EVALUATION OF EARTHQUAKE-INDUCED DEFORMATION OF EMBANKMENT DAMS USING CUMULATIVE DAMAGE THEORY

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ABSTRACT

The seismic performance evaluation of embankment dams are applied to confirm that the earthquake-induced settlement of embankment dams crest is enough small, because the overflow from the crest could cause catastrophic failure of embankment dams. Now in Japan, trial of *"Guidelines for Seismic Performance Evaluation of Dams during Large Earthquakes (Draft)"* has been conducted. According to the guidelines, earthquake-induced settlement of dam crest should be basically evaluated by the method combining dynamic analysis and Newmark's method. This method uses the dynamic analysis based on the equivalent linearization method, and next calculates the deformation as an amount of displacement of the slip circle due to earthquake response. But other deformation evaluation method for embankment dams has been recently developed based on cumulative damage theory. The deformation evaluation method based on cumulative damage theory also uses the dynamic analysis based on the equivalent linearization method, and calculates the deformation as a residual strain by cyclic loading due to earthquake response.

In this paper, we first describe dynamic analysis based on the equivalent linearization method for a rockfill dam with a central earth core, and then evaluate the settlement obtained from cumulative damage theory. The results of our research make it clear that the settlement of model dam crest ranges from about 36cm to 160cm based on cumulative damage theory due to an earthquake motion with a maximum acceleration of more than 700 gal. The observed settlement of the crest of existing embankment dams due to earthquakes are compared to the calculated values based on the cumulative damage theory, and we confirm that relationship between observed values of existing dams and calculated values of model dam has a good coincidence and that cumulative damage theory is available to evaluate the settlement of dam crest due to a large earthquake.

INTRODUCTION

In Japan, after the Kobe Earthquake, which caused serious damages to civil engineering structures, seismic design standards for various kinds of structures have been revised so as to consider earthquake motions ("Level 2 earthquake motions" defined as the "largest-class earthquake motions, approximately corresponding to the concept of Maximum Credible Earthquake (MCE)) much larger than those used in conventional designs. In March 2005, Japanese standards method for seismic performance evaluation of dams against "Level 2 earthquake motions" was conducted as the trial of *"Guidelines for Seismic Performance Evaluation of Dams (Draft)"* (MLIT, 2005).

According to the guidelines, the seismic safeties of embankment dams are investigated by the

deformation due to the "Level 2 earthquake motions". The earthquake-induced settlement of the embankment dam crest should be basically evaluated by the Newmark's method and/or Watanabe-Baba method. These methods use the dynamic analysis based on the equivalent linearization method, and in the following calculation the deformation is obtained as an amount of displacement of the slip circle due to earthquake response.
The method investigated in this paper is based on a concept of residual deformation of embankment dams, which is caused by the residual strain inside the embankment dam body due to cyclic loading of an earthquake. The method estimates residual deformation by static deformation analysis using stress-strain relationships, in which accumulation of residual strains are included, and assuming that the rigidity is apparently decreased by cumulative damage. (Lee et al., 1974)
Such deformation of embankment dams due to an earthquake has been considered to be precedent settlement, prior to settlement caused by self-weight over a long period of time, and has been included in freeboard as surplus banking in design. (JDEC, 2001) On the other hand, during the Niigata-ken Chuetsu Earthquake in 2004, a large settlement was observed at dam at which consolidation settlement had almost finished before the earthquake, such as New-Yamamoto Regulating Reservoir of East Japan Railway Company. However, actual settlement values based on the earthquake mechanism are not usually known in most dams, because of the lack of meaning data just before the earthquake. In this study, settlement was estimated by giving earthquake motions to dam models based on the above-mentioned concept of cumulative damage theory.

PREVIOUS STUDIES ON EARTHQUAKE-INDUSED DEFORMATION

Classification of residual deformation

Permanent deformation of the embankment dam body due to an earthquake can be broadly classified into shear deformation and shaking deformation (JDEC, 2001). The shear deformation is also called plastic deformation and is conceptually defined as permanent deformation caused by irreversible strain that is caused by increased shearing stress during an earthquake. When the shearing stress exceeds the peak strength, large strain remains and sometimes causes failure that accompany large slip surface.
The embankment dams settle down during construction and after completion by their self-weight. The settlement gradually slows down but continues over a long period. The shaking deformation is the precedent settlement caused by an earthquake, and that of an embankment dam is considered to be caused by resultant compressive deformation from breakage of contact face of rock materials and relative movements of particles. Thus, the amount of shaking deformation of dam is strongly affected by the period after its completion. Such deformation of an embankment dam is usually contained in the free board of surplus banking so as to cover the estimated deformation, and is considered not to cause a problem. (JDEC, 2001)

Observed earthquake-induced deformation of embankment dams

Before showing the deformation analysis method, the observed earthquake-induced settlements of the existing embankment dam are introduced after Yamaguchi and Sawada (2003). Fig. 1 shows the relationship between the maximum horizontal ground acceleration observed or estimated at dam site foundations of 12 dams in Japan, which suffered settlement of the bodies during large earthquakes of at least M6.5, and the ratio of maximum settlements "d_{vmax}" at the crest to the dam heights H. In the figure, dams (square symbol) that were older than three years (after the completion of embankment) when they were affected by the earthquakes and those (circle symbol) that were younger are shown with different symbols. Embankment dams gradually settle down along with passage of time, but settlements are relatively large during the first three years after completion. Hence the risk of suffering large settlement is likely to be high when embankment dams experience earthquake motions in their first three years. Makio Dam and Miboro Dam whose maximum ground accelerations were estimated by calculation are shown with "E" after the dam's name.

The maximum observed settlement was approximately 15.0 cm at the crest of Makio Dam (H = 105.0 m) during the Western Nagano prefecture Earthquake (1984), which was assumed to have given a maximum ground acceleration of 0.4 to 0.5G (1G: gravitational acceleration, 980 gal). The ratio of the settlement to the dam height was 0.143%, which was also the largest among 12 dams.

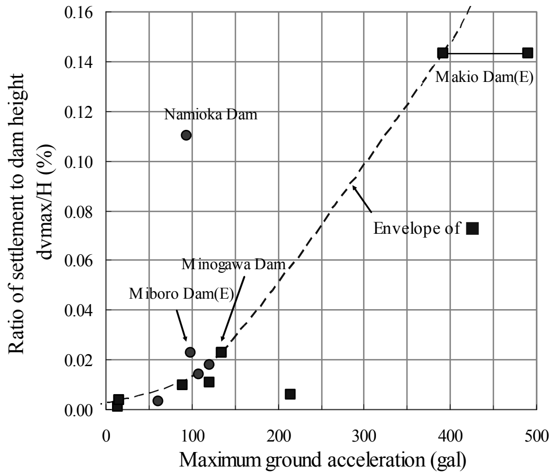


Fig. 1 Maximum ground acceleration vs. ratio of settlement to dam height (Yamaguchi and Sawada, 2003)

At the time of the Western Nagano prefecture Earthquake (1984), Makio Dam was more than 20 years after the completion, and settlement by self-weight was likely to have almost ended. Thus, the value shown in Fig. 1 is likely to correspond to settlement exceeding time historical settlement, which was caused by the maximum ground acceleration of 0.4 to 0.5G shaking.

Although the maximum ground acceleration was only about 100 gal, Namioka Dam (H = 52.0 m) settled down for 0.110% of the height (settlement of 5.7 cm). The dam was attacked by the 1983 Nihonkai-chubu Earthquake in about two years after the completion of the embankment. Maximum ground accelerations of about 100 gal or smaller have caused maximum settlements of the crest of about 0.02% in dams that were at least 3 years old and 0.11% in dam that was newer than 3 years. Since no signs of slip were observed in these dams, such a subsidence is likely to correspond to the settlements caused by shaking of the bodies by maximum ground acceleration of about 100 gal.

ANALYTICAL METHOD

Analytical procedure

Yamada et al. (1990) used the theory of cumulative strain softening to investigate the residual settlement of the foundation of Akashi Strait Bridge. And an embankment dams had also been analyzed by Sato et al, 2001. This paper briefly describes an application of the cumulative damage theory to embankment dams. In dynamic analysis of an embankment dam, the initial stress states before an earthquake greatly affect the stress and deformation during and after the earthquake. Thus, initial stresses should be first calculated so as to reflect the history of loading, such as banking and reservoir filling, and then dynamic analysis is executed. Since these processes are also used for ordinary seismic performance evaluation.

Method of analyzing deformation due to cumulative damage

This section describes a method for analyzing deformation based cumulative damage theory in detail. The method assumes that the permanent displacement of the embankment dam body due to an earthquake is attributable to the residual strain of the construction materials caused by cyclic loading. A procedure of the analysis used in this study is shown in Fig. 2.

The detailed procedures are described below,

[1] Based on the results of cyclic triaxial tests for the construction materials or experimental results of similar materials of the other dams, cyclic undrained strength (relationship

between cyclic shear stress ratio, SR_d and the number of cyclic load, N_c) and deformation properties (G - γ and h - γ) are modeled.

Here, the ratio between the dynamic shear stress ($(\sigma_1 - \sigma_3)/2$) calculated from the results of dynamic analysis and the mean effective consolidation stress (σ'_{mc}) is defined as the shear stress ratio ($SR_d = (\sigma_1 - \sigma_3)/2\sigma'_{mc}$).

[2] Initial stress analyses (analyses of embankment, seepage and reservoir filling) and seismic response analysis based on the equivalent linearization method are conducted to calculate the cyclic shear stress at each element of the dam body model. The time history of the shear stress ratio SR_d is arranged as "pulse". Time history of strain is calculated using the time history of the pulse and the results of the dynamic strength (cyclic triaxial) tests described in [1].

[3] Based on the stain softening properties in the cumulative damage theory, "SR_d–N_c" curve determined in [1] is applied to the time histories of cyclic shear stress ratio SR_d and the number of cycles N_c. A detail of the method is described in the next section. Shear rigidity $G_d(t)$ is calculated using the following equation and the time histories of strains generated at each element. The rigidity G_1 at the final step is determined.

$$G_d(t) = \frac{\sigma'_{mc}\, SR_d(t)}{(1+\nu)\varepsilon_1^{+}(t)} \tag{1}$$

where, $G_d(t)$: shear rigidity at time t,
σ'_{mc} : mean effective confining stress,
$SR_d(t)$: shear stress ratio at time t,
ν : dynamic Poisson's ratio,
$\varepsilon_1^{+}(t)$: strain at time t.

[4] Using the rigidity values (G_0, G_1) before and after an earthquake, a self-weight deformation analysis is executed, and the difference is calculated as the residual deformation after the earthquake.

The processes of the deformation analysis and examples of calculated response results at an element are shown in Fig. 2.

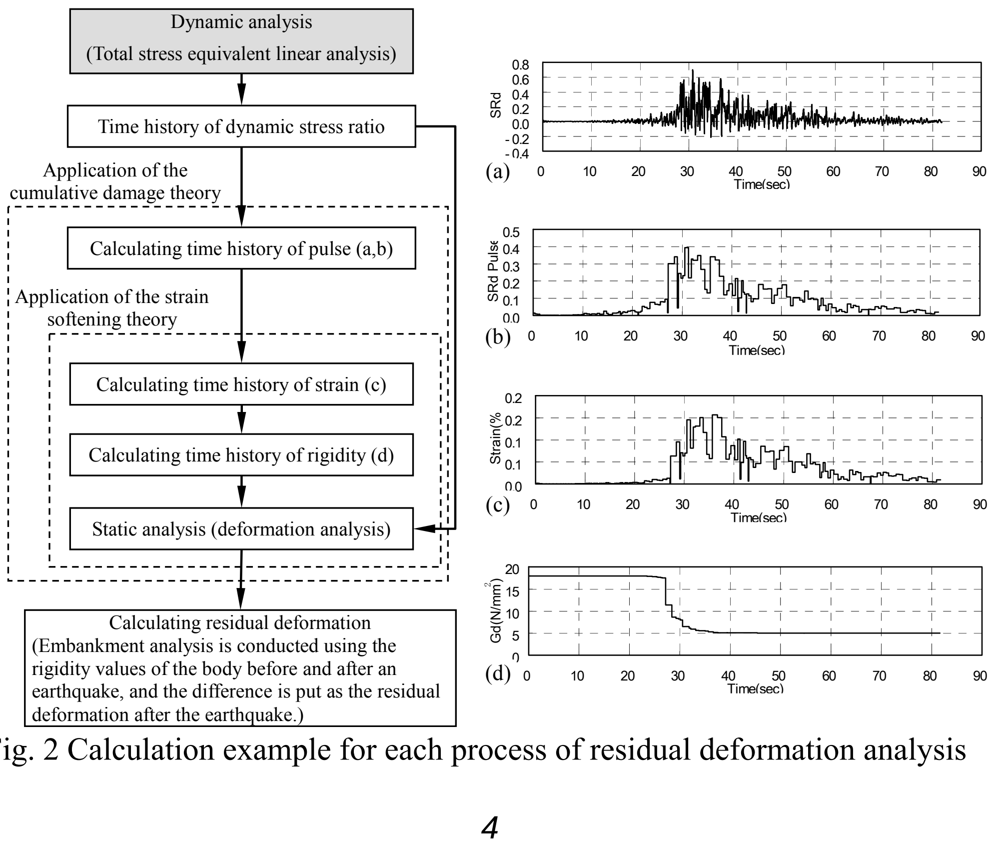


Fig. 2 Calculation example for each process of residual deformation analysis

Stain softening properties in the cumulative damage theory

Methods for deciding pulses and determining the time history of strains from the time history of the pulses and the results of cyclic triaxial tests described in the former section are here called the "stain softening properties in the cumulative damage theory", are illustrated in Fig. 2, and are explained in detail as follows.

STEP 1

SR_d time history is calculated from dynamic shear stress as described in previous section. The time history of all SR_d pulses is determined for pulses SR_{d1}, SR_{d2}, SR_{d3}, ···. Each pulse is obtained from the zero crossing method. The process is shown in Fig. 3 (a). Each pulse value is the mean of amplitudes in the absolute value of two waves, which are formed by two zero crossing points next each other.

STEP 2

$(\varepsilon_1^+)_1$ generated by the first pulse SR_{d1} is the strain generated by the first loading cycle, N_c=1, during a cyclic loading test of an amplitude of SR_{d1}(Fig. 3 (b)).

STEP 3

$(\varepsilon_1^+)_2$ generated by the next pulse SR_{d2} is determined as described below:

[1] Damage level D_1, which is defined as received reciprocal of deformation modulus, of the specimen just after loading of SR_{d1} is indicated in Eq. (2).

$$D_1 = \frac{(\varepsilon_1^+)_1}{SR_{d1}} \tag{2}$$

[2] As long as the SR_d and ε_1 has a linear relationship, strain $(\varepsilon_1^+)_{20}$ generated with a pulse intensity of SR_{d2} under this damage level can be expressed as:

$$(\varepsilon_1^+)_{20} = \frac{SR_{d2} \times (\varepsilon_1^+)_1}{SR_{d1}} \tag{3}$$

This shows the strain when a pulse of SR_{d2} is applied on a specimen that has been damaged by loading of one SR_{d1} pulse.

[3] This can also be regarded as a state in which SR_{d2} is cyclically applied for N_{20} times, which gives a damage equivalent to a strain of $(\varepsilon_1^+)_{20}$, to a specimen to which pulses of a certain intensity SR_{d2} have been cyclically and continuously applied from the beginning. This N_{20} is calculated (Fig. 3 (c)).

[4] In a dynamic analysis, a loading of SR_{d2} pulse is applied on a specimen after the state of [3]. At that time, the strain $(\varepsilon_1^+)_2$ is determined by the results of cyclic triaxial tests as $N_c = N_{20}+1$ (Fig. 3 (d)).

[5] For each of the pulses SR_{d3}, SR_{d4}, ···, $(\varepsilon_1^+)_3$, $(\varepsilon_1^+)_4$, ··· are determined by cyclic processes [1] to [4]. This enables strain to be determined for each pulse, which is the time history of strain.

STEP 4

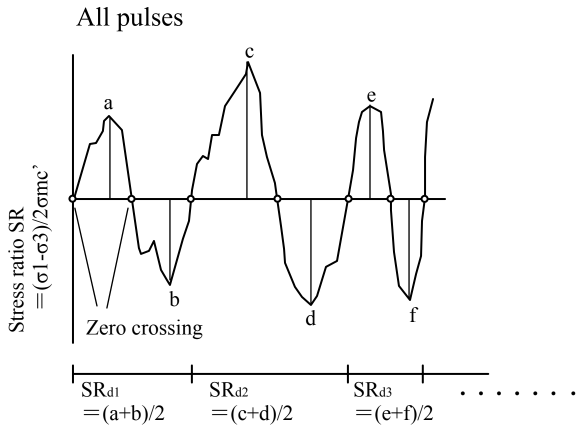
For each element, the time history of ε_1^+ is obtained.

APPLICATION OF THE METHOD TO AN EMBANKMENT DAM

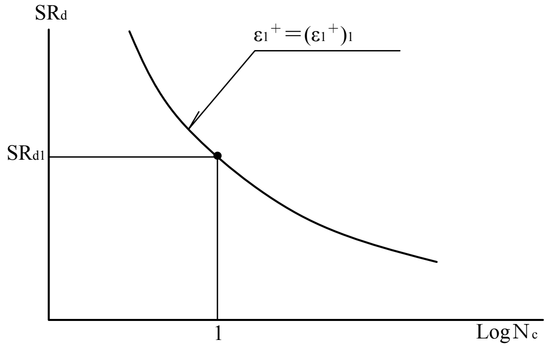
This chapter describes an application of the above-mentioned deformation analysis method to an embankment dam. The dam shown in Fig. 4 is used as a model. Main features of the model dam are summarized in Table 1. Finite element model is shown in Fig. 5. For embankment analysis, the

Table 1 Specifications the model dam

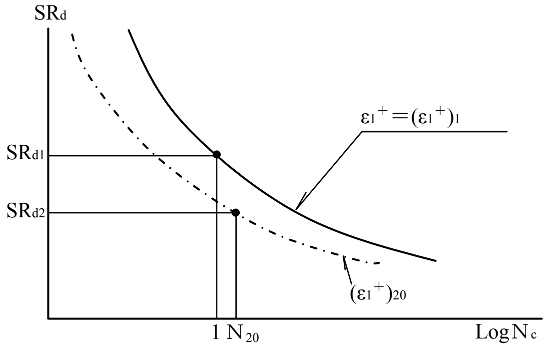
Item	Specifications
Type of dam	Rockfill dam with a central earth core
Dam height	90.000 m
Dam crest length	565.000 m
Dam volume	4,418,000 m^3
Elevation of the dam crest	EL.308.000 m
Normal water level (water depth)	EL.293.500 m (73.5 m)
Water depth / dam height (%)	83.9 %
Inclination of the upstream face	1.0:2.6
Inclination of the downstream face	1.0:2.0



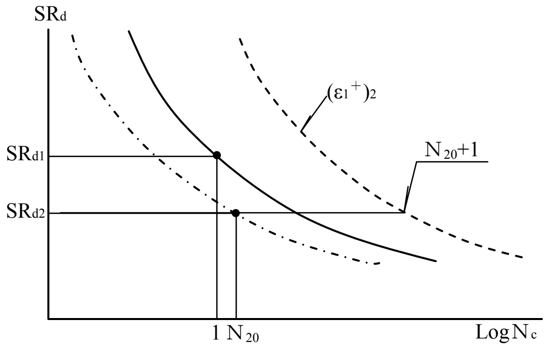
(a) STEP 1: Calculating all pulses



(b) STEP 2: Strain by the first pulse



(c) STEP 3-[3]: Calculating the number of cyclic loading of SR_{d2} resulting in strain equivalent to that in STEP 2



(d) STEP 3-[4]: Calculating strain by SR_{d2} loading

Fig. 3 Concept of the strain softening properties in the cumulating damege theory

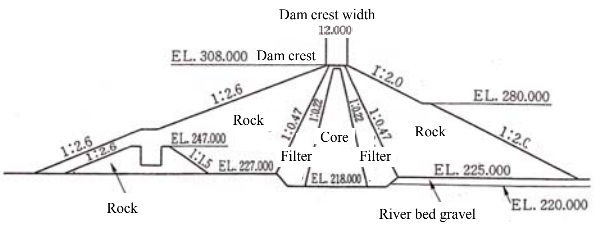


Fig. 4 The cross section of the model dam

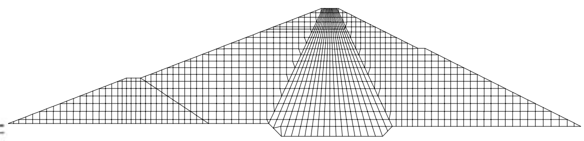


Fig. 5 The finite element model

model with the foundation is used. The model without the foundation shown in Fig. 5, is used for seismic response analysis and residual deformation analysis.

The earthquake motion shown in Table 2 is prepared by assuming that an active fault directly under the dam caused an earthquake of M7.8, applying the *distance attenuation formula for dams* (Inomata et al., 2005) between the fault and the dam. The attenuation formula calculates the acceleration response spectra at the dam rock foundation. The response spectra of waveforms observed at the foundation of Hitokura Dam during the Kobe Earthquake are adjusted so as to coincide with these from the attenuation formula in amplitudes. This earthquake motion is supposed to be considerably great as that at dam sites in Japan, because it is the inland earthquake of the M7.8 scale. The time history of waveform is shown in Fig. 6.

The dynamic properties for the analysis are shown in Table 3. The shear strain dependent curves of shear modules ratio G/G_0 and the damping values of the embankment materials are based on the laboratory test results. (Matsumoto et al., 1987)

Table 2 Specifications of input earthquake motion

Original waveform	Direction of acceleration	Maximum acceleration (gal)	Time of maximum acceleration (sec)	Duration (sec)
Hitokura waves	Along the stream	758	7.67	10.48
	Vertical	530	6.96	10.48

The distribution of maximum horizontal response acceleration calculated by dynamic analysis is shown in Fig. 7. The horizontal response acceleration near the crest was as large as 1,600 gal. The natural period of the dam body, T was 0.494 seconds before the earthquake, but the dynamic analysis of the equivalent linearization method makes T shift to 0.773 seconds.

Analysis of deformation caused by cumulative damage needs results of undrained cyclic strength tests. There are not test data available in Japan, and data was not available for the dam used in this study. Since materials to be newly used for

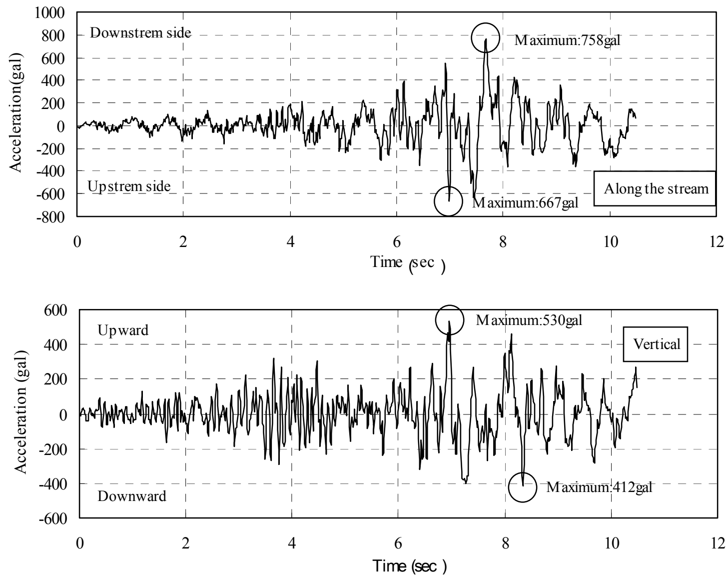


Fig. 6 Time history of acceleration of input earthquake motion

Table 3 Dynamic properties

Zone	Density		Initial shear rigidity	Poisson's ratio	Dependency of G/G_0 and h on shear strain	Dissipation damping
	Wet (t/m^3)	Saturated (t/m^3)	G_0 (N/mm^2)			
Core	2.22	2.23	$\{299(2.17-e)^2/(1+e)\}\sigma_m^{0.7}$ Note2	Sawada method Note 1	Note 2	10%
Filter	2.13	2.24	$\{299(2.17-e)^2/(1+e)\}\sigma_m^{0.7}$ Note2			
Rock	1.94	2.15	$\{299(2.17-e)^2/(1+e)\}\sigma_m^{0.6}$ Note2			

Note 1: Poisson's ratio (by Sawada and Takahashi, 1975):

$\nu = 0.375 - 0.006\ Z^{0.58}$ for rock and filter materials (on the downstream side and above the seepage line on the

$\nu = 0.490 - 0.001\ Z^{0.95}$ for rock and filter materials (below the seepage line)

$\nu = 0.450 - 0.006\ Z^{0.60}$ for core materials

where, Z (m) is the depth from the surface of the dam body.

Note 2: $G/G0$-γ of core materials are the same as that of filter materials.

h-γ of core materials are given by Ogata and Yasuda (1984).

The mean principal effective stress σ_m was calculated as follows.

$\sigma_m=\{\sigma_1+\sigma_3+\nu(\sigma_1+\sigma_3)\}/3$

The void ratios of the materials of the dam body were:

Core: 0.345, Filter: 0.164, Rock: 0.284

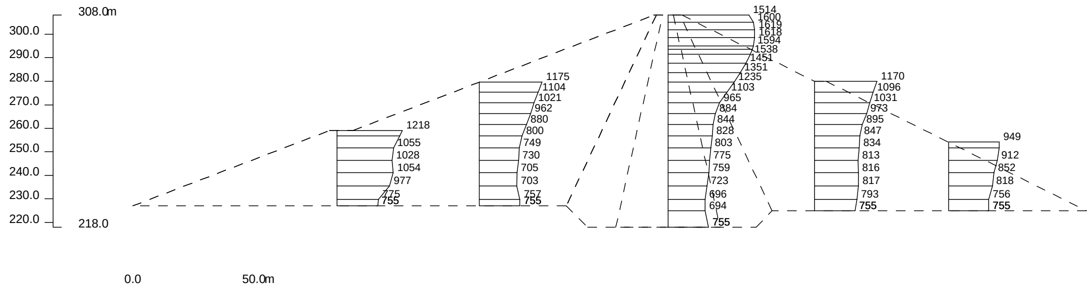


Fig. 7 Distribution of maximum horizontal response acceleration

testing were not available, test results of similar materials of other dams were decided to be used for the analyses in various combinations to minimize the effects attributable to the differences in materials. (TICSEY(2006), Yonezaki et al.(2000), Nakamura et al.(1995) and Matsumoto et al.(1996)

Analytical cases are shown in Table 4 with the combination of materials and their properties. Properties of each material are shown in Table 5. Undrained cyclic strengths (number of cycles N, and cyclic shear stress ratio SR_d) are shown in Fig. 8. “Experimental” in the figure denotes experimental values, and “regression” denotes the regression line estimated from the experimental values and data obtained in previous studies.

Table 4 Analyzed cases

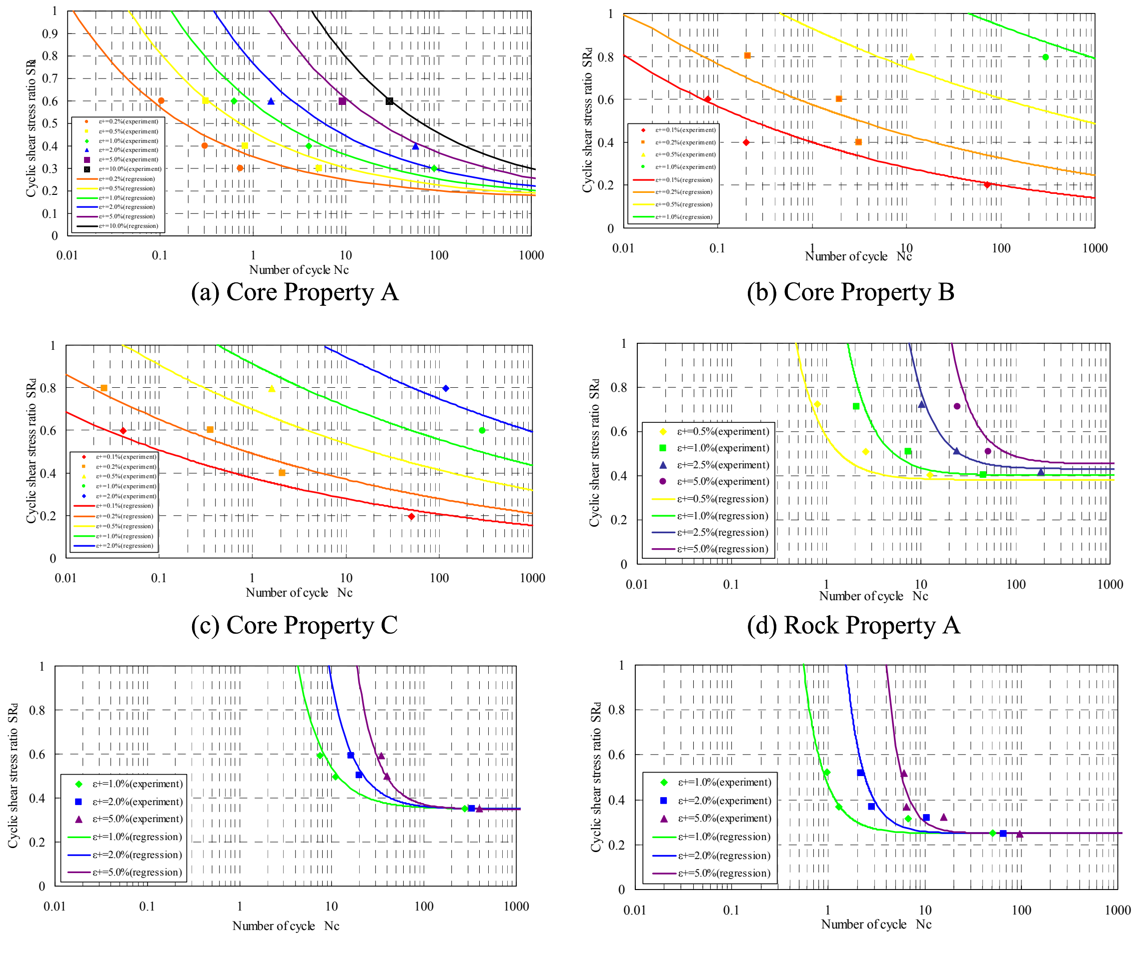
Dam body zone		Case-1	Case-2	Case-3
Core materials	Saturated	Core Property A	Core Property B (saturated)	Core Property A
	Un-saturated		Core Property C (un-saturated)	
Filter materials		Filter Property A	Rock Property B	Rock Property B
Rock materials		Rock Property A	Rock Property B	Rock Property B

Note:
Core Property A is after Yonezaki et al. (2000).
Core Property B,C are after TICSEY (1998).
Filter Property A and Rock Property B are after Nakamura et al. (1995).
Rock Property A is after Matsumoto et al. (1996).

Table 5 Comparison of material properties

Property	Zone	Void ratio e	Dry dencity (t/m^3)	ϕ'(°)
Model dam	Core	0.345	1.97	35.0
	Filter	0.164	2.32	36.0
	Rock	0.284	2.04	41.0,42.0
Core Property A	Core	0.467	1.93	-
Core Property B,C	Core	1.800	0.94	32.0
Rock Property A	Rock	0.206	2.46	-
Rock Property B	Rock, Filter	0.215-0.271	2.126-2.210	49.4
Filter Property A	Filter	0.283-0.369	1.975-2.112	42.4

Properties of core materials were classified into Core Properties A, B and C. Properties B and C are the properties of the same material under the saturated and the unsaturated conditions, respectively. The strength regression lines for Core Property A were stronger than the experimental strength values, but the regression lines by Yonezaki et al.(2000) were decided to be used.



(a) Core Property A (b) Core Property B

(c) Core Property C (d) Rock Property A

(e) Rock Property B (f) Filter Property A

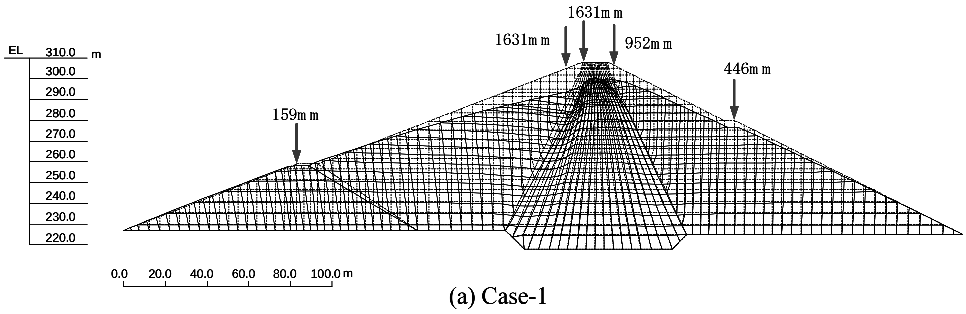
Fig. 8 Relationship between the number of cycle and cyclic stress ratio

Since Core Property A was smaller in dry density and larger in void ratio than the property of the materials of the model dam, use of Core Property A possibly leads larger settlement than that the existing dam. Although, Core Properties B and C also were larger in void ratio and smaller in dry density than the property of the model dam, they would result in small settlement because of higher cyclic strength. A comparison between Core Property B (Fig. 8(b)) and Rock Properties A and B (Fig. 8(d) and Fig. 8(e)) suggests that the core property deformed little by dynamic loads. In general, soil materials deform more easily than coarse-grained materials, but Core Property B was more difficult to deform than Rock Properties A and B. Thus, use of Core Property B may result in underestimation of settlement.
Filter Property A was very prone to deformation as shown in Fig. 8(f). Since filter materials used in most embankment dams in Japan are highly rigid and hardly deformed, Rock Property B was used instead of Filter Property A as the filter materials in Case-2 and Case-3.
Maximum settlement values calculated based on the strain softening properties in the cumulative damage theory are shown in Table 6. In the table, values under the upstream and downstream sides denote the maximum on the corresponding surface of the dam. Deformation diagrams are shown in Fig. 9 for Cases 1 to 3.
Settlement at the crest was the largest in Case-1 with 1,631 mm. This was likely attributable to the use of Core Property A, whose effects are described above, and Filter Property A, which was easy to deform. On the other hand, Case-2 resulted in the smallest settlement of only 360

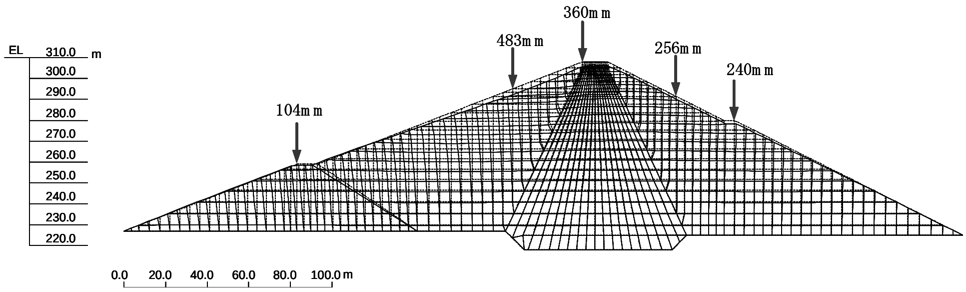
mm. This value could be underestimated since Core Properties B and C were used for the core materials. Case-3 resulted in intermediate settlement because the core materials were set to be prone to settlement, and filter and rock materials were set to be strong in deformation, From the view point of material properties Case-3 was likely state to analyze the settlement of the model dam.

Table 6 Calculated results of maximum settlement of the dam body

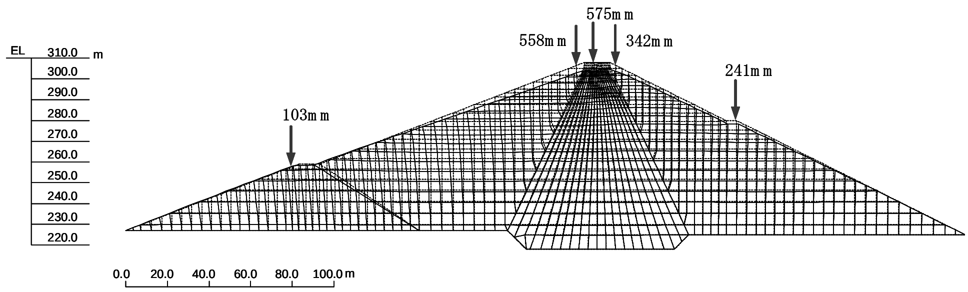
Case	Upstream side (mm)	Crest (mm)	Downstream side (mm)	Core Property	Filter Property	Rock Property
Case-1	1,631	1,631	952	Core Pproperty A	Filter Property A	Rock Property A
Case-2	483	360	256	Core Property B,C	Rock Property B	Rock Property B
Case-3	558	575	341	Core Property A	Rock Property B	Rock Property B



(a) Case-1



(b) Case-2



(c) Case-3

Fig. 9 Calculated deformation

The calculated results were comparatively analyzed the trends in observed settlement of 12 rockfill dams due to earthquake in Japan. The settlement value of the dam may be greatly changed by the situation of the foundation, a kind and the length of the earthquake and so on. However, here, the calculated results were analyzed based on the observed data. The settlements of dams for which observed data sets were available are shown in Fig. 10. The calculated values were larger than regression line based on the observed results in Case-1 and smaller in Case-2. Thus, Case-3 was the most similar to the regression line. Thus, the analytical results were likely to be valid from the settlements observed at other dams in the past.

CONCLUSIONS

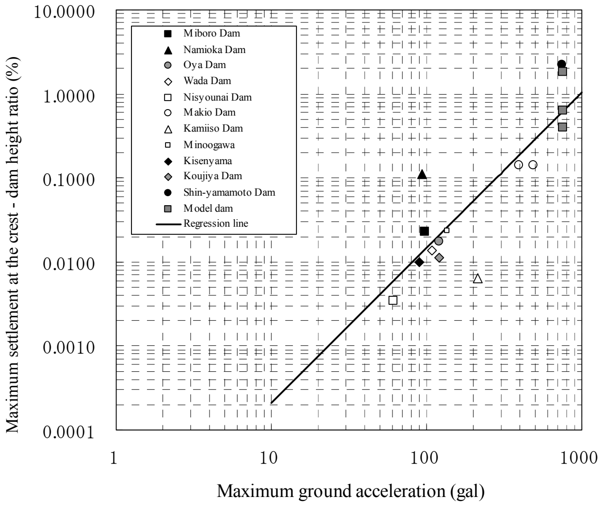


Fig. 10 Relationship between settlement at the crest and maximum acceleration of the ground

This paper described a method for analyzing deformation of embankment dams caused by cumulative damage and an application of this method to a model dam.
The method for analyzing deformation caused by cumulative damage involves determining residual deformation of embankment dam body doe to an earthquake by assuming that accumulation of residual strain by cyclic stress appears to be declines in rigidity. In this analysis, the dynamic shear properties for the cumulative damage method were obtained from the characteristic of other embankment dams.
Evaluation of the analytical results was judged to be appropriate comparing with the observed settlement data of other dams after earthquakes. Thus, the method is likely to be effective for predicting settlement of embankment dams due to an earthquake.
However, not only property values of dam bodies but also more case studies using observed data are necessary to obtain prediction values that can be used in practice.
The dams increasingly need seismic performance evaluation against large earthquakes. However, much is left unknown on the mutual relationship between settlement caused by shaking deformation and by shear deformation. In the analysis described in this paper, settlements of several tens of centimeters to about 1.5 m were predicted to occur in a dam of about 90 m height during a large earthquake motion exceeding 700 gal. On the other hand, another analysis, in which shear deformation was considered under the same conditions, estimated settlement of several tens of centimeters. It is not yet known which of the settlements becomes dominant during an actual earthquake and whether the two should be added or not in the case of the estimation of earthquake-induced settlement.
Methods for evaluating the seismic safety of embankment dams should be investigated and elaborated, and the authors hope that this paper would contribute to the establishment of evaluation methods of settlement.

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