Estimation of residual deformation of fill dams subjected to earthquake

S. Tani[1], S. Tsukuni[2] and T. Shiomi[3]

[1] *Director, Dept. of Structural Engineering, National Institute for Rural Engineering, Tsukuba, Japan*
[2] *Senior Researcher, Takenaka Civil Engineering & Construction Co. Ltd., Inzai, Japan*
[3] *Senior Researcher, Takenaka Construction Co. Ltd., Inzai, Japan*
Email: stani@affrc.go.jp, stsukuni@takenaka.co,.jp, shiomi@takenaka.co.jp

ABSTRACT :

There are many fill dams in Japan that have been constructed using empirical techniques, and they have been used safely. However, their safety has not been sufficiently verified for level 2 earthquakes. Besides, the safety of fill dams during level 2 earthquakes cannot be guaranteed from the point of view of slope stability. In this paper, actual value is proposed about performance object based on the behaviors of past earthquakes, modeling constructed fill dams, and [1] setting a performance object, [2] assessment of present stability at earthquakes of level 2, [3] safety at earthquakes supposing actual cross sections in case of reinforcement for earthquake-resistant and methods of reinforcement meeting requirements of the performance object are studied. The fill dams are constructed on the basis of proven techniques. In the examples investigated, first, the performance objective indicating the amount of residual settlement at the crest of fill dams is set for earthquake-resistant dams for a level-2 earthquake motion. Next, a level 2 reference earthquake motion is set, and the amount of deformation of the dam structure is determined by performing earthquake response analysis using the finite element method. Because the performance objective of fill dams does not meet the requirements of level 2 earthquakes, the construction of counterweight fills for making the dams earthquake resistant is also studied.

KEYWORDS: Fill dam, Performance-based design, Earthquake

1. INTRODUCTION

In large-scale seismic events such as the 2004 Mid Niigata Prefecture (Chuetsu) Earthquake and the 2008 Iwate-Miyagi Inland Earthquake, earthquake motions of approximately 500–1000 cm/s^2 have been observed at the bases of dams; however, dams that have been constructed using modern techniques have not been seriously damaged. In Japan, there are many fill dams that have been constructed using empirical techniques, and they have been used safely. However, the safety of the fill dams has not been sufficiently verified for level 2 earthquakes. These fill dams have been traditionally constructed according to a “specification design” based on design standards and other guidelines that specify details of embankment materials, structural design methods, and safety factors against sliding. For traditional construction, limit equilibrium stability analyses is performed by using methods such as the circular arc method; however, this method is insufficient for designing fill dams that can withstand large-scale earthquake motions. Since the 1995 Hyogoken-Nanbu Earthquake, the concept of “performance-based design” that focuses on only the performance of structures has been introduced. Currently, the design of earth structures is currently shifting from the conventional specification design to a performance-based design involving performance criteria.

Since the 1995 earthquake, methods for analyzing the amount of deformation of dam structures during earthquakes have been actively studied to understand the response of the structures to level 2 earthquake motions. A performance objective for earthquake-resistant dams is defined in the Guidelines for Seismic Safety Evaluation of Dams (Draft) and Explanation[1)] and some other documents; the guidelines also deal with the verification of deformation, and the evaluation criterion for the performance objective is considered to be the amount of settlement. In this study, the safety of fill dams during large-scale earthquakes is examined by performing dynamic response analysis, and the amount of deformation of dam structures is determined. The earthquake resistance performance of the fill dams is verified. Furthermore, reinforcement methods meeting the requirements of the performance objective are also considered.

2. BEHAVIOR OF FILL DAMS AND PERFORMANCE-BASED DESIGN TO WITHSTAND LARGE-SCALE EARTHQUAKES

There are very few cases where existing fill dams have been subjected to level 2 earthquake motions, but damages have been observed in all dams. Due to the continuity of basic capabilities, the fill dams that have been constructed in accordance with modern design standards are considered to have high earthquake resistance. The cases where fill dams have been damaged by earthquakes in Japan are discussed in the following. Fig. 1 shows the Kawanishi Dam damaged by the 2004 Mid Niigata Prefecture (Chuetsu) Earthquake. This is a center-core-type rockfill dam with a height of 43 m that was constructed in 1979. The dam was considerably damaged; the earthquake caused the left bank and upstream slopes to slide and resulted in a maximum settlement of 28 cm at the crest. However, the dam was not critically damaged. The base input acceleration reached 558 cm/s^2, while the maximum acceleration response reached 582 cm/s^2. Figs. 2 and 3 show the Aratozawa Dam damaged by the 2008 Iwate-Miyagi Inland Earthquake. This is a center-core-type rockfill dam with a height of 74 m that was constructed in 1998. The damage to this dam was not determined since water was stored in it, but a settlement of approximately 20–40 cm at the crest of the dam and faulting of approximately 10 cm at the interface between the natural ground and the dam body on the right bank are currently observed. A base input acceleration of 1024 cm/s^2 (the actual acceleration is considered to have exceeded this value, which represents the measuring limit of the accelerometer) and a maximum acceleration response of 525 cm/s^2 at the crest are observed. This indicates that an input earthquake motion is attenuated on the dam and the acceleration at the crest is substantially small. The fill dams faced significant earthquake motions, but did not lose their capabilities; this fact demonstrates the high safety level of fill dams constructed using modern techniques.

Figure 1 Damaged Kawanishi Dam (shallow sliding of the upstream slope, 2004 Mid Niigata Prefecture) Earthquake)

Figure 2 Aratozawa Dam

Figure 3 Faulting of approximately 10 cm at the interface between the natural ground and the dembankment on the right bank (2008 Iwate-Miyagi Inland Earthquake)

Fig. 4 shows the Niwaikumine Dam that was significantl ·Oki

Earthquake; the earthquake caused the upstream slope of the embankment to slide and resulted in a maximum settlement of 1.6 m. The dam was constructed around 1930 using empirical techniques. The maximum velocity at the dam's base is estimated to be 200–250 cm/s^2, and it is lower than that of the fill dam damaged by the Iwate-Miyagi Inland Earthquake. These old fill dams were significantly damaged by past earthquakes. For this reason, it is necessary to verify the safety of this type of old fill dams by considering the occurrence of level 2 earthquake motions.

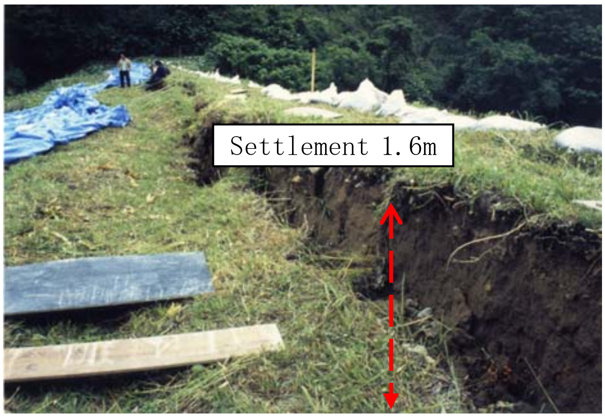

Figure 4 Damaged Niwa- Ikumine Dam (1993 Hokkaido-Nannsei-Oki Earthquake)

Next, the performance-based design of fill dams is described. In response to the Technical Barriers to Trade (TBT) Agreement of the World Trade Organization (WTO) in 1995, Japan's design method for social infrastructure has been shifting from the traditional specification design system to performance-based design for enhanced reliability. The need for consistency with international technical criteria represented by ISO 23469 (Bases for design of structures—Seismic actions for designing geotechnical works)[2)] is also a reason for the introduction of the performance-specified design. The most important feature of the performance-based design is the performance criteria.

In view of the functionality and safety of fill dams, the dams must limit their settlement so that their water-storing capability can be maintained, even after a level 2 earthquake. For this reason, it is necessary to design a fill dam such that the crest elevation is not below the reservoir level or such that its settlement falls within a fill's freeboard. A freeboard of 1.0 m or more is ensured in the fill dams under the design criteria. Consequently, approximately 1 m of freeboard should be defined as the performance objective regardless of the dam height. However, in view of variations in the soil constants used for the analysis, the errors in the analysis, and other factors, a settlement of approximately 50 cm, or 50% of 1 m, is considered as an allowable settlement in this report. In the following section, the earthquake resistance performance of a modeled fill dam is examined, and reinforcement methods meeting the requirements of the performance objective are considered.

3. ANALYSIS

The fill dam being analyzed is a modeled dam based on the existing dams. The modeled dam is a center-core-type rockfill dam with a height of 36.5 m. Fig. 5 shows a lateral profile of the fill dam.

3.1. Analysis conditions

For the examination of the safety of the fill dam for a level 2 earthquake, it is assumed that the dam does not lose its capabilities even after being subjected to a level 2 earthquake motion; however, the usability of the dam without loss of capabilities after the earthquake is defined to be equivalent to its earthquake resistance performance against a level 1 earthquake. Similarly, the safety of the dam is defined to be equivalent to the earthquake resistance performance against a level 2 earthquake. Since a freeboard from a high water level to the crest in a reservoir represents a distance of at least 1 $m^{3)}$, its performance objective is set as the maximum settlement of 0.5 m or less at the crest of the embankment in view of the precision of the analysis.

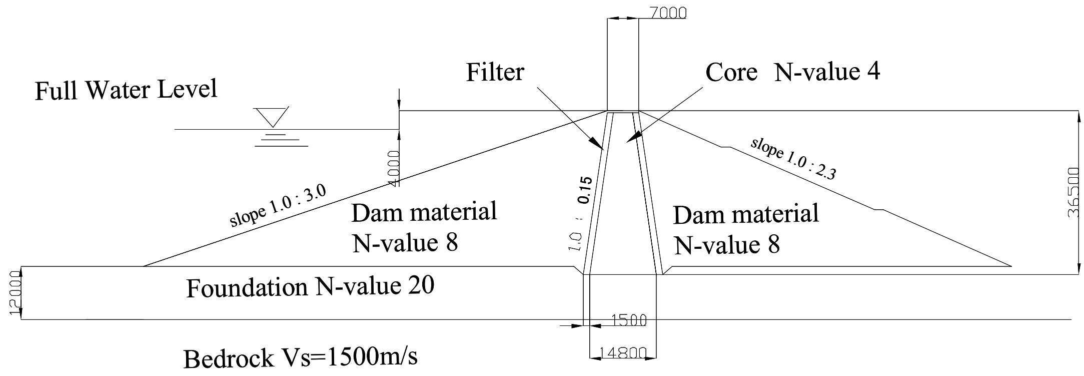

Figure 5 Cross section of the modeled dam used for the analysis

3.2. Earthquake motion used for the for analysis

It is necessary to define an earthquake motion for seismic design. In the literature[1)], an earthquake motion (earthquake motion scenario) expected to occur at a point is considered for the purpose of the design. However, when a maximum earthquake motion that was actually observed in the past at the dam point or in its vicinity or an earthquake motion with a lower-limit acceleration response spectrum for verification is dominant, the motion is used. The following two types of seismic waves are used in a reference earthquake motion that is considered as level 2 due to the absence of the maximum earthquake motion actually observed in the past; 1) a seismic wave[3)] generated using the phase characteristics of the observed wave and a seismic wave conforming to the lower-limit acceleration response spectrum(shown in Fig. 6) used for verification, a seismic wave (called Kawanish Wave) generated using the upstream and downstream elements observed on the foundation on the left bank of the Kawanishi Dam (center-core-type fill dam with a height of 43 m) in Tokamachi City for the 2004 Mid Niigata Prefecture Earthquake, 2) a seismic wave[4)] (called Miyagi Wave) resulting from the earthquake motion at a virtual site in Sendai City generated by using a fault model for the 1978 Miyagi-Oki Earthquake (shown in 7). Figs. 6 and 7 show the acceleration time history and the acceleration response spectrum of the above-mentioned two types of seismic waves.

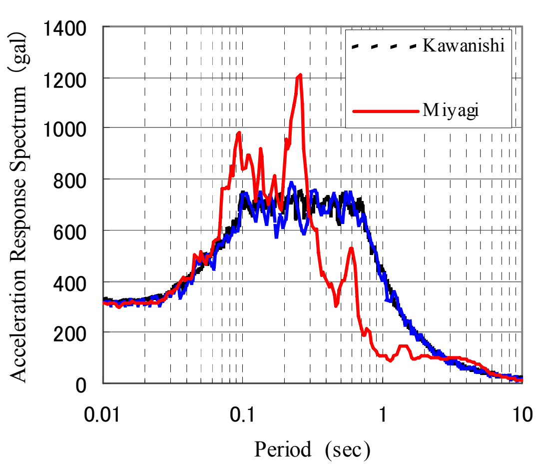

Figure 6 Seismic wave spectrum used for the analysis

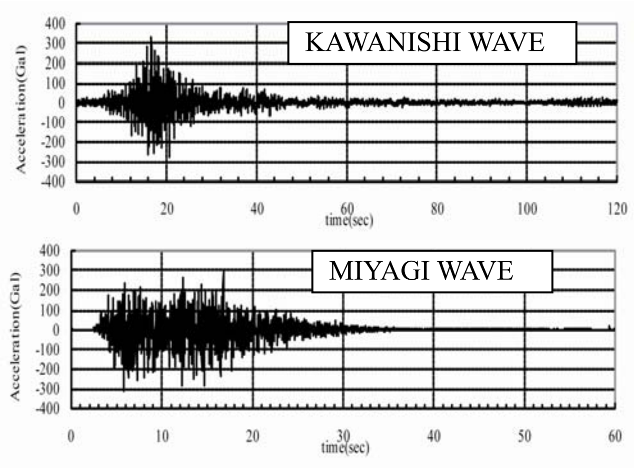

Figure 7 Seismic wave time history used for the analysis

3.3. Cross section and soil parameters used for the analysis

The cross section of the modeled dam that is modeled to be similar to an existing center-core-type fill dam is shown in Fig. 5. This section is assumed to have a height of 36.5 m, a crest width of 7.0 m, an upstream grade of 1.0:3.0 and a downstream grade of 1.0:2.3. The dam's water level is assumed to be the normal top water level (4.0 m below the crest). A foundation with an N value of 20 is set under the dam body, and a base resistant to a shear wave velocity of Vs = 1500 m/s is set under the foundation.

Figs. 8 and 9 show two sections of a counterweight fill that is considered for retrofitting measures: one section has a crest width of 5.0 m and a height of 15.3 m and the other section has a crest width of 15.0 m and a height of 15.3 m. The gradient of the fill is 1.0:3.5 upstream and 1.0:2.8 downstream; these values are smaller than those of the section. Two materials, the material of the embankment and improved soil with cement, are examined for use in the counterweight fill.

The initial stress required for FEM dynamic analysis was determined by the self-weight analysis of the Mohr-Coulomb model. For boundary conditions in the self-weight analysis, the model's bottom was fixed, and its side formed a vertical roller. In the dynamic analysis, a free field was provided at the side of the model. For the boundary conditions, a viscous boundary was set at the bottom and another viscous boundary was set between the side and free field.

Table 1 shows the soil parameters used for the analysis[5),6),7)]. Vs was calculated from the N values. The N values of the dam body and core and the internal friction angles were set on the basis of boring data obtained from existing dams. In the self-weight analysis, Poisson's ratio was set at 0.41 above the seepage line and 0.49 below the seepage line, and the coefficient of earth pressure at rest was in the range of 0.5 to 0.33 during the self-weight analysis. In the dynamic analysis, Rayleigh damping was considered as the attenuation source. The parameters for Rayleigh damping were set for 2.5% damping for horizontal primary and secondary natural periods of 0.704 and 0.363 s. A constitutive equation for soil that was used in the dynamic analysis is the Mohr-Coulomb model represents the Mohr-Coulomb model.

Table 1 Soil parameters used for the analysis

Material category	N value	Vs (m/s)	Saturation density (t/m^3)	Internal friction angle φ (°)	Cohesion C (kN/m^2)	Shear rigidity (σ_C = 100 kN/m^2)	Young's modulus (kN/m^2)
Embankment material	8	170	1.86	32	5	53754	-
Core	4	133	1.74	28	10	30778	-
Filter	-	-	2.17	40	0.1	203200	-
Foundation	20	237	2.1	32.3	0.1	117955	-
Foudation	-	1500	2.2	-	-	4950000	-
Improved soil	-	-	1.86	10	50	-	100000

*Poisson's ratio was set at 0.41 above the seepage line and 0.49 below the seepage line.

3.4. Result of FEM analysis

The maximum settlement at the crest of the dam body that is obtained from the FEM dynamic analysis is shown in Table 2. It is observed from the table that the maximum settlement in the case of the seismic wave (called Kawanishi Wave) that was generated using the phase characteristics of the observed wave and in the case of the seismic wave conforming to the lower-limit acceleration response spectrum for verification is considered as input earthquake motion is larger than that when Miyagi Wave is used. This may be due to the fact that the power near the horizontal primary natural period of 0.704 s for the Miyagi Wave is smaller than that for the Kawanishi Wave. Figs. 10, 11 and 12 show the deformations in the final stage. All the sections indicate considerable deformation upstream. The γmax peak appears upstream and corresponds to a position higher than the counterweight fill in the reinforced section. Consequently, when the dam body is retrofitted with the counterweight fill, increasing the height of the fill is more effective than widening the fill. In addition, the use of viscous improved soil in the fill increases the seismic strength.

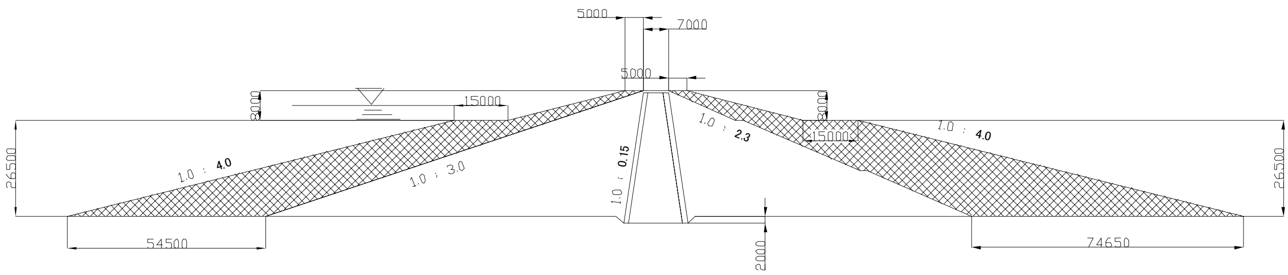

Figure 8 Reinforced section with counterweight fill ①

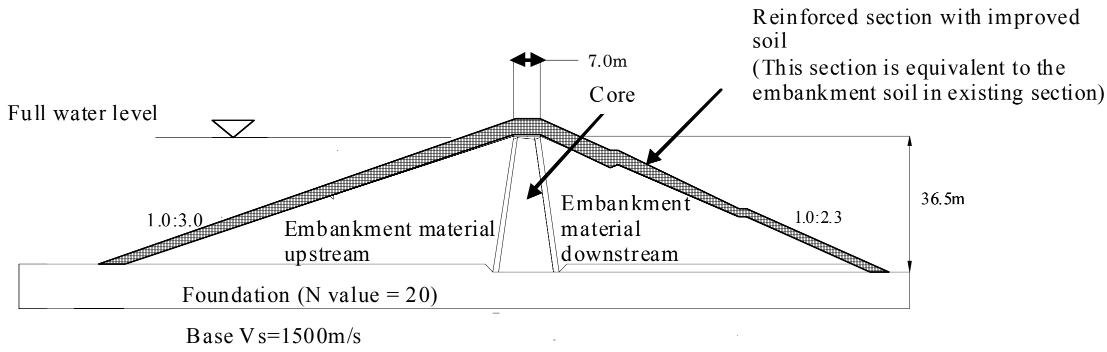

Figure 9 Reinforced section with counterweight fill ②

Table 2 Final settlement at crest

Seismic wave	Maximum settlement at the crest of the dam (m)	
	Kawanishi Wave	Miyagi Wave
Non-countermeasure	0.676	0.268
Reinforced section ①	0.384	0.073
Reinforced section ②	0.244	0.102

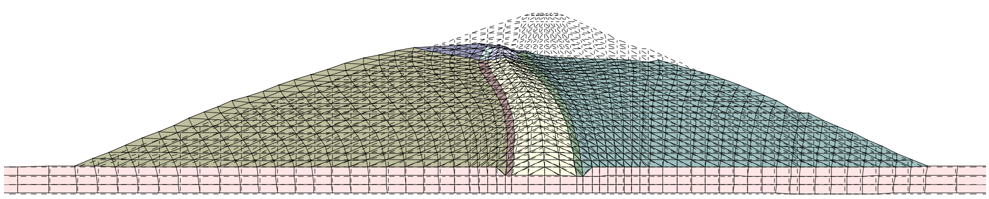

Figure 10 Final deformation of non-reinforced section

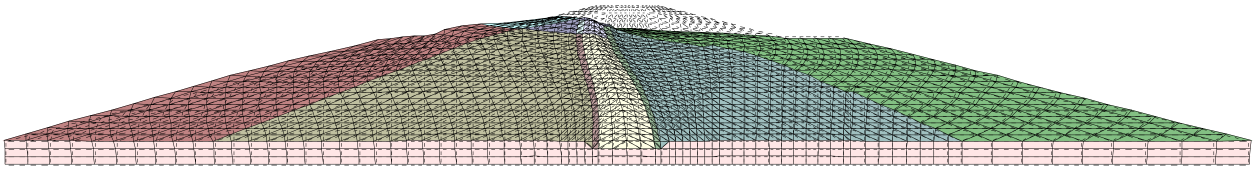

Figure 11 Final deformation of reinforced section ①

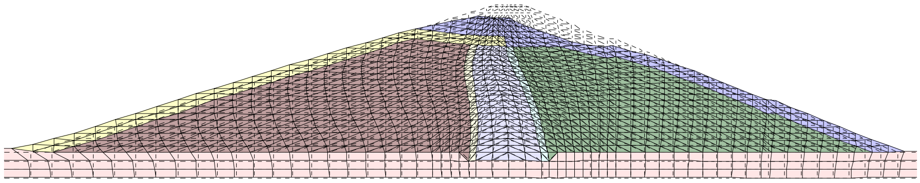

Figure 12 Final deformation of reinforced section ②

4. CONCLUSION

The findings obtained from the study can be summarized as follows:

1) In the FEM analysis, the settlement of a dam's crest varies considerably with the characteristics of the cceleration response spectrum of a seismic wave. Thus, the manner of determining the reference earthquake motion is important.
2) The train is centered on the crest of the dam body that is retrofitted with a counterweight fill. For this reason, increasing the height of the fill rather than its width is considered to be effective in reducing the settlement of the crest.
3) The seismic strength varies considerably with the method used for constructing the counterweight fill. Therefore, it is necessary to select the best antiseismic reinforcement method as well as the reinforcement cost. In this study, reinforcement methods are also examined. Therefore, the findings of this study are considered to be useful for application to actual cases.

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