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SOME TRIALS ON SEISMIC PERFORMANCE EVALUATION OF ROCKFILL DAMS

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Abstract

In Japan, seismic design of dams is carried out by the “Seismic Intensity Method”. In this method, it is possible to perform structural calculations that take into account the inertial force and dynamic water pressure due to earthquakes by applying a horizontal seismic intensity determined by the type of dam and regional classification to a dam that is assumed to be a rigid body. However, when large-scale earthquakes are targeted, there is a problem that dam response and material nonlinearity cannot be considered. Therefore, after the 1995 Hyogoken-Nanbu Earthquake, a method of modeling the dam body and dam-related facilities with FEM, calculating the response by dynamic analysis, and evaluating the seismic performance based on the result is being tested. This paper describes a case where the seismic performance of the center core-type rockfill dam was evaluated based on the results of dynamic analysis using 2D FEM models. In this study, the plastic deformation due to the failure of the slip line set on the upstream and downstream surfaces of the dam was calculated by the Newmark’s method using the response acceleration by dynamic analysis. Then, focus on the shear strength of the embankment material, and calculated the case using only peak strength and the case using peak strength and residual strength. In addition, the calculation was performed by increasing the acceleration amplitude of the input seismic motion to estimate the maximum acceleration when the amount of settlement at the top of dam is larger than the freeboard. The remarkable point of this study is that the initial shear modulus of embankment material was identified based on the natural period of the dam body obtained by microtremor observation, and the analysis accuracy was improved.

Keywords: rockfill dam, seismic performance evaluation, Newmark’s method, dynamic analysis, finite element method

1. Introduction

The seismic design of Japanese dams is traditionally carried out by the “Seismic Intensity Method”. In this method, it is possible to perform structural calculations that take into account the inertial force and dynamic water pressure due to earthquakes by applying a horizontal seismic intensity determined by the type of dam and regional classification to a dam that is assumed to be a rigid body. Thus, the effects of earthquakes can be easily considered, but there is a problem that the response of dams and the nonlinearity of materials cannot be considered when large-scale earthquakes are targeted. To solve this problem, after the 1995 Hyogoken-Nanbu Earthquake, a method of modeling the dam body and dam-related facilities with FEM, calculating the response by dynamic analysis, and evaluating the seismic performance based on the results is being tested [1]. However, there have been reports of large-scale earthquakes, such as the collapse of the Fujinuma Dam caused by the 2011 off the Pacific coast of Tohoku Earthquake, that caused the dam to lose its storage function and cause serious damage to downstream areas [2]. Therefore, there is a social demand for evaluate the safety of dams and related facilities against large-scale earthquakes, regardless of whether they are new or existing, and taking measures if the safety is not satisfactory.

This paper describes an example of evaluating the seismic performance of a center core-type rockfill dam against large-scale earthquakes. In this case, based on the response acceleration of the dam body by nonlinear dynamic analysis using a 2D FEM model, the plastic deformation of the slip line set on the upstream and downstream surfaces of the dam was calculated by the Newmark's method [3]. At this time, this study is examining the effect of the difference in shear strength of the embankment material on the amount of plastic deformation. In addition, in order to confirm the margin of the seismic performance of the dam body, calculations were performed when the acceleration amplitude of the input seismic motion was increased, and the maximum acceleration when the amount of settlement at the top of dam became larger than that of the freeboard was estimated.

2. Overview of Target Dam

The dam examined in this study is a center core-type rockfill dam with a height of 62 m and a crest length of 290 m. It is a dam for power generation about 40 years after its construction in 1980. Photo 1 shows a panoramic view of the target dam. Furthermore, Fig. 1 shows the maximum section. This dam is composed of four zones, from the center, zone 1: core, zone 2: filter, zone 3: random fill, and zone 4: rock fill. Among them, the random fill can be classified into zone 3A: fine and zone 3B: coarse. In addition, the slope gradient of the dam is 1: 2.3-2.5 on the upstream side and 1: 1.7 on the downstream side, and it is characteristic that the foundation rock inclined downstream.

Photo 1 – Panoramic photo of the target dam

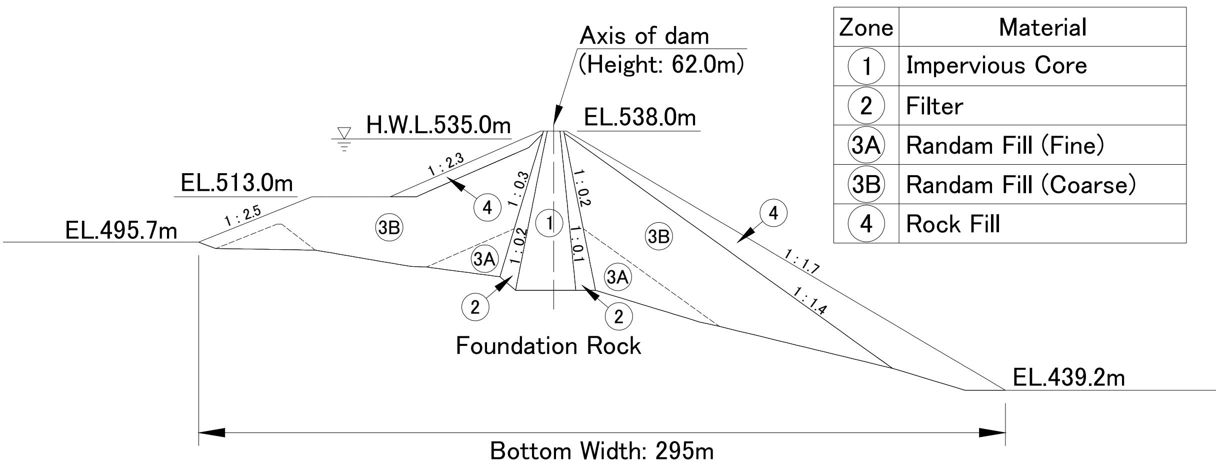

Fig. 1 – Maximam cross section

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3. Evaluation of Seismic Performance of Rockfill Dam against Large-Scale Earthquake

3.1 Overview of seismic performance evaluation

Fig. 2 shows the flow chart of the seismic performance evaluation of the dam body carried out in this study. Firstly, the seismic performance required for the dam body during a large-scale earthquake was set. Next, based on the design and construction information, the specifications of dam body were arranged and the physical properties used for analysis were set. After that, the maximum cross section of the dam body was modeled by 2D FEM, and the seismic response was calculated by nonlinear dynamic analysis in order to evaluate the seismic performance according to the guidelines. In order to improve the accuracy of the analysis, the shear modulus of the embankment material was identified based on the natural period of the dam body obtained by microtremor observation. Using the response acceleration of the levee obtained by the dynamic analysis, the plastic deformation of the slip line set on the upstream and downstream surfaces of the dam was calculated by the Newmark’s method. At this time, in this study, in order to examine the effect of the difference in the shear strength of the embankment material on the amount of plastic deformation, a case using only the peak strength and a case using the peak strength and the residual strength were calculated. In addition, as an attempt to confirm the margin of seismic performance of the dam body, nonlinear dynamic analysis and plastic deformation analysis were performed using input seismic motion with increased acceleration amplitude. Then, the maximum acceleration when the amount of settlement at the top of dam became larger than that of the freeboard was estimated. Fig. 3 shows the acceleration response spectrum and time history waveform of the input seismic motion used in this study. These are simulated seismic motions that conform to the lower-limit acceleration response spectrum for reference shown in Reference 1. The details of each study are described below.

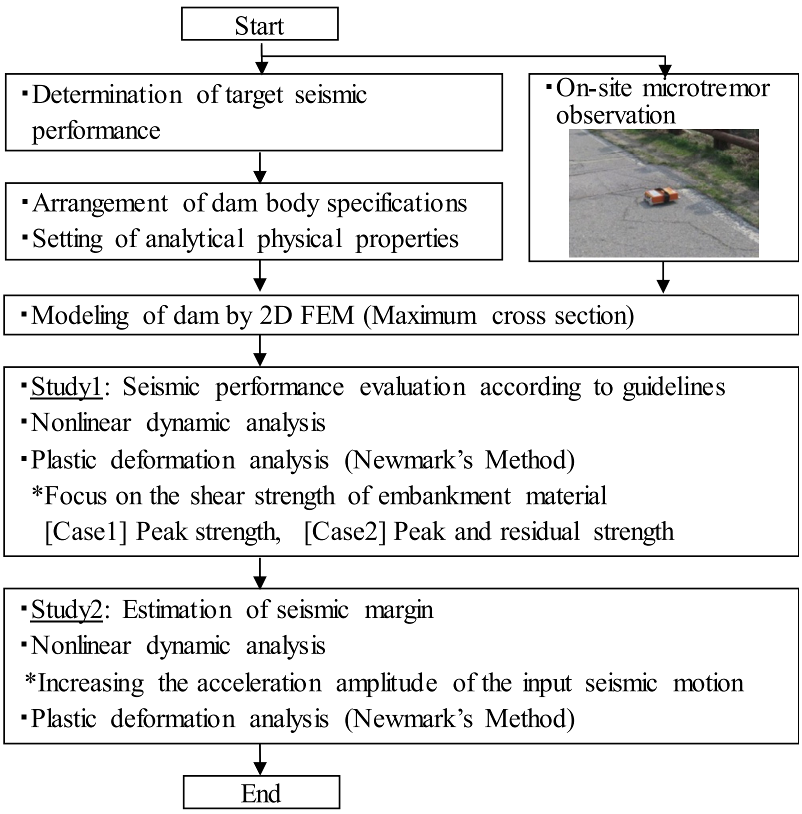

Fig. 2 – Flow chart of seismic performance evaluation in this study

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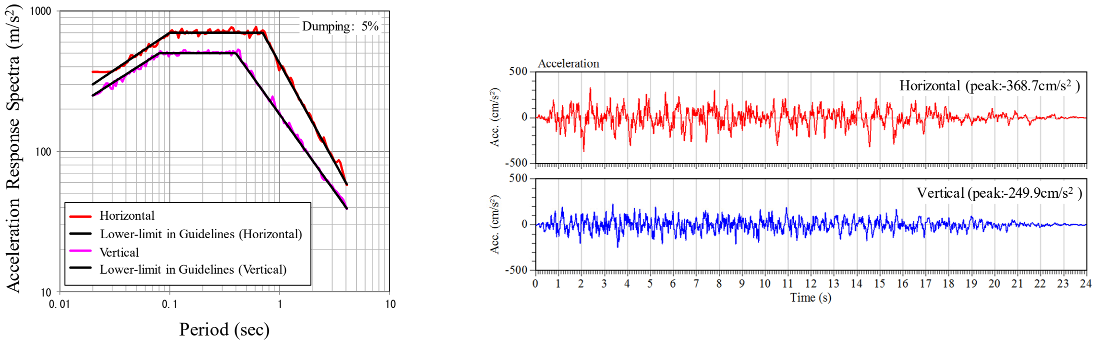


(a) Acceleration response spectrum (b) Time history waveform of Acceleration

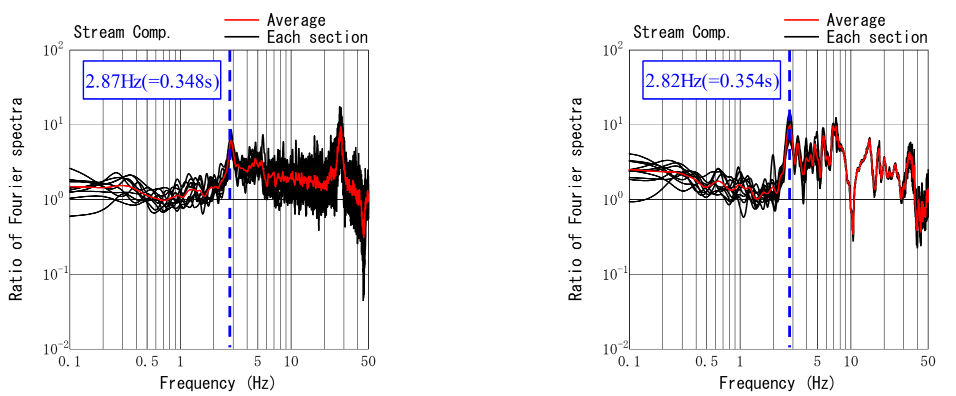
Fig. 3 – Acceleration response spectrum and time history waveform of the input seismic motion

3.2 Determination of target seismic performance

The required seismic performance of the dam body during a large-scale earthquake was set based on Reference 1. Specifically, in order to maintain the water storage function, the vertical component of the amount of plastic deformation (settlement) caused by the earthquake was decided to be smaller than 1 m of the freeboard.

3.3 Estimation of natural period of dam by microtremor observation

Microtremor observation using an accelerometer was carried out on site to determine the natural period of the dam body. The observation positions were on the top of the dam with the maximum cross section, on the bedrock of downstream and left side of the dam. The three points were observed at the same time, with 100 Hz sampling for 15 minutes. From the observation results, ten sections of about 40 seconds were selected, and the Fourier spectrum of each point and the ratio of the Fourier spectrum of the top of the dam to the downstream and left side bedrock shown in Fig. 4 (upstream and downstream directions) were calculated. As a result, it was found that the average frequency of 2.87 Hz (= 0.348 s) and 2.82 Hz (= 0.354 s) were the dominant frequencies in the average of 10 sections. Each value is almost the same, and is a value within the range of the past empirical formula (for example, Reference 4). Therefore, the average value of both, 2.85 Hz (= 0.351 s), was set as the target value in the eigenvalue analysis.



(a) Top of dam / downstream bedrock (b) Top of dam / left side bedrock

Fig. 4 – Fourier spectrum ratio (Upstream and downstream direction)

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3.4 Setting of analytical physical properties and modeling of dam by 2D FEM

In this study, the 2D FEM model for the maximum section of the dam shown in Fig. 5 was used. In this model, each zone of the embankment is modeled by a plane strain element, and is not modeled because the foundation rock is very hard. The thicknesses of the elements of the analysis model are set in consideration of the frequency of the input seismic motion and the shear wave velocity of the embankment material. Table 1 shows the analytical properties of the embankment materials. These are mainly set based on dam design and construction information and references. As the unit weight, different values were set in the saturated region and the unsaturated region obtained by the steady state seepage flow analysis at the High water level (H.W.L.). The peak strength and the residual strength were set for the shear strength in order to examine the effect on the amount of plastic deformation. The initial value of the shear wave velocity was set based on Reference 5. However, the natural period of the dam body by the eigenvalue analysis using the initial values was different from the target value by the microtremor observation described above. Therefore, in order to improve the accuracy of the analysis, the initial shear modulus was multiplied by 1.2, and after confirming that the natural periodf matched, it was used for the subsequent nonlinear dynamic analysis. In addition, a modified Ramberg-Osgood (R-O) model is applied to consider the non-linearity of the embankment material during an earthquake [6].

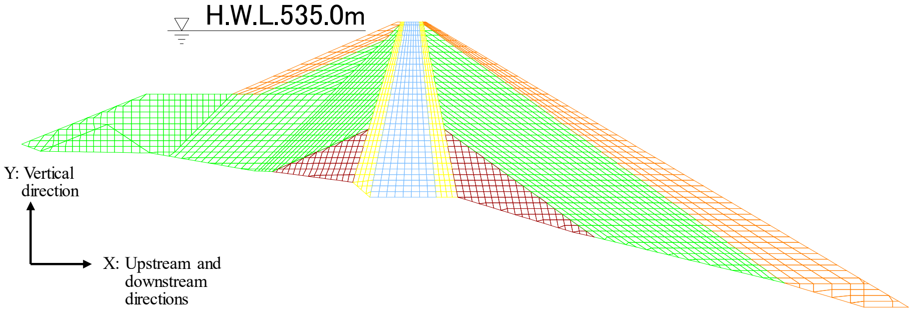

Fig. 5 – 2D FEM model used in this study

Table 1 – Physical properties of embankment materials

Parameters				Unit weight	Peak strength: Cohesion	Peak strength: Internal friction angle	Residual strength: Cohesion	Residual strength: Internal friction angle	Shear wave velocity*	Dynamic Poisson's ratio	Standard strain	Maximum damping ratio
Symbol				γ	c_p	φ_p	c_r	φ_r	V_s	ν_d	$\gamma_{0.5}$	h_{max}
Unit				(kN/m^3)	(kN/m^2)	(°)	(kN/m^2)	(°)	(m/s)	(-)	(-)	(-)
Embankment Material	Zone 1	Impervious core	Saturated	20.6	30.4	25.6	30.4	19.2	Depth:z =0-5m; 210 Depth:z=5- m; 180Z$^{0.35}$	0.35	0.00059	0.196
			Unsaturated	20.2								
	Zone 2	Filter	Saturated	21.7	0.0	33.0	0.0	24.8	Depth:z =0-5m; 245 Depth:z=5- m; 245Z$^{0.20}$	0.33	0.00030	0.236
			Unsaturated	20.3					Depth:z =0-5m; 245 Depth:z=5-30m; 245Z$^{0.20}$ Depth:z=30- m; 200Z$^{0.315}$			
	Zone 3A Zone 3B	Randam fill (Fine, Coarse)	Saturated	21.1	0.0	39.0	0.0	29.3	Depth:z =0-5m; 245 Depth:z=5- m; 245Z$^{0.20}$	0.30		
			Unsaturated	18.8					Depth:z =0-5m; 245 Depth:z=5-30m; 245Z$^{0.20}$ Depth:z=30- m; 200Z$^{0.315}$			
	Zone 4	Rock fill	Saturated	20.8	0.0	42.0	0.0	31.5	Depth:z =0-5m; 245 Depth:z=5- m; 245Z$^{0.20}$	0.30		
			Unsaturated	18.1					Depth:z =0-5m; 245 Depth:z=5-30m; 245Z$^{0.20}$ Depth:z=30- m; 200Z$^{0.315}$			

*Shear wave velocity: Set based on Reference 5

**Initial shear modulus: $G_0=(\gamma/9.81)Vs^2$

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3.5 Seismic performance evaluation according to guidelines (Study 1)

In this section, after describing the seismic performance evaluation of the dam body in accordance with the guidelines shown in Reference 1, the effect of the difference in shear strength on the amount of plastic deformation is described. Fig. 6 shows the maximum response acceleration distribution diagram of the absolute value in the horizontal direction obtained when nonlinear dynamic analysis was performed using the 2D FEM model. In the nonlinear dynamic analysis, the seismic motions in the two directions shown in Fig. 3 were simultaneously input to the bottom of the dam body. The response acceleration in the horizontal direction increases from the bottom to the top of the dam. The magnification of the response acceleration in the horizontal direction at the bottom and top of the dam is about 1.3 times. Fig. 7 shows the maximum shear strain distribution diagram. The maximum value of the shear strain is generated in zone 4 on the downstream surface, and the value is a strain of the order of 10 minus the third power, which is not a value that causes a problem in analysis.

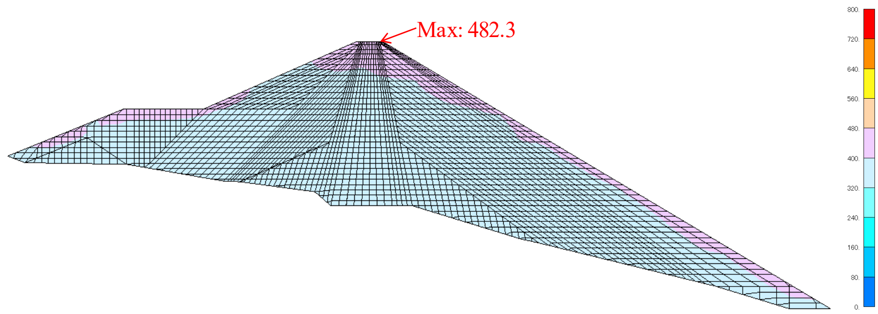

Fig. 6 – Maximum response acceleration distribution diagram of the absolute value in the horizontal direction (Unit of acceleration: cm/s^2)

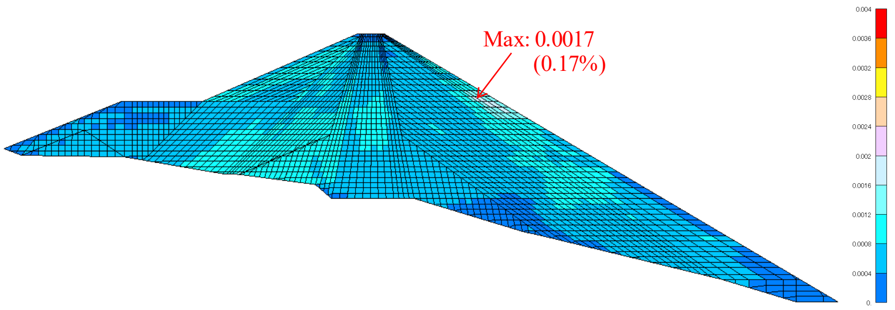

Fig. 7 – Maximum shear strain distribution diagram

Using the response acceleration obtained by the nonlinear dynamic analysis, the plastic deformation of the slip line set on the upstream and downstream surfaces of the dam was calculated by the Newmark's method. Fig. 8 shows the slip line set in this study. These slip lines are set with reference to Reference 7, and 20 slip lines with different starting points and depths are arranged on the upstream and downstream surfaces of the dam. Since the response acceleration differs depending on the position of the slip line, the time history response acceleration of the elements included in the slip line was averaged, and the time history of the equivalent seismic intensity was set. In this study, plastic deformation analysis was performed for the following two cases in order to confirm the effect of the difference in the setting method of the shear strength of the embankment material after the sliding failure had on the amount of plastic deformation. In the first case (Case 1), similarly to the guidelines, the shear strength of the embankment material was assumed to be constant regardless of the equivalent seismic intensity, and the analysis was performed using only the yield seismic intensity calculated from the peak strength. In the second case (Case 2), assuming that the shear strength of the embankment material changes with the equivalent seismic intensity, when the equivalent seismic intensity reaches the yield seismic intensity calculated from the peak strength, it is immediately reduced to the yield seismic intensity calculated from the residual strength. It is known that a material having

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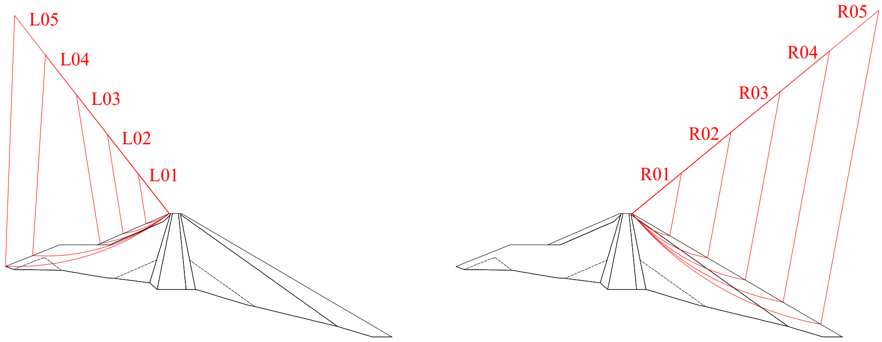
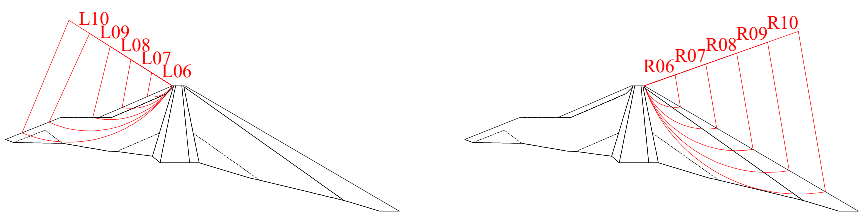
a large particle size, such as a rock material, shows a peak strength and then shows a strain softening to reach a residual strength [8]. However, it is still at the research stage to reproduce it by analysis [9]. Therefore, it should be noted that the plastic deformation calculated in the second case is larger than the actual phenomenon.



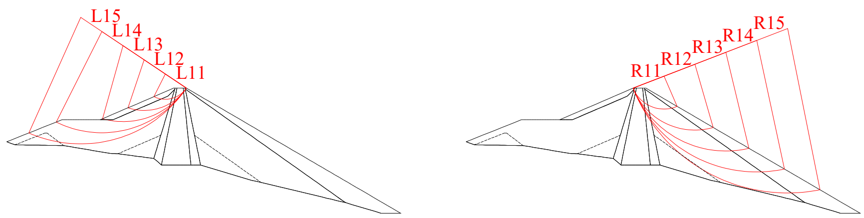
a) Upstream b) Downstream

(a) Type1: Shallow slip line starting from top end



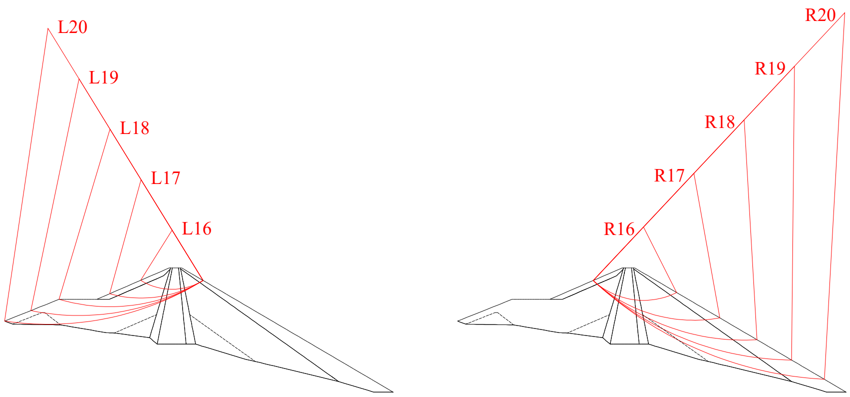
a) Upstream b) Downstream

(b) Type2: Deep slip line starting from top end (Type A)



a) Upstream b) Downstream

(c) Type3: Deep slip line starting from top end (Type B)



a) Upstream b) Downstream

(d) Type4: Deep slip line starting from opposite slope

Fig. 8 – Slip line set in this study

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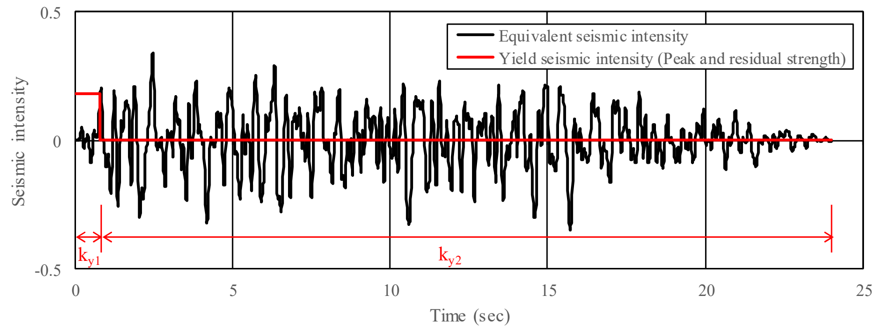
Table 2 shows the results of plastic deformation analysis by the Newmark's method. This table shows the yielding seismic intensity, the equivalent maximum seismic intensity, the deformation in the direction of the slip line, and the settlement at the top of the dam at each slip line. Deformation and settlement tend to increase at the slip line located near the top of the embankment where the response acceleration by nonlinear dynamic analysis is large. In addition, in Case 1 using peak strength, the maximum value of the settlement satisfied the allowable value of 1 m, but in Case 2 using the peak and residual strength, the maximum value of the settlement exceeded the allowable value. Therefore, when the peak strength is used in the plastic deformation analysis, the result satisfies the seismic performance required for the dam body during a large-scale earthquake. However, when the peak and residual strength are used, the result does not satisfy the seismic performance. The maximum deformation and settlement is the slip line R04 of Case 2. In this slip line, the equivalent seismic intensity reaches the yield seismic intensity calculated from the peak strength immediately after the occurrence of the earthquake according to the time history diagram shown in Fig. 9. In addition, since the yield seismic intensity calculated from the residual strength is much smaller than the equivalent seismic intensity, it is considered that the amount of deformation and settlement has increased.

Table 2 – Results of plastic deformation analysis by Newmark's method (Study 1)

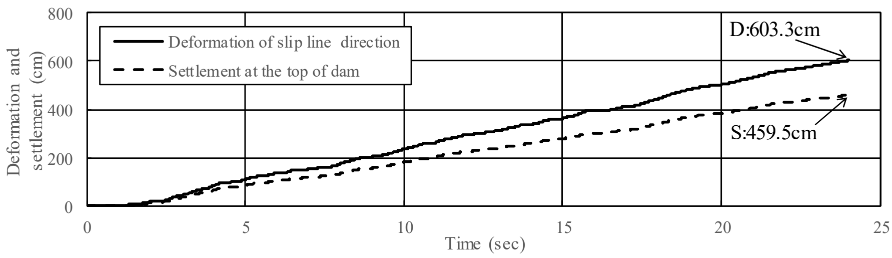
Type of slip line			Yield seismic intensity		Equivalent maximum seismic intensity	Case1 (Using peak strength)		Case2 (using peak and residualstrength)	
			Peak strength*	Residual strength*		Deformation of slip line direction	Settlement at the top of dam	Deformation of slip line direction	Settlement at the top of dam
Slope	Type	No.	k_{y1}	k_{y2}	k	D	S	D	S
			(-)	(-)	(-)	(cm)	(cm)	(cm)	(cm)
Upstream	Type1	L1	0.205	0.081	0.442	25.9	15.9	144.6	86.8
		L2	0.187	0.078	0.397	24.1	14.8	127.4	77.5
		L3	0.183	0.078	0.357	17.6	10.5	99.7	59.4
		L4	0.252	0.145	0.335	1.4	0.8	14.6	9.0
		L5	0.245	0.140	0.301	0.8	0.5	12.6	7.7
	Type2	L6	0.212	0.088	0.443	24.1	20.3	132.1	108.9
		L7	0.181	0.075	0.380	22.7	19.1	120.0	100.2
		L8	0.178	0.075	0.355	15.8	13.3	93.6	78.5
		L9	0.256	0.147	0.337	1.3	1.1	15.2	12.8
		L10	0.272	0.160	0.294	0.1	0.1	8.2	6.9
	Type3	L11	0.393	0.258	0.444	0.3	0.3	11.6	9.6
		L12	0.253	0.140	0.399	7.6	6.3	44.7	36.9
		L13	0.218	0.111	0.369	8.5	7.0	51.3	42.4
		L14	0.255	0.144	0.354	1.6	1.3	18.2	15.1
		L15	0.273	0.160	0.315	0.2	0.2	9.2	7.6
	Type4	L16**	-	-	-	-	-	-	-
		L17	0.423	0.292	0.370	0.0	0.0	0.0	0.0
		L18	0.351	0.231	0.356	0.0	0.0	3.6	1.9
		L19	0.328	0.212	0.338	0.0	0.0	3.4	1.8
		L20	0.305	0.192	0.307	0.0	0.0	3.4	1.8
Downstream	Type1	R1	0.195	0.012	0.387	16.9	13.0	459.8	341.0
		R2	0.185	0.004	0.332	9.0	7.0	548.9	413.7
		R3	0.183	0.003	0.342	5.3	4.1	588.1	446.3
		R4	0.182	0.002	0.337	3.4	2.6	603.3	459.5
		R5	0.182	0.002	0.317	1.9	1.5	600.7	458.7
	Type2	R6	0.238	0.045	0.371	6.0	5.6	200.2	185.9
		R7	0.235	0.043	0.326	2.0	1.8	158.5	148.5
		R8	0.234	0.042	0.331	1.0	1.0	131.8	123.9
		R9	0.233	0.042	0.318	0.6	0.5	115.2	108.3
		R10	0.233	0.042	0.293	0.3	0.3	98.5	92.7
	Type3	R11	0.397	0.197	0.365	0.0	0.0	0.0	0.0
		R12	0.295	0.099	0.322	0.1	0.1	53.4	49.7
		R13	0.267	0.072	0.325	0.4	0.3	70.8	65.8
		R14	0.254	0.060	0.314	0.3	0.3	74.5	69.3
		R15	0.247	0.054	0.295	0.2	0.2	72.6	67.6
	Type4	R16	0.418	0.245	0.323	0.0	0.0	0.0	0.0
		R17	0.282	0.112	0.321	0.2	0.1	35.2	24.1
		R18	0.239	0.069	0.320	0.7	0.5	69.0	47.1
		R19	0.219	0.048	0.314	0.9	0.6	95.4	65.2
		R20	0.207	0.035	0.302	0.8	0.6	116.5	79.5

*Shear strength of embankment material

**Slip line No.L16 : Excluded from calculation due to very large yield seismic intensity



(a) Equivalent seismic intensity and the yield seismic intensity



(b) Deformation of slip line direction and Settlement at the top of dam
Fig. 9 – Time history diagram (Case2, Slip lines: R4)

From the above, in the plastic deformation analysis, the method of setting the shear strength of the embankment material after the occurrence of slip failure has a large effect on the deformation and settlement, and the seismic performance may not be satisfied. Therefore, it is necessary to accumulate knowledge about strain softening from the peak strength to the residual strength.

3.6 Estimation of seismic margin (Study 2)

This section describes the attempt to confirm the margin of seismic performance of the dam body. Specifically, nonlinear acceleration analysis and plastic deformation analysis were performed by increasing the acceleration amplitude of the input seismic motion shown in Fig. 3 to estimate the maximum acceleration when the amount of settlement at the dam top became larger than the freeboard. At that time, the shear strength of the embankment material was assumed to be constant regardless of the equivalent seismic intensity, as in Case 1 described above, and analysis was performed using only the yielding seismic intensity calculated from the peak strength.

Table 3 shows the results of plastic deformation analysis by the Newmark's method when the acceleration amplitude of the input seismic motion is 1, 1.5, and 2 times. This table shows the yield seismic intensity, equivalent maximum seismic intensity, deformation of the direction of the slip line, and settlement of the dam top at each slip line. Regardless of the magnification of the acceleration amplitude, the deformation and settlement tend to increase at the slip line located near the top of the embankment where the response acceleration by nonlinear dynamic analysis is large. Also, when the acceleration amplitude of the input seismic motion was 1 and 1.5 times, the maximum value of the subsidence satisfied the allowable value of 1 m, but when it was twice, the maximum value of the subsidence exceeded the allowable value.

Therefore, it is estimated that the seismic performance can be maintained even if the acceleration amplitude of the input seismic motion set in this study is increased to about 1.5 times as the margin of seismic performance of the dam body.

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Table 3 – Results of plastic deformation analysis by Newmark's method (Study 2)

Type of slip line			Yield seismic intensity / Peak strength*	Acceleration amplitude of input seismic motion ×1.0 (Original) Equivalent maximum seismic intensity	×1.0 (Original) Deformation of slip line direction	×1.0 (Original) Settlement at the top of dam	×1.5 Equivalent maximum seismic intensity	×1.5 Deformation of slip line direction	×1.5 Settlement at the top of dam	×2.0 Equivalent maximum seismic intensity	×2.0 Deformation of slip line direction	×2.0 Settlement at the top of dam
Slope	Type	No.	k_{y1} (-)	k (-)	D (cm)	S (cm)	k (-)	D (cm)	S (cm)	k (-)	D (cm)	S (cm)
Upstream	Type1	L1	0.205	0.442	25.9	15.9	0.568	81.3	49.3	0.723	155.4	93.1
		L2	0.187	0.397	24.1	14.8	0.519	77.2	47.1	0.622	150.5	91.4
		L3	0.183	0.357	17.6	10.5	0.474	59.9	35.8	0.586	124.7	74.2
		L4	0.252	0.335	1.4	0.8	0.467	12.8	7.8	0.594	40.3	24.7
		L5	0.245	0.301	0.8	0.5	0.429	11.0	6.8	0.555	36.6	22.5
	Type2	L6	0.212	0.443	24.1	20.3	0.569	77.0	64.1	0.729	150.4	123.5
		L7	0.181	0.380	22.7	19.1	0.506	72.4	60.7	0.614	145.1	120.9
		L8	0.178	0.355	15.8	13.3	0.472	54.7	46.0	0.582	116.1	97.3
		L9	0.256	0.337	1.3	1.1	0.448	12.3	10.4	0.562	37.8	31.9
		L10	0.272	0.294	0.1	0.1	0.415	6.8	5.7	0.537	25.0	21.1
	Type3	L11	0.393	0.444	0.3	0.3	0.558	8.1	6.7	0.691	25.3	20.9
		L12	0.253	0.399	7.6	6.3	0.520	32.7	27.0	0.624	75.6	62.2
		L13	0.218	0.369	8.5	7.0	0.492	35.8	29.6	0.602	79.3	65.4
		L14	0.255	0.354	1.6	1.3	0.467	13.8	11.4	0.573	39.2	32.4
		L15	0.273	0.315	0.2	0.2	0.431	6.8	5.7	0.548	24.5	20.3
	Type4	L16**	-	-	-	-		-	-		-	-
		L17	0.423	0.370	0.0	0.0	0.488	1.2	0.6	0.597	8.3	4.3
		L18	0.351	0.356	0.0	0.0	0.473	3.2	1.7	0.581	14.6	7.7
		L19	0.328	0.338	0.0	0.0	0.454	2.7	1.4	0.566	14.0	7.3
		L20	0.305	0.307	0.0	0.0	0.424	2.6	1.3	0.540	13.9	7.3
Downstream	Type1	R1	0.195	0.387	16.9	13.0	0.481	66.1	50.7	0.542	138.1	105.3
		R2	0.185	0.332	9.0	7.0	0.449	44.8	34.5	0.541	107.5	82.6
		R3	0.183	0.342	5.3	4.1	0.460	34.3	26.4	0.556	89.0	68.5
		R4	0.182	0.337	3.4	2.6	0.448	27.0	20.8	0.545	77.7	59.8
		R5	0.182	0.317	1.9	1.5	0.427	19.4	14.9	0.529	63.8	49.2
	Type2	R6	0.238	0.371	6.0	5.6	0.464	37.2	34.9	0.528	88.8	83.1
		R7	0.235	0.326	2.0	1.8	0.448	18.5	17.5	0.547	57.1	53.7
		R8	0.234	0.331	1.0	1.0	0.449	10.7	10.1	0.553	41.2	38.8
		R9	0.233	0.318	0.6	0.5	0.428	7.0	6.6	0.528	30.7	28.9
		R10	0.233	0.293	0.3	0.3	0.401	3.9	3.7	0.502	20.9	19.7
	Type3	R11	0.397	0.365	0.0	0.0	0.467	1.2	1.1	0.551	12.5	11.6
		R12	0.295	0.322	0.1	0.1	0.445	5.7	5.3	0.546	25.8	24.0
		R13	0.267	0.325	0.4	0.3	0.443	5.2	4.8	0.547	24.9	23.2
		R14	0.254	0.314	0.3	0.3	0.425	4.0	3.7	0.525	20.4	19.0
		R15	0.247	0.295	0.2	0.2	0.403	2.8	2.6	0.502	15.6	14.5
	Type4	R16	0.418	0.323	0.0	0.0	0.448	0.1	0.1	0.558	3.8	2.6
		R17	0.282	0.321	0.2	0.1	0.447	5.1	3.5	0.554	23.8	16.3
		R18	0.239	0.320	0.7	0.5	0.435	8.1	5.5	0.537	32.9	22.5
		R19	0.219	0.314	0.9	0.6	0.426	8.8	6.0	0.523	35.3	24.1
		R20	0.207	0.302	0.8	0.6	0.412	8.5	5.8	0.510	34.3	23.4

*Shear strength of embankment material
**Slip line No.L16 : Excluded from calculation due to very large yield seismic intensity

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4. Conclusions

In this paper, the nonlinear dynamic analysis using the 2D FEM model and the plastic deformation analysis using the Newmark's method were performed on the center core-type rockfill dam, and the seismic performance against large-scale earthquakes was evaluated. The findings obtained in this paper are summarized below.

Microtremor observation is an effective method to determine the natural period of a dam body. Also, identifying the shear modulus of the embankment material by eigenvalue analysis based on the results will lead to improvement of the analysis accuracy.

Similar to the guidelines, when the peak strength was used as the shear strength of the embankment material in the plastic deformation analysis, the amount of settlement at the top of the dam was less than the allowable value. Therefore, the dam in this study satisfied the seismic performance required for a large-scale earthquake. However, in the plastic deformation analysis, the method of setting the shear strength of the embankment material after the occurrence of slip failure has a large effect on deformation and settlement, and the seismic performance may not be satisfied. Therefore, it is necessary to accumulate knowledge about strain softening from the peak strength to the residual strength.

In order to confirm the margin of the seismic performance of the dam body, calculations were performed when the acceleration amplitude of the input seismic motion was increased, and the maximum acceleration when the settlement amount at the dam top became larger than the freeboard was estimated. As a result, it is estimated that the dam of this study can maintain the seismic performance even if the acceleration amplitude of the input seismic motion increases to about 1.5 times.

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