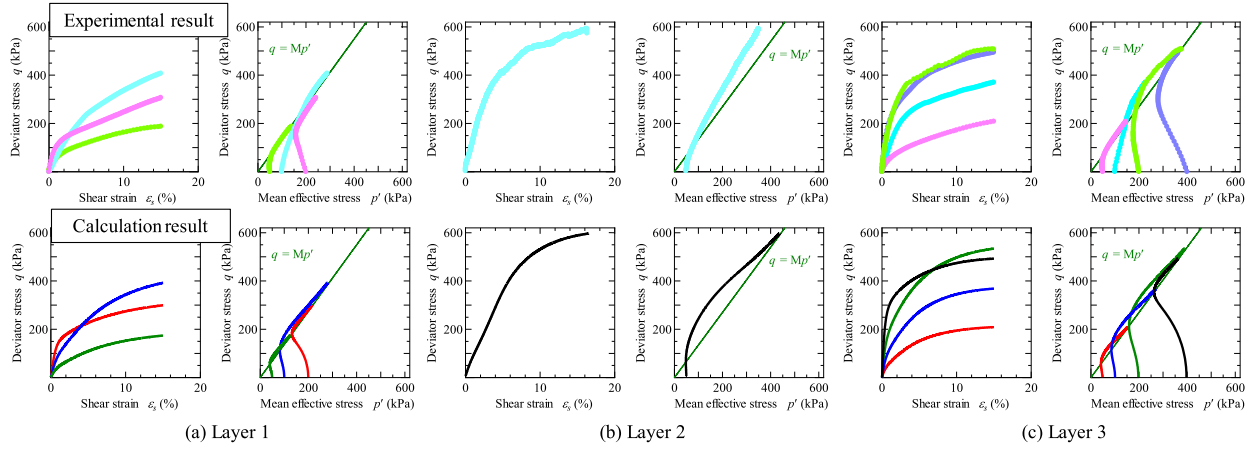


Fig. 5. Experimental (upper) and calculation (lower) results of $\overline{CU}$ tests on undisturbed specimens.

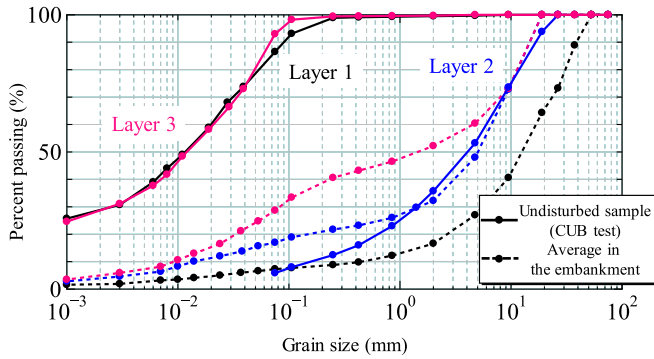


Fig. 6. Grain size distribution.

reveals that the strength decreased as the slaking progressed. Comparison of Figs. 3 and 6 shows that the grain size distribution of the undisturbed samples contained numerous fine grains, indicating that slaking was in progress. Therefore, this analysis was conducted on more progressed slaking than that of the actual soil at the time of the investigation.

Fig. 7 shows the results of cyclic triaxial tests to determine the deformation properties of geomaterials (JGS 0542–2020) on Layer 3 and the analysis results of these tests. The vertical axis in Fig. 7 represents E_{eq}/E_{max} , the normalized modulus of elasticity, where E_{eq} is the equivalent Young's modulus and E_{max} is the maximum E_{eq} obtained in the test. In many studies, the normalized modulus of elasticity is used to determine parameters for seismic response analysis (Yee et al., 2013;

Groholski et al., 2016; Puri et al., 2020; Guzel, 2024). In this study, the specific volume v in the JGS 0542 test was 1.315, while that in the $\overline{CU}$ test in Fig. 5 was 1.445. Therefore, in the analysis of the cyclic triaxial test in Fig. 7, the specific volume was adjusted to that in the JGS 0542 test. The E_{eq}/E_{max} was used instead of directly comparing stiffness because the strain rates in $\overline{CU}$ test in Fig. 5 and cyclic triaxial test in Fig. 7 were different, and therefore the initial stiffnesses were different in the two tests. The effect of strain rates on the initial stiffness was described in previous studies, such as Vardanega and Bolton (2013). Moreover, the specimen densities in the two tests were different, as mentioned above.

Fig. 5 indicates the q -axial strain ε_q and q - p' were reproduced for all the layers. Fig. 7 shows that the decrease in E_{eq}/E_{max} was smaller in the analytical results than in the experimental results up to 0.1 % axial strain. However, the process of decreasing E_{eq}/E_{max} with increasing strain was represented, and E_{eq}/E_{max} was almost the same at an axial strain of about 1 %. The evolution rule parameters and the initial values were determined by reproducing the triaxial test results on the undisturbed specimens. The results are indicated in Table 1. The material constants of the soil in Layers 2 and 3 were set to the same value because

Table 1

Initial states and material constants of embankment.

Material name		Layer 1	Layer 2	Layer 3
Parameters for elastoplasticity				
Compression index	$\bar{\lambda}$	0.085	0.080	0.080
Swelling index	$\bar{\kappa}$	0.015	0.018	0.018
Critical state index	M	1.375	1.350	1.350
Intercept of NCL	N	1.350	1.380	1.380
Poisson's ratio	ν	0.300	0.150	0.150
Parameters for evolution rule				
Normal consolidation index	m	0.200	0.300	0.300
Structure decay index ($b = c = 1$)	a	0.400	0.500	0.500
Ratio of $\ D_p^p\ $ to $-D_p^p \dots^*$	c_s	0.100	0.100	0.100
Rotational hardening index	b_r	0.010	0.010	0.010
Rotational hardening limit constant	m_b	1.000	1.000	1.000
Physical properties				
Coefficient of permeability (m/s)	k	10^{-9}	10^{-6}	10^{-9}
Soil particle density (g/cm ³)				
Initial state values				
Specific volume	v_0	1.476	1.440	1.445
OCR	$1/R_0$	55.0	220.0	158.9
Structure	$1/R^*_0$	12.5	19.2	17.0
Anisotropy	ζ_0	0.000	0.000	0.000
Stress ratio	η_0	0.100	0.150	0.160
* $\ D^p\ = \sqrt{D^p \cdot D^p}$ $D^p_v = \text{tr} D^p$				

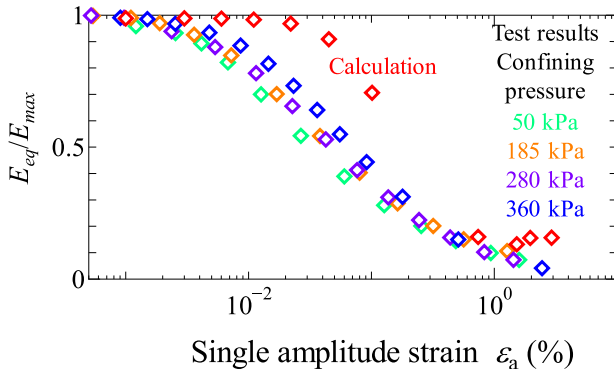


Fig. 7. Experimental and calculation results of JGS 0542 tests on undisturbed specimens of Layer 3.

the reconstituted samples for both layers exhibited similar mechanical behaviors. The slaking progression is modeled by the difference in R and R^* states in each layer. Specifically, $1/R$ and $1/R^*$ decrease due to slaking. In other words, slaking phenomenon is described as decrease in OCR and decay of structure. The permeability coefficient was determined from the results of oedometer tests and the isotropic consolidation process during shear tests on undisturbed samples.

4. Modeling of target embankment for numerical analysis and validity of input parameters

4.1. Modeling of the embankment cross section: finite element mesh and boundary conditions

This analysis was performed using GEOASIA, a soil–water coupled finite deformation analysis code. Fig. 8 illustrates the finite element mesh and boundary conditions after the embankment construction. The embankment was divided into three layers by classifying the other three borehole samples according to the physical properties described in Section 2.2. Riverbed gravel, talus, and the original ground were considered subsoils in this analysis. This analysis was carried out under plane-strain conditions. The width of the analysis area was 2,000 m, which was sufficiently large to avoid affecting the results. The hydraulic boundaries were set with undrained conditions on both edges and at the bottom, and with drained conditions (atmospheric pressure) on the surface of the ground. Layer 3 and the basement, which are below the groundwater level, were set as saturated, whereas Layers 1 and 2 were assumed as quasi-unsaturated. The elements in the quasi-unsaturated state were designed as follows: four sides of all elements were assigned the condition that the water pressure was zero, to prevent pore water pressure from occurring during an earthquake. In addition, the coefficient of permeability was set to a high value. This hinders generation of water pressure and allows for volume changes during an earthquake. The density of the unsaturated soil as a mixture decreased as the degree of saturation decreased, even if the porosity remained the same, which is expressed as follows:

$$\rho = (1 - n)\rho_s + \frac{nSr}{100}\rho_w \quad (1)$$

where ρ is the density of the soil skeleton-porewater mixture, n is the porosity, ρ_s is the soil particle density, and ρ_w is the water density. Sr is the degree of saturation, which is always constant. Hence, according to Eq (1) the density ρ varies with changes in the porosity.

The geotechnical investigations revealed that the embankment above the groundwater level was approximately 50 % saturated. Therefore, for Layers 1 and 2, which were assumed to be quasi-unsaturated, the density of the mixture in the unsaturated layer was expressed using the soil density ρ_{ss} , which was converted so that the density of the mixture obtained in Eqs. (1) and (2) were the same.

$$\rho = (1 - n)\rho_{ss} + n\rho_w \quad (2)$$

During the simulated earthquake, a viscous boundary ($V_s = 700$ m/s) was defined in the horizontal direction at the bottom of the basement (Joyner and Chen, 1975; Noda et al., 2009), to prevent the complete reflection of the seismic wave, and both lateral ends of the basement were set as simple shear deformation boundary with lateral boundary elements (Uzuoka, 2000; Yoshimi and Fukutake, 2005; Sun et al., 2010; Nakai et al., 2024). This boundary ensures that the displacements of each node at both ends and their neighboring nodes are equal.

4.2. Numerical analysis from embankment construction to soil investigation

One of the features of this analysis is to demonstrate embankment construction. The construction was represented by adding finite elements to the inclined ground (Noda et al., 2009; Takaine et al., 2010). In addition, the embankment of the post-slaking state was simulated with the parameters listed in Table 1. These parameters were obtained from the samples that have already progressed to slaking (especially, Layers 1 and 3). The construction speed was set to 0.05 m/day based on construction records. A relatively large earthquake occurred near the embankment 8 years into its service period. Therefore, the seismic wave was input, and the seismic response is evaluated qualitatively in Appendix D. It was left in place until the investigation was conducted 15 years later.

4.3. Validation of analysis model

The elements 1, 2, and 3 (hereinafter ele-1, 2, and 3) in Fig. 8 illustrate the sampling positions of the undisturbed sample used for testing in Layers 1, 2, and 3. In this section, the embankment construction process was represented by GEOASIA using the parameters in Table 1, and then obtained states (specific volume, R and R^* , etc.) of ele-1 to ele-3 were used as initial states for the next calculation. The SYS Cam-clay model calculated the process from (b) sampling to (e) shearing in Fig. C1 in Appendix C using the states obtained in ele-1, 2 and 3 and the material constants in Table 1 (exclude for the initial states), and the results were compared with the experimental results. As shown in Fig. 9, the calculated results were in good agreement with the experimental one, confirming the validity of the estimated initial states and material constants in this analysis.

Another result on the validity of the parameters was presented. The microtremor characteristics obtained from field measurements and simulation data were compared. This involved comparing the data of microtremors measured at the toe of the slope and shoulder near the analysis cross section at the actual site during the embankment's in-service period with the data from numerical simulation. Specifically, the Fourier spectrum of the measured data was calculated, and the Fourier spectrum ratio (transfer function) between the toe and shoulder was obtained. In the analysis, this ratio was calculated for the toe and shoulder data under the application of white noise with the same amplitude as that of the microtremors. The comparison between field

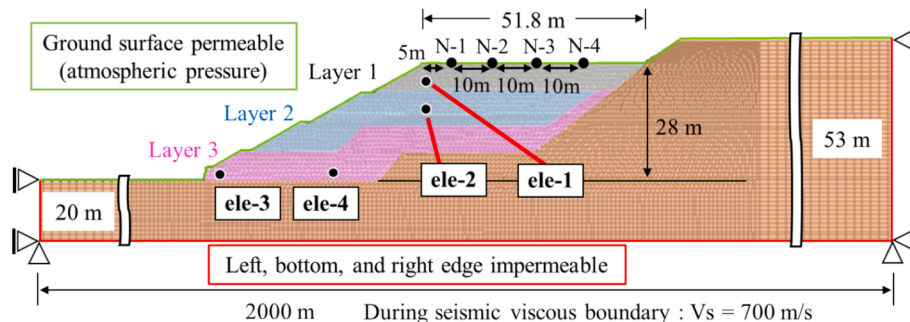


Fig. 8. Finite element analysis after embankment construction and boundary conditions.

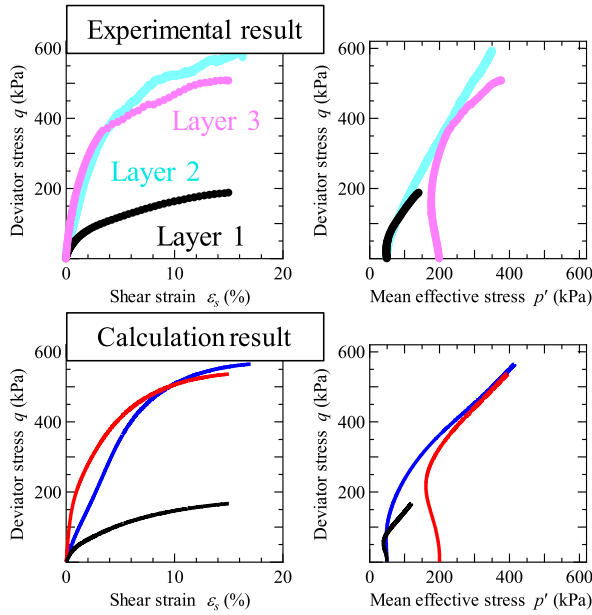


Fig. 9. Comparison of $\overline{CU}$ test results (upper) and analysis results (lower).

measuring and simulation was based on this Fourier spectrum ratio, which exhibited the wave transfer features within the embankment. Fig. 10 illustrates the relationship between the Fourier spectrum ratio and the period. The analytical model tended to amplify more near the 1 s period, but the other ranges were generally in agreement. The model overestimated the response around 1 s period. To assess the effect of the differing responses between the analytical and experimental models, the period range of approximately 0.9 to 1.8 s was removed from the Type I earthquake ground motion. The results, shown in Appendix E, indicate that the impact of these periodic bands with different responses was small.

5. Prediction of embankment deformation under two types of huge earthquake motion

In this section, to evaluate the seismic stability of mudstone embankments with slaking, the deformation behavior was simulated for two types of huge earthquake motion: Type I, a subduction zone earthquake such as the Nankai Trough earthquake, and Type II, an epicentral earthquake such as the Hyogo-ken Nanbu earthquake. The two types of seismic motions are typical ones obtained from Specifications for Highway Bridges (Japan Road Association, 2017). Huge

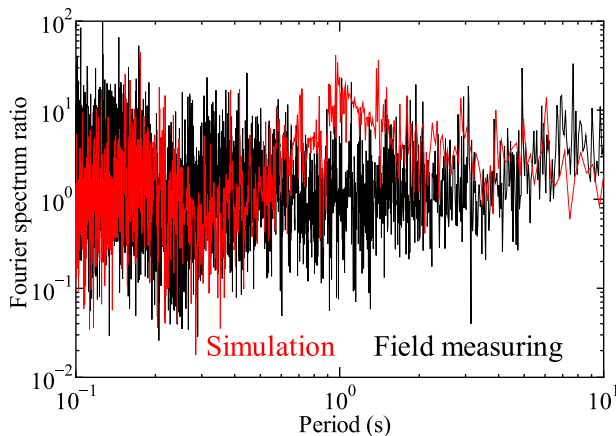


Fig. 10. Fourier spectrum ratio.

earthquake motion indicates an earthquake with high intensity but a low probability of occurrence during the service period.

A seismic motion was inputted after dissipation of excess pore water pressure that was caused by the past earthquake (Appendix D) that occurred 8 years into its service period.

5.1. Two types of input seismic waves: Epicentral and subduction zone earthquakes

Fig. 11 illustrates the two input seismic waves. These seismic motions are horizontal. According to the Specification for Highway Bridges, only the horizontal direction is considered in embankment seismic response analysis. The blue line represents Type II, and the seismic duration is 30 s. The red line represents Type I, and the seismic duration was 223 s. Type I waveform is characterized by two large acceleration groups from (a) the start to 50 s and (b) then to 100 s. This study referred to them as “1st acceleration group” and “2nd acceleration group”, respectively.

In addition to seismic duration, two representative durations employed by Kamiyama (1984) were also defined. The first is the “bracketed duration” proposed by Bolt (1973), defined as the elapsed time between the first and last acceleration excursions greater than 10 % of the maximum amplitude, denoted by T_b . The second is a duration based on power accumulation by Trifunac and Brady (1975). The duration denoted by T_p satisfies Eq. (3), where a is the acceleration, t is the time, and T is the seismic duration. Type 1 earthquakes have larger T_p and T_b , as shown in Table 2.

$$0.05 \leq \int_0^t a^2(t)dt / \int_0^T a^2(t)dt \leq 0.95 \quad (3)$$

5.2. Analysis results: Embankment behavior and settlement under Type I and Type II seismic waves

5.2.1. Type II seismic wave

The shear strain distribution at the end of the earthquake is illustrated in Fig. 12. The displacements of the toe and shoulder of the slope are also indicated to capture the degree of embankment deformation. A strain of 30 % and horizontal displacement of 0.83 m occurred around the bottom of the embankment and toe of the slope by the end of the earthquake. Shear strain developed around the slope shoulder of Layer 1, resulting in a settlement of 1.2 m. However, the shape of the embankment did not significantly change.

5.2.2. Type I seismic wave

Fig. 13 illustrates the shear strain distributions (a) 50 s and (b) 100 s

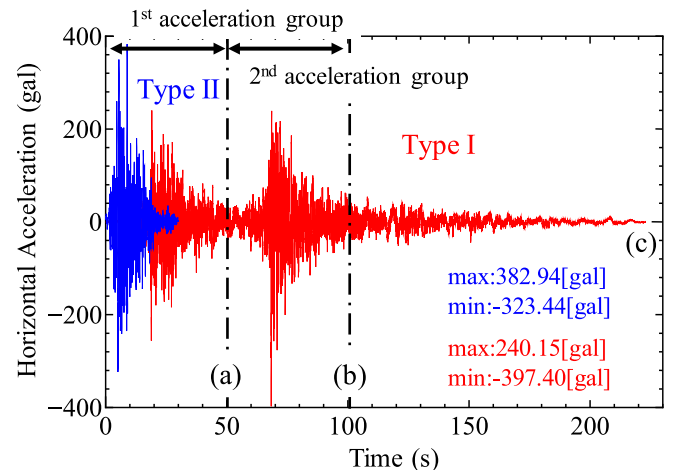


Fig. 11. Two input seismic waves.

Table 2
Durations of input earthquake.

	T (s)	Tb (s)	Tp (s)
Type II	30	17.95	12.16
Type I	223	135.23	91.13

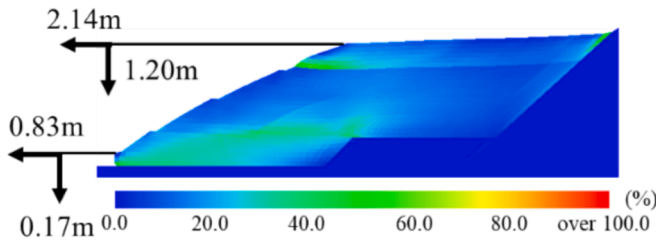


Fig. 12. Shear strain distribution at the end of Type II.

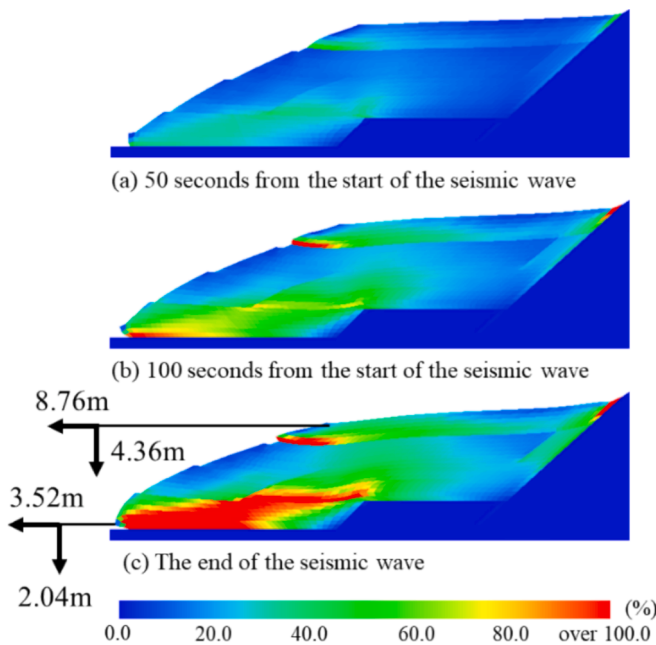


Fig. 13. Shear strain distribution at the time of Type I.

after the earthquake and (c) at its end. Fig. 13(a) and (b) show the distributions after the 1st and 2nd acceleration group inputs, respectively. The 1st acceleration group caused 30 % shear strain around the toe of the slope and bottom of the embankment. In addition, the mean effective stress distribution after the 1st acceleration group of Type I was confirmed to be the same as that at the end of Type II. This indicates that the stress distribution in the embankment after the 1st acceleration group was similar to that at the end of Type II. This is corroborated by the soil element behavior discussed below. The 2nd acceleration group had a higher acceleration than the 1st, so the shear strain at the toe progressed more remarkably. After the 2nd acceleration group (i.e., after 100 s), the acceleration of the seismic wave gradually decreased from 50 Gal. However, the strain progressed significantly (Fig. 13(c)) despite the small seismic wave amplitude.

Table 3 lists the settlement characteristics at the end of the earthquake from points N-1 to N-4 in Fig. 8, including the differential settlement between N-1 and N-2 and the road gradient, which was calculated by dividing the differential settlement by a width of 10 m between the two points. In Type II, the largest settlement in the four points was 0.98 m at N-1, which is less than 1 m, and the differential

Table 3
Settlement for Type II and Type I earthquakes.

	Type II	Type I
N-1 (m)	0.98	3.98
N-2 (m)	0.65	2.67
N-3 (m)	0.42	2.10
N-4 (m)	0.38	1.83
(N-1)-(N-2) (m)	0.33	1.31
Road gradient (%)	3.3	13.1

settlement was as small as 0.33. Therefore, emergency restoration work can be minimized, and emergency transportation routes can be secured. In contrast, for Type I, the settlement at N-1 was 3.98 m and that at N-4 was 1.83 m; hence, early restoration and emergency transportation routes efforts may be difficult to ensure.

5.3. Discussion: Soil element behavior at ele-3 under two types of seismic waves

In the 1st acceleration group of the subduction zone earthquake in this study, the embankment deformation was similar to that of the epicentral earthquake. In other words, the 2nd acceleration group and subsequent waves caused the embankment to deform significantly. The mechanism is based on the behavior of the soil elements. Fig. 14(a) and (b) depict the behaviors of soil elements ele-3, where shear strain develops significantly at the toe of the slope in the Type II and Type I, respectively. The element ele-3 is in Layer 3 and correspond to mudstone elements with progressed slaking. Fig. 14(b) is color-coded for the 1st and 2nd acceleration groups and subsequent waves, respectively. In addition, the behavior of the 1st acceleration group propagation (green line) in the q - p' relationship is shown separately.

Fig. 14(a) and the 1st acceleration group (from the earthquake onset to 50 s) in Fig. 14(b) depict nearly identical behavior. At the earthquake's initiation, the relationship between deviator stress q and mean effective stress p' that the stress state is close to the CSL. The overconsolidation R -shear strain relationship reveals R of about 0.2, indicating an overconsolidation state. Therefore, when a large shear force is generated by an earthquake and the stress state is above the CSL, hardening behavior with plastic expansion (increase in deviator stress q with an increase in mean effective stress p' above the CSL $q = Mp'$) occurs. Although R increases in this situation, neither a large decrease in p' nor does a significant increase in water pressure occur. Therefore, in the Type II and Type I 1st acceleration group, although plastic deformation developed, the repeated loading owing to cyclic loading by the earthquake, no significant shear strain development occurred, and the soil exhibited a hardening behavior with plastic expansion during the earthquake because the soil is enough overconsolidated at the start of the earthquake.

In the 2nd acceleration group (red line) shown in Fig. 14(b), the hardening behavior with plastic expansion is observed while overconsolidation R is increased above the CSL same as 1st acceleration group. Since the 2nd acceleration group has a higher acceleration than the 1st acceleration group, R was rapidly increased while the shear strain developed significantly.

One hundred seconds after the earthquake (purple line) in Fig. 14(b), R experiences minor fluctuations, remaining relatively constant owing to repetitive loading from small acceleration waves. In contrast, R^* gradually increases, signifying a gradual decay of structure. This behavior occurred the decrease in q with decrease in p' , indicates softening behavior with plastic compression. By a decrease in the stiffness of soil elements attributed to the reduction in p' , the shear strain developed despite a small seismic wave amplitude.

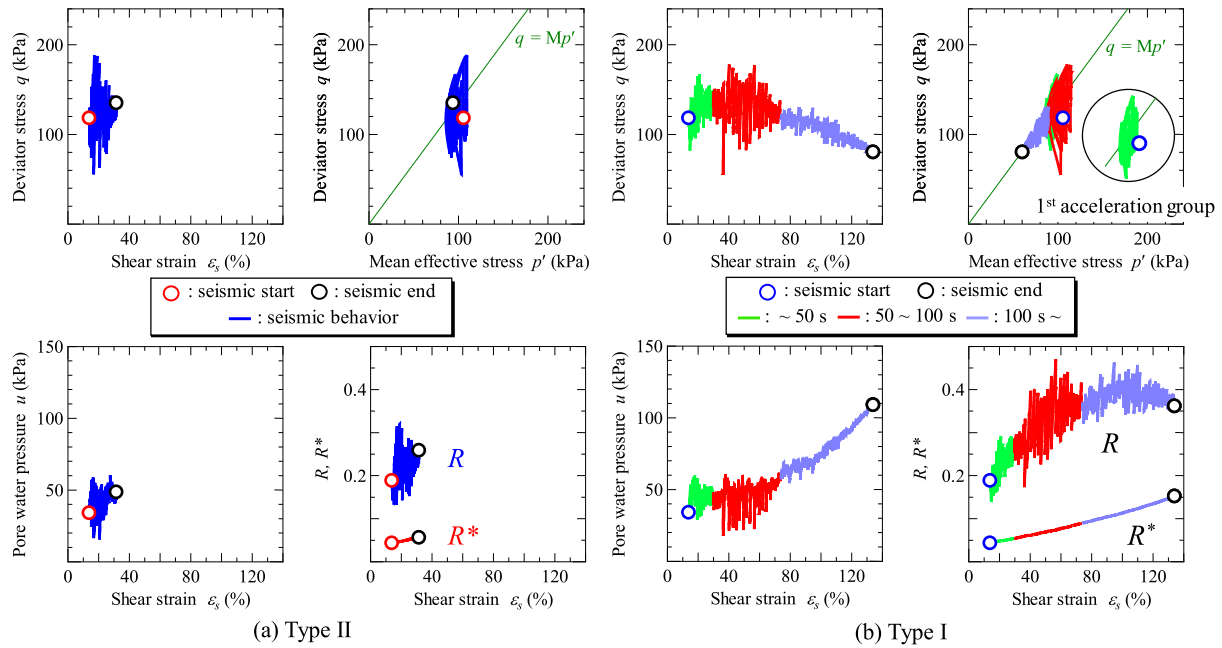


Fig. 14. Behavior of ele-3.

6. Countermeasures

The embankment under Type I earthquake deformed too severely to allow for early recovery and ensure emergency transportation, as described in Section 5. Therefore, to improve seismic resistance against Type I earthquakes, three countermeasures were examined (Fig. 15). When discussing the seismic response of embankments, the focus is often on the failure line from the slope shoulder to the toe of the slope (Bakir and Akiş, 2005; Aydan and Ulusay, 2002). However, in this study, the results of shear strain distribution during the earthquake in the mudstone embankment with progressive slaking indicate that the localization of shear strain appears at the bottom of the embankment where slaking has progressed below the groundwater level and that the

development of shear strain from the slope toe caused large deformation of the embankment. Therefore, the countermeasures were designed to reduce the shear strain at the toe of the slope and bottom of the embankment and did not extend the toe position.

Case 1 involved soil improvement through cement mixing. The improved area was a two-phase elastic body, with the material constants listed in Table 4. These constants were determined with reference to Hsiao and Hsieh (2022). The countermeasure suggested for Case 2 was replacement/counterweight fill construction (Sakai et al., 2023), in which the embankment was excavated with a 1:0.8 gradient from the uppermost berm, and then a counterweight fill was constructed at the top of the embankment. The method involved excavating a portion of the embankment surface and replacing it with high-quality soil to act as a counterweight. This method densifies the embankment by increasing the mean effective stress below the counterweight fill. It was confirmed to be effective as a seismic countermeasure for the existing embankment. The physical properties and material constants of the counterweight fill were obtained from Sakai et al. (2023). It is an easily compressible purchased soil with $\rho_{dmax} = 2.092 \text{ g/cm}^3$ and $w_{opt} = 8.9\%$ and is compacted with a degree of compaction $D_c = 95\%$. Case 3 is a combination of Case 1 and Case 2. The toe of the slope and bottom of the embankment were enhanced with cement, and a counterweight fill was constructed on the slope above the soil improvement area. All reinforcement methods are facile for implementation.

Type I was input after each countermeasure method had been implemented and excess pore water pressure had dissipated. In addition, the post-earthquake behavior was investigated to study post-disaster rehabilitation. The past seismic motion was not input, as the seismic motion in the untreated case had little influence on the subsequent seismic behavior owing to Type I or II.

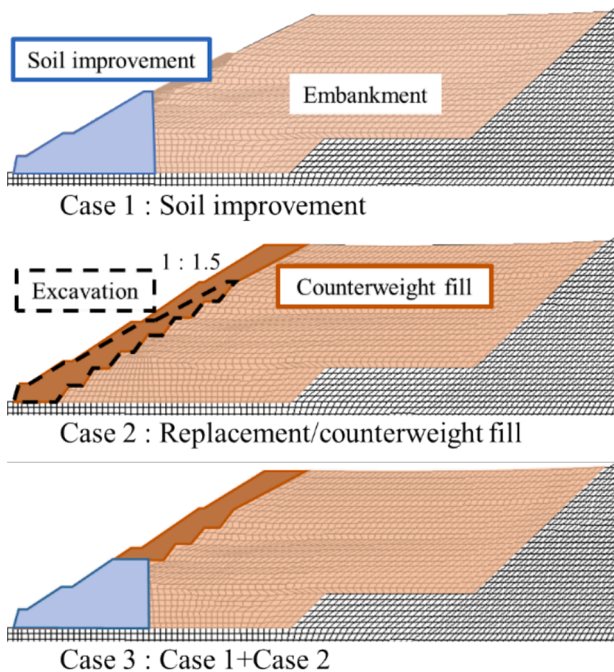


Fig. 15. Embankment deformation countermeasures.

Table 4

Material constants used for soil improvement.

Young's modulus (kN/m ²)	2.20×10^4
Poisson's ratio	0.17
Density (g/cm ³)	2.30
Coefficient of permeability (m/s)	10^{-9}

6.1. Analysis results: Embankment behavior and settlement under Type I earthquake when countermeasure is applied

Fig. 16 displays the shear strain distribution at the end of the earthquake for the three cases of countermeasures: in Case 1, the improved toe of the slope exhibited almost no strain, and the shear strain development at the bottom of the embankment was reduced to approximately half of that of the untreated slope shoulder. However, shear strain developed significantly at the shoulder of the slope in Layer 1 due to a lack of countermeasures. In Case 2, on the other hand, the shear strain at the shoulder of the slope did not progress because of the counterweight fill. The shear strain was reduced by half compared with that of the untreated toe. However, the shear strain around the bottom embankment still developed to 50 %, and the strain also developed in the counterweight fill. A comparison between Cases 1 and 2 showed that the soil improvement around the toe of the slope and bottom of the slope was effective against shear deformation. Case 3, in which the improvement was carried out at the bottom of the embankment, exhibited no significant development in shear strain at either the lower part of the embankment or the shoulder of the slope.

Table 5 lists the settlement at the end of Type I for points N-1 to N-4. N-1 had the largest settlements of 2.26, 2.01, and 1.24 m in Cases 1, 2, and 3, respectively, compared with 3.72 m in the untreated. The differential settlement was approximately 0.6 m in Cases 1 and 2 and a reduction of 50 % compared to the case with no countermeasures, and the differential settlement was 0.24 m in Case 3. In Case 3, which showed the greatest reduction in shear strain development and settlement, damage to the roads on the embankment was limited, and the embankment recovered swiftly. Based on the shear strain and settlement, the countermeasure in Case 3 is the most effective.

6.2. Discussion: Change in ele-4 element behavior owing to application of countermeasure

Next, the mechanism of seismic resistance improvement with the countermeasures is discussed by examining the behavior of the soil elements in the embankment for Case 3, which is the case with the highest improvement. Fig. 17 illustrates the seismic and post-seismic behaviors of the element (ele-4) at the bottom of the embankment in Untreated and

Table 5
Settlement.

	Untreated	Case 1	Case 2	Case 3
N-1 (m)	3.72	2.26	2.01	1.24
N-2 (m)	2.46	1.66	1.45	1.00
N-3 (m)	2.01	1.45	1.30	0.99
N-4 (m)	1.81	1.36	1.25	1.04
(N-1)-(N-2) (m)	1.26	0.60	0.55	0.24
Road gradient (%)	12.6	6.0	5.5	2.4

Case 3.

During an earthquake, the deviator stress q repeatedly increases and decreases owing to cyclic loading. In the q - p' relationship, a decrease in q is more remarkable than an increase in q for the green line in Fig. 17(b) than in Fig. 17(a). This behavior indicates that the countermeasure rendered unloading more significant than loading. This remarkable unloading is supported by the R - ϵ_s relationship in Fig. 17(b), which shows that the overconsolidation R decreases with the earthquake. In other words, the overconsolidation ratio ($1/R$) increases due to unloading. In contrast, an increase in R , i.e., a decrease in OCR ($1/R$), was observed in the elemental behavior of the Untreated, as depicted in Fig. 17(a), which is a major difference between the two. The soil elements of the countermeasures exhibited predominantly elastic behavior as the OCR increased during the earthquake. Therefore, plastic strain did not develop easily even if the soil elements were repeatedly loaded with a small p' . In the 2nd acceleration group (50–100 s) and after 100 s, the Untreated exhibits a gradual increase in R , while Case 3 shows a gradual decrease. This indicates that unloading is a factor in preventing strain development, even during an extended duration. Regarding the post-earthquake behavior, the mean effective stress recovers with the dissipation of water pressure, the specific volume decreases, and the embankment becomes denser and more stable.

The effective stress paths in the 1st acceleration group are significantly different, as shown in Fig. 17. Because the soil behaved elastically when the OCR increased, the stress ratio η ($=q/p'$)-shear strain relationship for the 1st acceleration group was investigated, as illustrated in Fig. 18. The stress ratio and shear stress of the untreated case were higher than those of Case 3 due to seismic loading. In addition, plastic strain developed after a single loading, and shear strain developed significantly as the loading was repeated. However, in Case 3, the stress ratio η was always smaller than that of the case with no countermeasures. In other words, the shear stress was weaker. Furthermore, the OCR ($1/R$) increased when the shear strain had developed by 10 %, as depicted in Fig. 17. Therefore, the plastic strain development with single loading was small.

As indicated by the behavior of soil elements in Fig. 17, positive excess water pressure was generated and accumulated in Layer 3 at the end of the earthquake. Therefore, settlement associated with excess pore water pressure dissipation is a concern after an earthquake. Fig. 19 illustrates the settlement-time relationship from the start of the earthquake to the dissipation of excess pore water pressure at N-1. After an earthquake, the soil consolidation occurs as the mean effective stress recovers with the dissipation of excess water pressure. The settlement that occurs after an earthquake is caused by this behavior. The amount of water pressure generated during the earthquake as shown in Fig. 20 is Case 1 = Case 3 > Case 2 > Untreated, and as shown in Table 6, the settlement after the earthquake is also in the same order. Cases 1 and 3 are more stabilized than before the earthquake because of the smaller overconsolidation R in addition to the compression after the earthquake. Moreover, although water pressure dissipation caused settlement, it did not occur rapidly after the earthquake for any of the countermeasures. Hence, an emergency transportation route can be secured through this restoration, after which the primary restoration work can be performed.

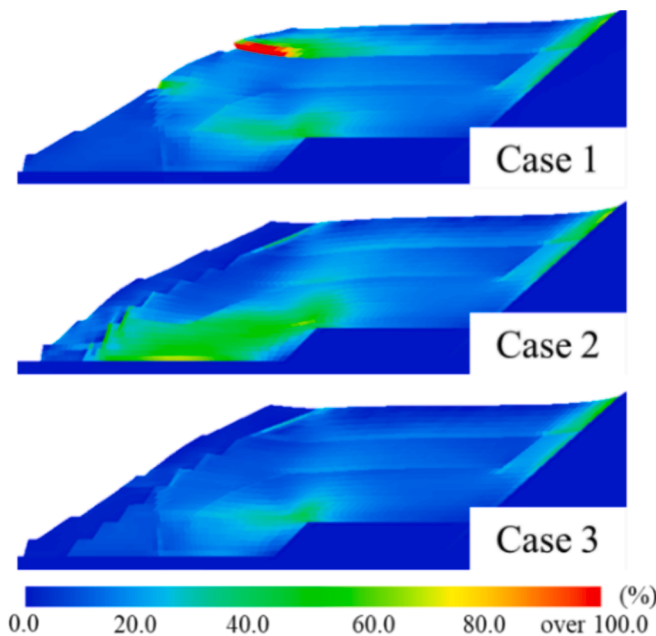


Fig. 16. Shear strain distribution of the three countermeasure methods at the end of Type I earthquake.

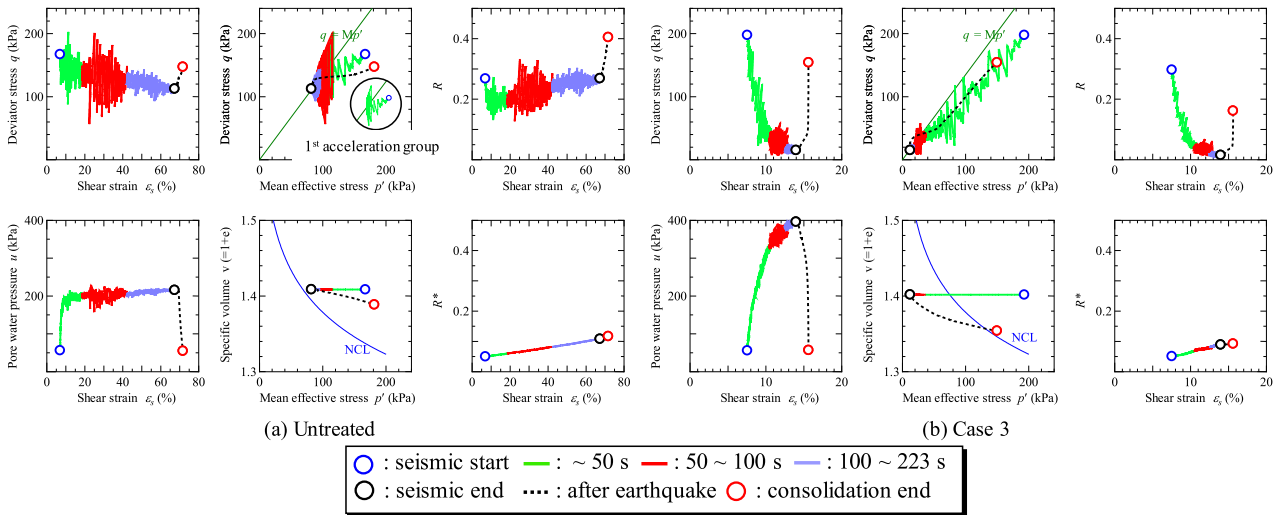


Fig. 17. Behavior of ele-4 for Untreated and Case 3.

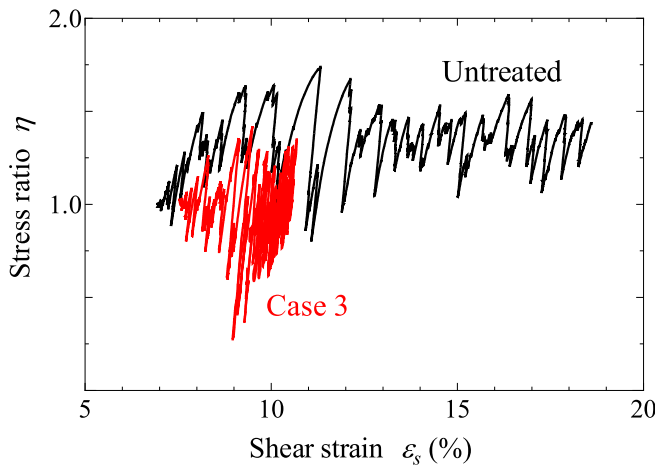


Fig. 18. Stress ratio vs. shear strain relationship in 1st acceleration group.

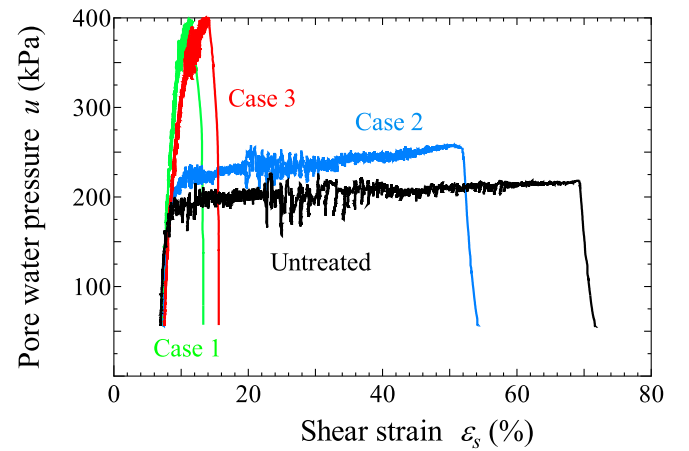


Fig. 20. Pore water pressure-shear strain relationship in ele-4.

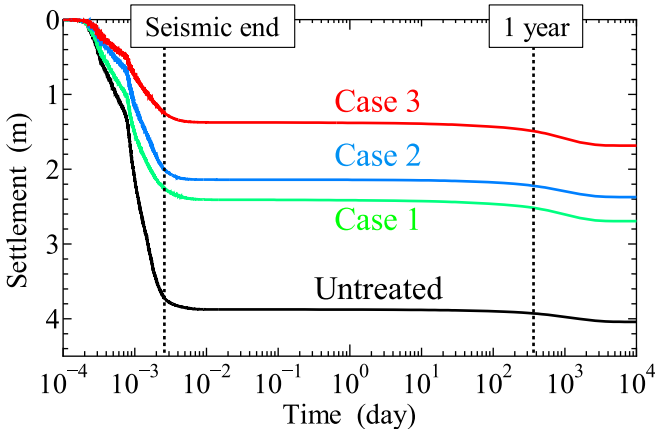


Fig. 19. Settlement-time relationship at N-1.

7. Conclusions

In this study, detailed geotechnical investigations were conducted on an actual embankment that had been in service for approximately 20 years, and mechanical tests were conducted on the sampled material.

Table 6

Settlement after Type I earthquake at N-1 (m).

Untreated	Case 1	Case 2	Case 3
0.32	0.44	0.36	0.44

Based on the results, the seismic responses to epicentral and subduction zone earthquakes were evaluated for a high mudstone embankment on inclined ground. The numerical analysis was performed on GEOASIA, which is a soil–water coupled finite deformation analysis equipped with the SYS Cam-clay model. The effectiveness of three countermeasures against subduction zone earthquakes with large deformations was evaluated. The conclusions are as follows:

- (1) The embankment was divided into three layers based on the differences in the N value and grain size distribution in the survey. The survey revealed that slaking progresses not only at the location of the groundwater level fluctuation but also below the lowest groundwater level. The slaking ratio was 6.6 % – 15.2 % at the time of construction and 73.9 % at the time of investigation. It is attributed to the fact that the fresh mudstone rock, at the time of construction, had been progressed slaking owing to repeated changes in stress state caused by groundwater level fluctuations, which weakened the bonding strength. In other words, the

slaking ratio varies greatly depending on the history before the test. Therefore, the slaking ratio of mudstone at the time of construction is remarkably different through the years.

- (2) The analysis parameters were validated in the following points. The experimental results with the undisturbed sample could be reproduced through the undrained triaxial compression test was after considering the embankment construction and sampling processes using the material constants and state of the elements at the same points where the tests were conducted. Moreover, the Fourier spectrum ratios obtained from the measured data where the toe of the slope and shoulder near the analysis cross section at the actual site when the embankment was in service and the data with white noise input to the analytical model, generally agreed except near 1 s period. And the impact of these periodic bands with different responses is small.
- (3) Approximately 30 % of the shear strain developed at the end of the Type II earthquake in Layer 3, but no major deformation occurred. Because the soil is enough overconsolidated at the start of the earthquake, the soil exhibited a hardening behavior with plastic expansion above the CSL $q = Mp'$ during the earthquake, and no significant shear strain development occurred.
- (4) In the Type I earthquake, the shear strain developed by approximately 100 % in Layer 3, resulting in a large deformation. This behavior was observed after the 1st acceleration group exhibited identical to that against Type II earthquake motion. The continuous occurrence of the 2nd acceleration group led to a significant loss of overconsolidation, and the subsequent earthquake motion of about 50 Gal was continued. The seismic resistance of the embankment decreased 100 s after the earthquake, and the subsequent small accelerations caused gradual structural degradation, resulting in the development of shear strain.
- (5) Three countermeasures were examined. In Case 1, where soil was improved to prevent the development of shear strain at the toe of the slope, significant shear strain developed at the shoulder of the slope. In Case 2, where replacement/counterweight fill was

applied, the shear strain increased to 50 % at the bottom of the embankment, although the deformation could be suppressed. In Case 3, where the two methods were combined, no significant shear strain development was observed, and differential settlement was reduced to 25 %, thus significantly improving seismic resistance.

This analysis indicates that the major shear strain in a high mudstone embankment on inclined ground develops around the toe of the slope and bottom of the embankment. Therefore, implementation of anti-seismic reinforcement in these areas is more effective. This study provides guidelines for reinforcing mudstone embankments. In addition, we found that, when rehabilitation is performed after a large disaster, the upper part of the embankment can be easily repaired, whereas repairing the lower part is difficult. Hence, countermeasures are required to reinforce the lower part of embankments for early restoration.

Future work is required to reproduce the entire slaking process, from the construction of fresh mudstone embankment to the post-slaking condition, to conduct more accurate analyses of mudstone embankments.

CRediT authorship contribution statement

Shogo Inukai: Writing – original draft, Visualization, Validation, Investigation. **Takayuki Sakai:** Writing – review & editing, Validation, Investigation, Conceptualization. **Masashi Nagata:** Writing – review & editing, Resources, Investigation. **Toshihiro Noda:** Writing – review & editing, Supervision. **Masaki Nakano:** Writing – review & editing, Supervision, Project administration.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Appendix A. Yield functions associated with the SYS Cam-clay model

Fig. A1 illustrates the three yield surfaces associated with the SYS Cam-clay model. The superloading surface is similar to the origin ($q = p' = 0$) as the similarity center outside the normal yield surface. The subloading surface is also an analog that lies inside the superloading surface. The normal yield surface is the yield surface of the modified Cam-clay model, where R^* is the ratio of the superloading surface to the normal yield surface for stress, and R is the ratio of the superloading surface to the subloading surface, and the inverse of R is the overconsolidation ratio. The current stress is always on the subloading surface and is represented by p' and q . The stresses on the superloading and normal yield surfaces are represented by $(\bar{p}', \bar{q})$ and $(\bar{p}, \bar{q})$, respectively. The superloading yield surface corresponds to the yield surface. Therefore, soil with the current stress state is inside the superloading yield surface represents overconsolidated soil, and the superloading yield surface has a memory effect of the loading.

$$R^* = \frac{\bar{q}}{\bar{q}'} = \frac{\bar{p}'}{\bar{p}}, R = \frac{q}{\bar{q}} = \frac{p'}{\bar{p}} \quad (\text{A.1})$$

The subloading surface where the current stress exists is described by Eq. (A.2).

$$\begin{aligned} f(p', \eta^*) + M \ln R^* - M \ln R + \int_0^t J \text{tr} \mathbf{D}^p d\tau \\ = M \ln \frac{p'}{\bar{p}_0} + M \ln \frac{M^2 + \eta^{*2}}{M^2} + M \ln \frac{R^*}{R} + \int_0^t J \text{tr} \mathbf{D}^p d\tau = 0 \end{aligned} \quad (\text{A.2})$$

where M is the critical state constant, $D = (\tilde{\lambda} - \tilde{\kappa})/(Mv_0)$ is the dilatancy coefficient, $\tilde{\lambda}$ is the compression index, and $\tilde{\kappa}$ is the swelling index. $\mathbf{D}^p$ denotes the plastic component of stretching tensor $\mathbf{D}$. The Jacobian J is the determinant of the deformation gradient tensor $\mathbf{F}$ of soil skeleton, expressed using the specific volume v and v_0 at the current ($t = t$) and initial time ($t = t_0$) as follows:

$$J = \det \mathbf{F} = \frac{v}{v_0} \quad (\text{A.3})$$

The superloading surface and normal yield (Cam-clay yield surface) surface, which are expressed by Eqs. (A.4) and (A.5), respectively, can be derived by substituting Eq. (A.1) into Eq. (A.2).

$$f(\bar{p}', \bar{\eta}^*) + M \ln R^* + \int_0^t J \text{tr} \mathbf{D}^p d\tau = M \ln \frac{\bar{p}'}{\bar{p}_0'} + M \ln \frac{M^2 + \bar{\eta}^{*2}}{M^2} + M \ln R^* + \int_0^t J \text{tr} \mathbf{D}^p d\tau = 0 \quad (\text{A.4})$$

$$f(\tilde{p}', \tilde{\eta}^*) + \int_0^t J \text{tr} \mathbf{D}^p d\tau = M \ln \frac{\tilde{p}'}{\tilde{p}_0'} + M \ln \frac{M^2 + \tilde{\eta}^{*2}}{M^2} + \int_0^t J \text{tr} \mathbf{D}^p d\tau = 0 \quad (\text{A.5})$$

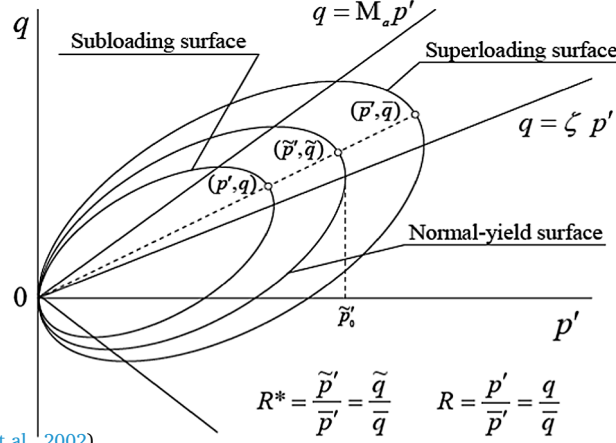


Fig. A1. Three yield surfaces (Asaoka et al., 2002)

The SYS Cam-clay model differs from the classical model in that plastic deformation occurs under loading, even when the stress state is in the superloading yield surface (even overconsolidated soil). However, when the overconsolidation R is very small (i.e., when the OCR $1/R$ is large), the soil behavior is relatively elastic. As the overconsolidation R approaches 1, that is, as the two yield surfaces approach each other, the plastic deformation increases. Thus, the SYS Cam-Clay model can continuously represent both overconsolidated, and normally consolidated soils.

Appendix B. Evolution rule and evolution parameters of structure and overconsolidation

The overconsolidation R moves toward the normal consolidation state as the plastic deformation progresses (loss of overconsolidation). The evolution rule of R is given below:

$$\dot{R} = JU \|\mathbf{D}^p\| \quad (\text{B.1})$$

The norm for plastic stretching $\|\mathbf{D}^p\|$ was applied as the degree of plastic deformation progression. The non-negative scalar function U is a function of R and satisfies $U(0) = \infty$ and $U(1) = 0$. In this study, the function U was defined by Hashiguchi (1989) as follows:

$$U = -\frac{m}{D} \ln R \quad (\text{B.2})$$

where m is a positive material constant (normal consolidation index); the larger it is, the faster the overconsolidation is lost.

The evolution rule of the structure R^* is given below.

$$\dot{R}^* = JU^* \left\{ (1 - c_s) (-D_v^p) + c_s \sqrt{\frac{2}{3}} \|\mathbf{D}_s^p\| \right\} \quad (\text{B.3})$$

D_v^p and $\|\mathbf{D}_s^p\|$ are the isotropic and deviation components of the plastic stretching $\mathbf{D}^p$; c_s determines which of the two is dominant. The non-negative scalar function U^* is a function of R^* and satisfies $U^*(0) = 0$ and $U^*(1) = 0$. Here, U^* is defined as follows:

$$U^* = \frac{a}{D} R^{*b} (1 - R^*)^c \quad (\text{B.4})$$

where a , b , and c are positive material constants (structure decay index) and $b = c = 1.0$. The larger the value of a , the faster the structure decays.

Appendix C. Stress history in reproduction calculation

Fig. C1 illustrates a schematic of the relationship between the deviator stress q and the mean effective stress p' for the stress history from construction to the $\overline{CU}$ test. First, the construction process was simulated using a one-dimensional consolidation, as shown in Fig. C1(a). Second, sampling and pulling out from the borehole were simulated using undrained and isotropic unloading, as depicted in Fig. C1(b) and (c), respectively. Finally, the $\overline{CU}$ test was simulated using an isotropic consolidation process and shearing, as shown in Fig. C1(d) and (e). The results of the $\overline{CU}$ tests were reproduced by simulations, considering the stress history and changing the initial states as input values. The initial embankment states, as well as material constants, were obtained from these reproduction processes (Noda et al., 2005).

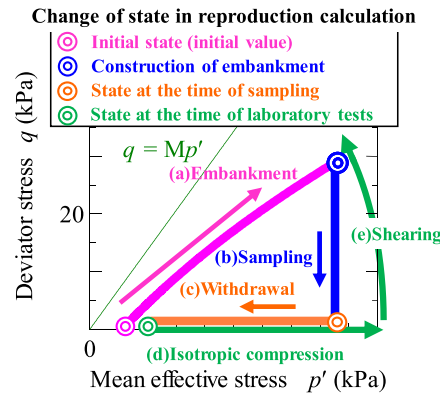


Fig. C1. Schematic diagram of stress history assumed in reproduction calculation

Appendix D. Qualitative evaluation of embankment response to past earthquake motion

A relatively large earthquake occurred near the embankment 8 years into its service period. Therefore, the seismic wave whose data is reference to NIED (National Research Institute for Earth Science and Disaster Resilience) input to the embankment was used for qualitative evaluation. Fig. D1 illustrates the seismic motion observed near the embankment. The shear strain distribution around the embankment after the earthquake is illustrated in Fig. D2. There was no deformation of the embankment or significant development of shear strain. However, a slight lateral displacement was observed at the toe of the slope. This behavior was qualitatively similar to the actual behavior, although displacement was not measured.

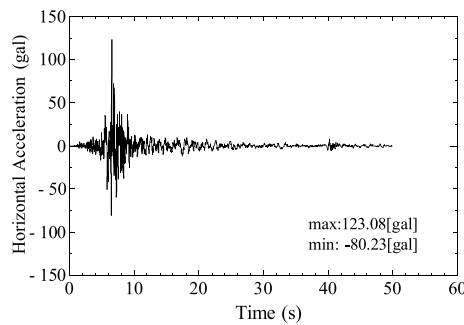


Fig. D1. Past seismic motion (the earthquake occurred 8 years after the site)

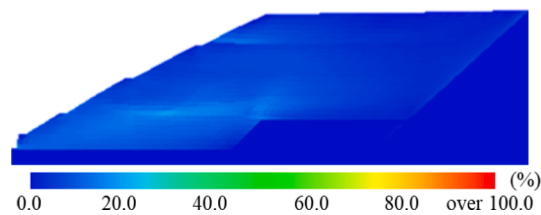


Fig. D2. Shear strain distribution after past seismic wave input

Appendix E. Stress history in reproduction calculation

The comparison of the Fourier spectrum ratios shown in Fig. 10 in Section 4.3 suggests that the analytical model may exhibit amplification around the 1 s period compared to the actual embankment. Therefore, the 0.9 to 1.8 s period was removed from the input earthquake motion, created a new earthquake motion, and input it into the analytical model to investigate the effect of different periodic bands of response. The newly created seismic motions are shown in Fig. E1.

Fig. E2 shows the shear strain distribution for the earthquake motion illustrated in Fig. E1. Compared to Fig. 13, the shear strain distribution of embankment is almost the same. Additionally, the horizontal and vertical displacements at the toe and shoulder of the slope were almost same, with only a slight decrease in horizontal displacement at the shoulder. This indicates that the effect of the periodic bands on the amplified area was small.

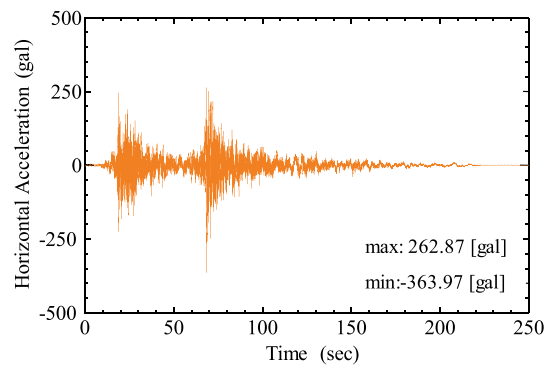


Fig. E1. Seismic motion with the 0.9 to 1.8 s periodic band removed from Type I

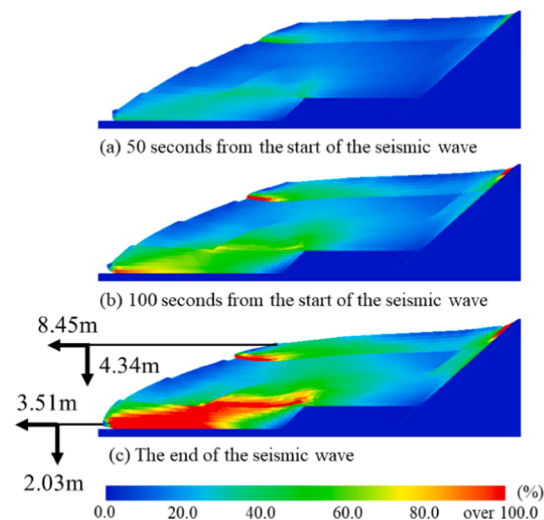


Fig. E2. Shear strain distribution for the input seismic motion shown in Fig. E1

Data availability

The authors do not have permission to share data.

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