Study of seismic performance evaluation method for concrete gravity dams on low stiffness foundation

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ABSTRACT

For evaluation of seismic performance of concrete gravity dams which are constructed on low-stiffness foundation, we should consider the permanent settlement of foundation due to earthquake loading in addition to stress of dam body and foundation. However the Guidelines for Seismic Performance Evaluation of Dams against Large Earthquakes (draft) in Japan don't mention how to estimate the permanent settlement of foundation. So, we studied the evaluation method, which is using FEM dynamic analysis by equivalent linearizing method, modeling the foundation as non-linear material and calculating the settlement by cumulative damage. Then we conducted the centrifugal loading vibration test and numerical analysis to estimate that the method we studied was reliable enough to be used for the first stage evaluation of seismic performance of concrete gravity dams on low-stiffness foundation.

1. INTRODUCTION

In March 2005, the Ministry of Land, Infrastructure, Transport and Tourism put into effect the guidelines for Seismic Performance Evaluation of Dams against Large Earthquakes (draft) (The Ministry of Land, Infrastructure, Transport and Tourism 2005). The draft guidelines stipulate that the dam should get storage under control even when the dam is damaged during a large earthquake as seismic performance requirement for dams.

When evaluating seismic performance of concrete gravity dams, the possibility of damage to the dam body is checked through dynamic analysis, and, if the damage is expected, it is verified that the damage is incurred only in a limited area.

The evaluation method described in the draft guidelines is applied to concrete gravity dams located on high-stiffness foundation. Some of dams which were constructed in old days with relatively low height above low-stiffness foundation, including weirs that are generally less than 15m in Japan, were located on low-stiffness foundation such as riverbed gravel and pyroclastic flow deposit. In evaluation of those dams, it was considered necessary to check not only about the damage of dam body but also about the foundation, whether the softening or failure of the foundation was likely to cause the settlement or collapse of the dam body or not. So, we conducted the centrifuge loading vibration test and numerical analysis to evaluate the applicability of the method. Then, we studied the evaluation method for concrete gravity dams located on low-stiffness foundation, which is based on the evaluation method for rockfill dams and earth dams shown in the draft guidelines, considers slip failure, accompanying displacement, or displacement due to post-shaking deterioration in stiffness.

2. CENTRIFUGE LOADING VIBRATION TEST

Centrifuge loading vibration tests were conducted for concrete gravity dams located on low-stiffness foundation in order to grasp the behavior of the foundation during a large earthquake and to verify analysis methods.

2.1 Test apparatus

The apparatus for centrifuge loading vibration tests is composed of a rotating device that generates centrifugal gravity and a centrifugal shaking table that generates seismic motions. The maximum radius of the rotating device is 7.0 m, the maximum loading capacity of the centrifugal shaking table is 6,860 kN, the loading area is 2.2 m long and 1.07 m wide, and the maximum exciting force is 1,176 kN.

The soil layer placed on the table is internally 2.0 m in length, 0.7 m in width and 0.65 m in height. Acrylic side walls make it possible to monitor the condition of the model during the tests.

2.2 Test model

The scale of the model was set at 1/30 based on the law of similarity effective in the centrifugal gravity field in order to apply a centrifugal force of 30 G fit for seismic motion of a large earthquake. The actual dimensions of the model dam were 15 m in dam height, 1.8 m in crest length, 1:0.6 in downstream slope and the upstream slope was assumed to be vertical. The dimensions of the 1/30-scale model are shown in Figure 1.

The height of the foundation was set at 30 cm based on the size of the soil tank used and on a height of the model of 80 cm (the height from the bottom of the foundation to the crest) that was determined by the performance of the test apparatus. Embedment for a depth of 2 cm was applied in the foundation right below the bedding to ensure the integrity between the dam body and foundation.

A reservoir water depth of 30 cm was secured the upstream of the dam to provide an allowance of 5 cm to the height of soil tank. Downstream water depth of 1 cm was secured to keep the saturated state in the foundation even in case of foundation deformation or heaving. It was considered difficult to totally prevent downstream leakage during the tests, so a leakage collection pit was installed at the downstream end of the soil tank.

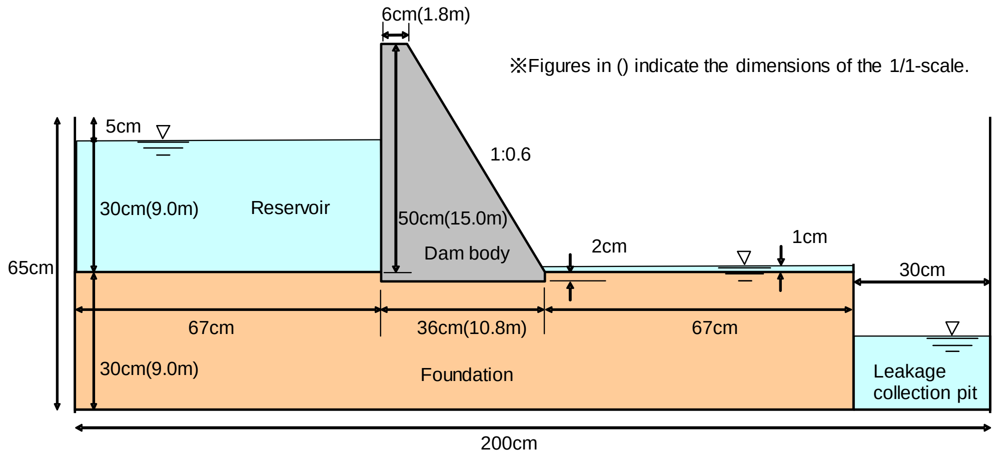

Figure 1. The dimensions of the 1/30-scale model

2.3 Material properties of the model

The main objective of the tests was to grasp the behavior of the foundation during a large earthquake. It was considered adequate for the dam body to have much higher strength and stiffness than the foundation. The target compressive strength of the mortar was therefore set at 20 N/mm^2 or greater.

In actual dams, the density of concrete is generally approximately 2.3 g/cm^3. In the tests, the target density was set at 2.0 g/cm^3 because the model was developed using mortar.

The parameters for the dam body and the foundation were set based on the results of geological surveys of actual dams located on low-stiffness foundation (Table 1).

Table 1. The parameters for the dam body and the foundation

Zone	Parameter	Target
Dam body	Density[g/cm^3]	Approximately 2.0
	Compressive strength[N/mm^2]	Approximately 20
foundation	Density[g/cm^3]	Approximately 2.0
	Internal friction angle[degrees]	28~38
	Cohesion[kN/m^2]	50~150
	Modulus of deformation(E_{50})[MN/m^2]	20~40
	Permeability[cm/sec]	1.0*10^{-7} orders

2.4 Conditions for creating the model

The conditions for developing the model were specified by conducting preliminary material tests so that the material properties shown in Table 1 could be obtained.

For the dam body, ordinary Portland cement was mixed with silica sand. Water-cement ratio was set at 60%. For the foundation, silica sand, kaolin clay and cement were mixed with one another and adjustments were made to achieve a water content of 20%.

2.5 Allocation of measuring instruments

Measured were acceleration, displacement, soil pressure and pore-water pressure. How the instruments were allocated is shown in Figure 2.

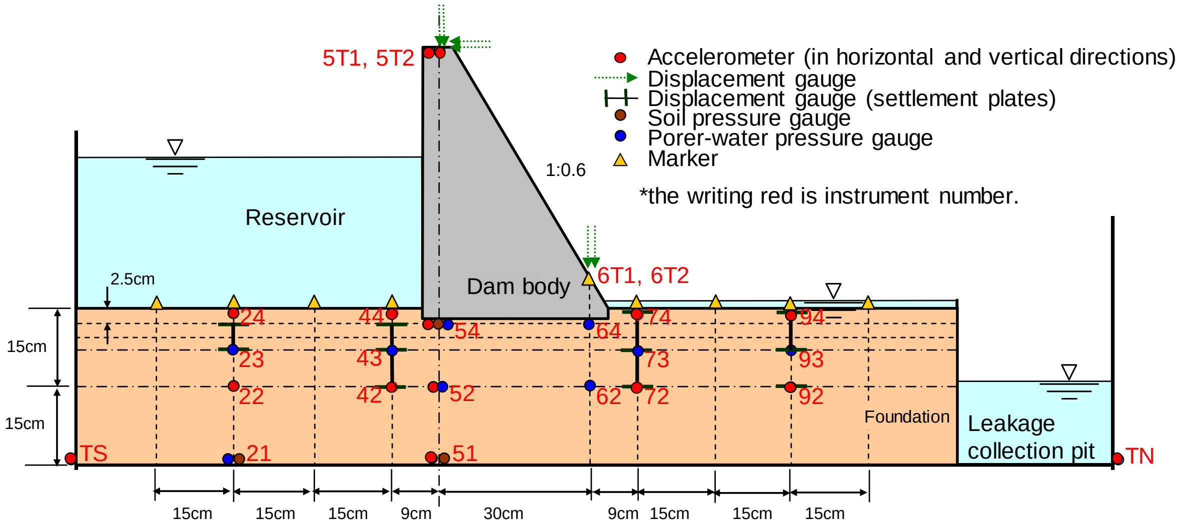

Figure 2. The allocation of measuring instruments

2.6 Centrifugal force and excitation

Centrifugal force was increased to a designated force of 30 G in 30 minutes and kept at 30 G for another 30 minutes. Following white noise excitation in the first stage, the maximum acceleration was increased to 1 m/s^2 (step 1) and 2 m/s^2 (step 2) in stages.

Seismic motions were applied in each stage using the waveform obtained by adjusting the maximum acceleration of the lower limit spectral waveform in compliance with the guidelines (maximum acceleration: 3 m/s^2). An example is shown in Figure 3, an acceleration waveform of seismic motion where a maximum acceleration of 2 m/s^2 was obtained after adjustment.

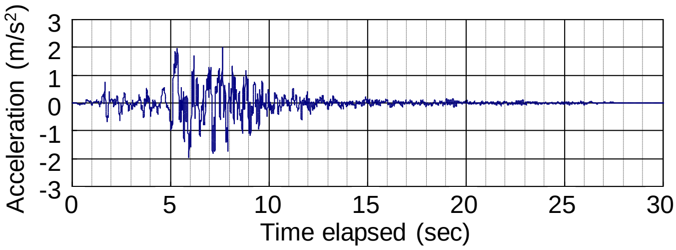

Figure 3. An acceleration waveform for step 2

2.7 Test results

2.7.1 Acceleration and transfer property

The maximum accelerations at the foundation and crest of the model in the tests are shown in Table 2. Acceleration waveforms and response spectrum ratios are shown in Figure 4.

As the maximum acceleration of input motion increases, the acceleration response magnification at the crest tends to decrease. This may be because the model exhibits nonlinear behavior with the increase of acceleration. This is also evident from the transfer functions.

The acceleration response magnification at the crest in the vertical direction (Z) and along the dam axis (Y) is less than 1.0. No reasons have yet been known. The water cutoff membranes or caulking materials may have an effect.

As for the transfer function, crest/foundation spectrum ratio, of upstream to downstream direction (X) at the crest, a primary peak is observed near 3.4 Hz.

Table 2. The maximum accelerations of the model in centrifuge tests

Excitation number	Direction	Maximum acceleration[m/s^2]		Acceleration response magnification (crest/foundation)
		Foundation (51)	Crest (5T1)	
Step 1	X	0.71	2.14	3.0
	Y	0.82	1.04	1.3
	Z	0.72	0.60	0.8
Step 2	X	2.47	6.19	2.5
	Y	4.24	3.20	0.8
	Z	3.90	1.78	0.5

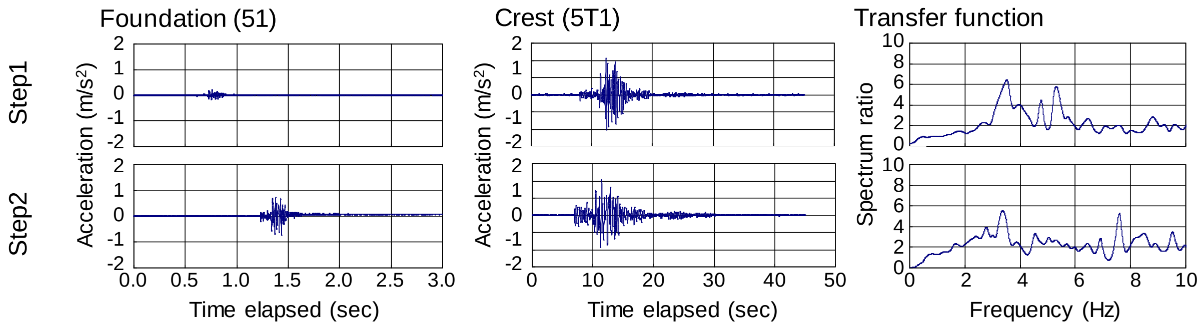

Figure 4. Acceleration waveforms and response spectrum ratios along X

2.7.2 Residual settlement

In step 1, no residual displacement occurred either vertically or horizontally. Elastic behavior was exhibited. In step 2, residual settlement of 12.0 mm was observed at the crest, and residual displacement of 27.3 mm downstream.

2.7.3 Pore-water pressure

With the increase of the maximum acceleration, excess pore-water pressure tends to increase.

3. ANALYSIS OF REPRODUCTION IN CENTRIFUGE LOADING VIBRATION TEST

Dynamic analysis was made using the equivalent linearization method (ELM) in order to obtain stress values required for evaluating the softening of the foundation and shear failure.

3.1 Analysis conditions

In centrifuge loading vibration test, accelerations were measured not only in upstream to downstream direction but also vertically and along the dam axis. For analysis, therefore, a three-dimensional FEM model was used (Figure 5).

As constraints for displacement, the bottom of the foundation was fixed horizontally along the dam axis and vertically. On the sides of the foundation, planes at the upstream and downstream ends were fixed along the X axis, and plane at the end along the dam axis was fixed along the Y axis.

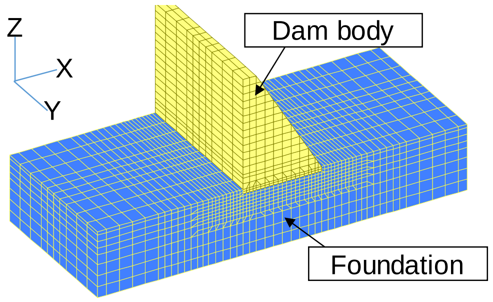

Figure 5. The model for 3D FEM anarysis

3.2 Input motions

As the input motions, the acceleration time histories in the foundation measured in the centrifuge loading vibration tests were used (Figure 6).

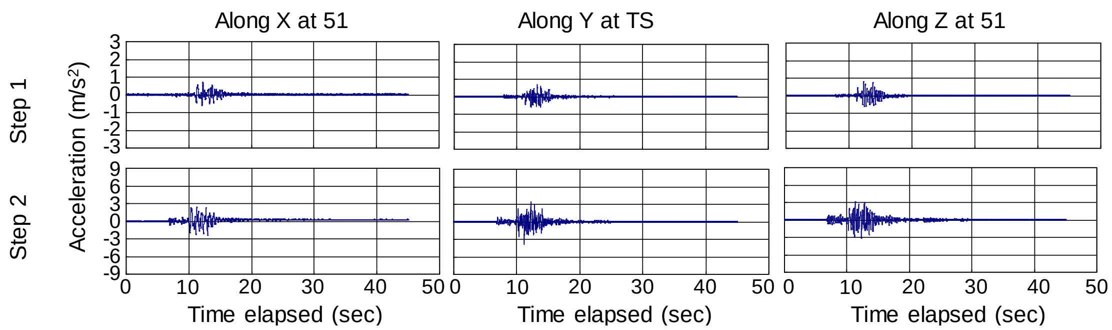

Figure 6. The acceleration time histories

3.3 Water levels

Hydrodynamic pressure was taken into consideration using the additional mass obtained by Westergaard's approximate formula.

3.4 Input physical property values

In the dynamic analysis, the dam body was regarded linear elastic and the foundation was simulated by a nonlinear model based on the Hardin-Drnevich model (Hardin, B. O. & Drnevich, V. P. 1972). The physical property values used in analysis are shown in Table 3. They were determined based on the results of preliminary material tests. The dissipation damping for the dam body was set at 10% and a combined total of internal damping and dissipation damping at 15%. The damping ratio and basic strain of the foundation were set by making analysis using trial-and-error methods so that the accelerations measured in the centrifuge tests could be reproduced.

3.5 Results of reproduction analysis in step 1

As a result of centrifuge tests, it was found that the foundation exhibited elastic behavior in step 1. The foundation was regarded to be linear elastic in dynamic analysis.

The maximum accelerations measured in centrifuge tests and analyzed are shown in Table 4. The acceleration waveforms are shown in Figure 7. There are deviations in maximum acceleration at the dam crest. This study focused on the deformation of the foundation after the earthquake. As long as the acceleration of the foundation was generally reproduced, little attention was paid to the reproducibility of the acceleration at the dam body.

3.6 Results of reproduction analysis in step 2

In centrifuge tests, nonlinear behavior was exhibited in step 2. Dynamic analysis was therefore made in the case where the foundation was simulated by a nonlinear model based on the H-D model.

Table 3. The physical property values for dynamic analysis

Zone	Parameter	Physical property value	Based on
Foundation (H-D model)	Density [g/cm^3]	2.01	Initial wet density of the specimen in triaxial compression test and dynamic test
	Initial shear stiffness ratio [MN/m^2]	$14.3\sigma_m^{0.45}$	Dynamic deformation test
	Basic strain	$1.31\sigma_m^{0.57}\times 10^{-4}$	Ten times the value obtained in dynamic deformation test
	Maximum hysteretic damping [%]	$19.2\sigma_m^{0.05}$	Reproduction in step 1 of excitation by seismic waves
	Dissipation damping [%]	10	
	Poisson's ratio	0.45	Ultrasonic test
	Internal friction angle [degrees]	30	Triaxial compression test (effective stress:φ')
	Cohesion [kN/m^2]	69.6	Triaxial compression test
	Tensile strength [kN/m^2]	63.2	Tensile strength test
Dam body (linear elastic)	Density [g/cm^3]	2.02	Elastostatic and Poisson's ratio tests, and density of specimen at the age of 21 days in tensile strength test
	Shear modulus [GN/m^2]	7.91	Ultrasonic test
	Damping ratio [%]	15	Reproduction in step 1 of excitation by seismic waves
	Poisson's ratio	0.21	Ultrasonic test
	Compressive strength [MN/m^2]	32.6	Hydrostatic physical property value incremented by 30%
	Tensile strength [MN/m^2]	2.87	

Table 4. The maximum accelerations at typical points in step 1

Direction	Instrument number	(i)Centrifuge test result[m/s^2]	(ii)Analysis result[m/s^2]	(iii) (i)/(ii)
X	24	0.92	0.85	1.08
	54	1.80	1.19	1.51
	5T1	2.14	3.53	0.61
	94	0.89	0.90	0.99
Y	5T1	0.60	0.77	0.78
Z	24	0.63	0.83	0.76
	54*	0.55	1.31	0.42
	5T1	1.04	3.53	0.29
	94	0.49	0.97	0.51
∗ Not properly measured				

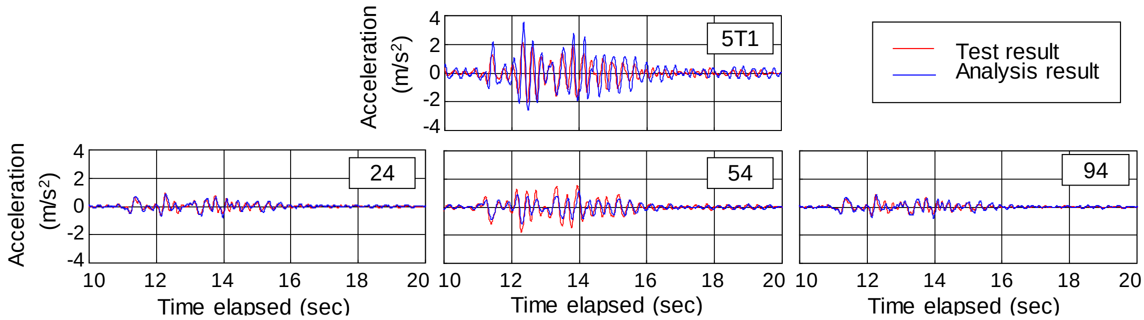

Figure 7. The acceleration waveforms along X at typical points in step 1

The basic strain of the foundation was determined using a trial-and-error method in order to reproduce the behavior during centrifuge tests. The basic strain was ten times as large as the value obtained in dynamic deformation tests.

The maximum accelerations measured in centrifuge tests and analyzed are shown in Table 5. The typical acceleration waveforms are shown in Figure 8. There are deviations between analysis and test results at the dam crest. It was, however, considered that the behavior was generally reproduced.

Table 5. The maximum accelerations at typical points in step 2

Direction	Instrument number	(i)Centrifuge test result[m/s^2]	(ii)Analysis result[m/s^2]	(iii) (i)/(ii)
X	24	4.21	3.26	1.29
	54	6.04	5.84	1.03
	5T1	6.19	10.75	0.58
	94	3.50	3.59	0.97
Y	5T1	1.76	2.95	0.60
Z	24	3.34	3.77	0.89
	54*	1.33	5.84	0.23
	5T1	3.20	6.03	0.53
	94	3.01	4.57	0.66
*Not properly measured				

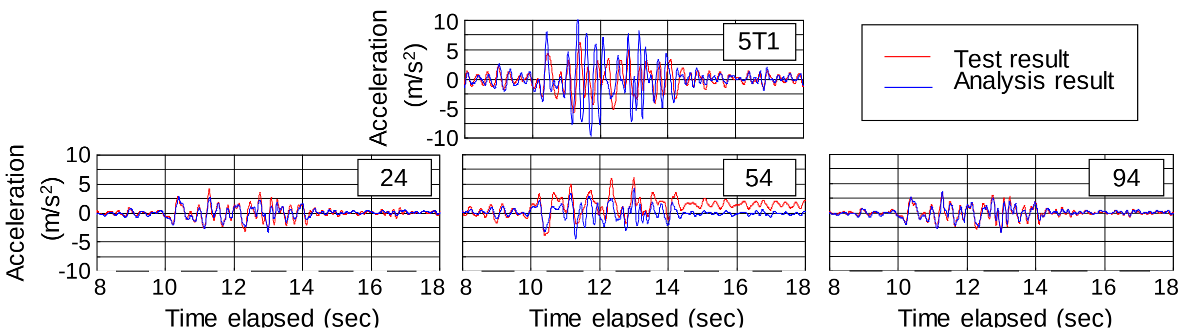

Figure 8. The typical acceleration waveforms along X at typical points in step 2

4. EVALUATION OF APPLICABILITY OF METHODS FOR ASSESSING THE SOFTENING AND FAILURE OF THE FOUNDATION

4.1 Cross section examined

Displacements at typical position (54) of the model identified in the dynamic analysis are shown in Figure 9. The figure shows that deformation along the dam axis is approximately 0. It was thus assumed that the state of 2D plane-strain was approximated. The applicability of methods for assessing the softening and failure of the foundation was therefore examined in 2D analysis.

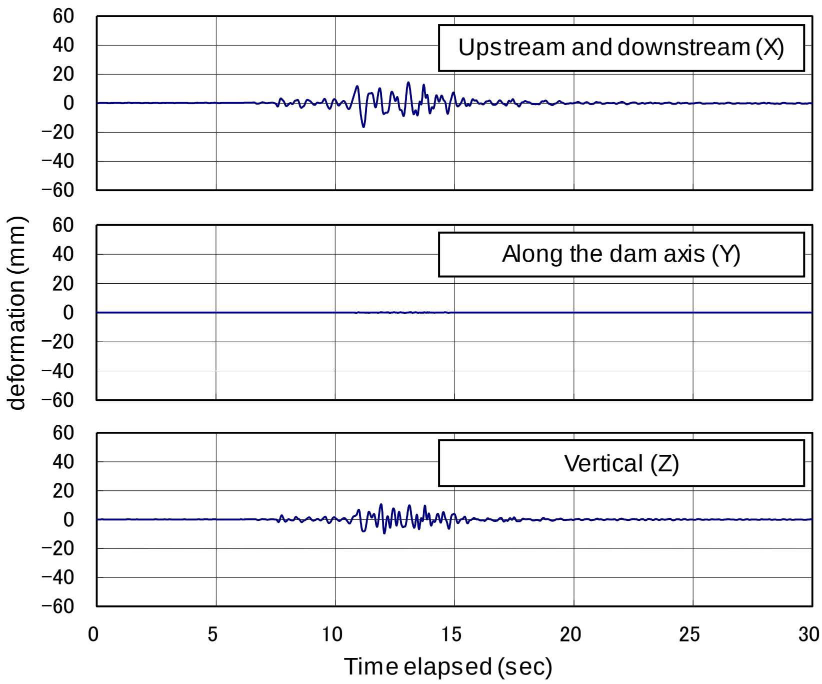

Figure 9. Displacements at typical position (54) in the dynamic analysis

4.2 Evaluation methods

The following two methods were examined as the methods for assessing the softening and failure of the foundation.

4.2.1 Evaluation of local shear safety factor

It was assumed that the area with a local shear safety factor of 1.0 or less was related to the location where residual strain occurred. Local shear safety factor was evaluated based on the Mohr-Coulomb's failure criterion.

4.2.2 Evaluation of residual deformation

Evaluation was made by calculating the plastic deformation in accordance with the accumulated damage index theory, which is used to judge soil requefaction and estimate residual settlement in a strong earthquake (Railway Technical Research Institute 1999).

4.2.3 Evaluation in terms of local shear safety factor

The distributions of local shear safety factors in the case where the maximum displacement occurred upstream and downstream at the crest in step 2 are shown in Figure 10. The left part shows that local shear safety factor was less than 1.0 in the foundation near both ends of the bedding in the case where displacement was largest upstream at the crest. The downstream area with a safety factor of less than 1.0 was wider than the upstream area. This may be because upstream displacement of the dam body reduced overburden pressure of the dam body in the foundation near the downstream end of the bedding and also deteriorated shear strength.

The right part shows that safety factor was less than 1.0 in a wide area from the upstream end of the bedding to the bottom of the foundation when the downstream displacement at the crest was largest. No area with a safety factor of less than 1.0 was found near the downstream end of the bedding. Safety factor was less than 1.0 from the upstream end of the bedding to the bottom of the foundation probably because a phenomenon occurred opposite to that when the crest was displaced upstream.

As described above, an area with a local shear safety factor of less than 1.0 occurred in step 2. It is, however, assumed that no slip or other types of failure occurs immediately after the safety factor lowered below 1.0 locally but that failure occurs as the area with a safety factor of 1.0 expanded beyond a certain range. Thus, the distribution of local shear safety factors is considered to provide a means of identifying the locations where large residual deformation or slip failure is highly likely to occur. In centrifuge loading vibration tests, no shear failure was observed at locations where local shear safety factor was less than 1.0 in analysis. The acceleration at the dam crest was calculated higher in analysis than in centrifuge tests. That means evaluation was made on the safe side.

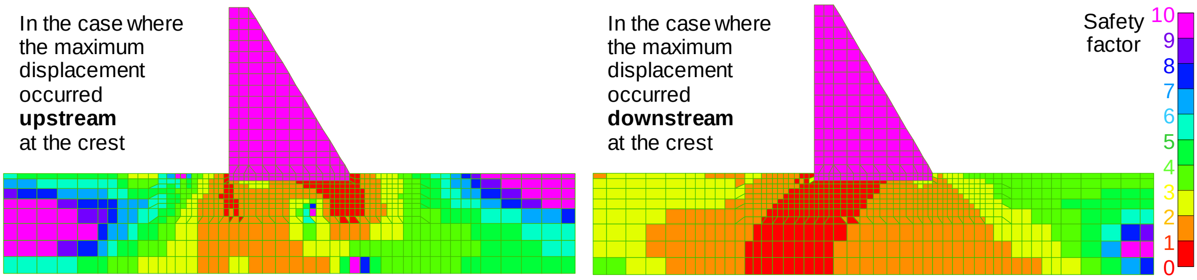

Figure 10. The distributions of local shear safety factors in step 2

4.2.4 Evaluation of residual deformation

For residual deformation, self-weight settlement analysis was made using the accumulated damage index method. We made a comparison between analysis and test results, and evaluated the applicability to checking.

In order to obtain residual deformation using the accumulated damage index method, it was necessary to formulate the relationship among strain (ε), frequency of repetitions (N) and repeated shear stress ratio (S_{rd}) (hereinafter referred to as accumulated deformation property). The accumulated deformation property was determined in accumulated deformation property tests (including liquefaction tests). The result is expressed by (1) and shown in Figure 11.

$$\varepsilon_a = 2.33 S_r^{3.57} N^{0.695} \tag{1}$$

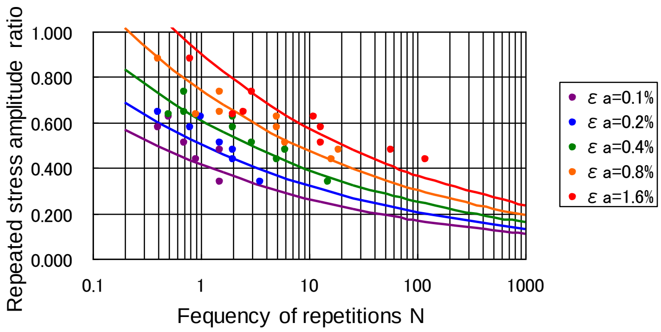

Figure 11. Accumulated deformation property

In the self-weight settlement analysis, hydrostatic pressure corresponding to reservoir water level was applied to the upper face of the dam body.

The vertical and horizontal displacements at the crest in centrifuge tests and analysis are shown in Table 6. In step 2, analysis results were 2.5 to three times higher than test results both in vertical and horizontal displacements.

The analysis method estimated the residual deformation at the crest nearly three times as large as the actual value, but reproduced the tendency of deformation to some extent. Deviations occurred in residual displacement at the crest by an approximate factor of three for two reasons.

(a) In analysis, it was assumed that the foundation and dam body were fully integrated with each other. In centrifuge tests, however, no complete integration was realized.

(b) There are limitations in grasping the actual properties because stress conditions in the actual foundation cannot be reproduced in laboratory material tests and for other reasons.

Point (a) above suggests that slight discontinuity between the dam body and foundation or local failure of the foundation causes stress release between the dam body and foundation. In the dynamic analysis, however, an equivalent linearization method was used. Deformation was therefore larger in analysis because stress was transferred to the foundation due to the response at the time of excitation of the dam body even where the local shear safety factor of the ground was less than 1.0.

The stress occurring in the foundation was considered on the safe side. If the analysis method used in this study was found to secure the safety of the dam, the results was considered effective.

Table 6. The vertical and horizontal displacements at the crest in step 2

Direction	(i)Centrifuge test result[mm]	(ii)Analysis result[mm]	(iii) (ii)/(i)
Z	-12.0	-39.2	3.27
X	27.3	70.5	2.58

5. CONCLUSIONS

Methods for evaluating the seismic performance of concrete gravity dams located on low-stiffness foundation were examined.

In concrete gravity dams on low-stiffness foundation, a risk was expected of downstream reservoir outflow when the settlement of dam crest exceeded the freeboard of the dam owing to the deformation of the foundation during an earthquake. It was therefore considered necessary to evaluate seismic performance of the foundation in addition to the stress check of the dam body.

As a method for evaluating the seismic performance of the foundation, examination was made of the applicability of a method for obtaining shear stress time history waveforms during an earthquake by making dynamic analysis using the equivalent linearization method, and for calculating the settlement of the ground by employing the accumulated damage index method. As a result of comparison between the results of centrifuge loading vibration test and of reproduction analysis, it was shown that the method overestimates settlement but is applicable on the safe side as a primary evaluation.

6. REFERENCES

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