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SAFETY OF DAMS DURING HYOGOKEN-NAMBU EARTHQUAKE ON JANUARY 17, 1995 IN JAPAN

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ABSTRACT

Immediately after the Hyogoken-Nambu Earthquake, engineers of site offices conducted special safety inspections of dams over a wide area. The Public Works Research Institute (PWRI) performed on-site inspections to investigate the safety of the dams. These inspections by site offices and detailed investigations by PWRI showed that there was no major damage to dams affecting the dam safety or requiring immediate protective countermeasures.
Many acceleration records were obtained at dam foundations during the earthquake and the earthquake resistance of dams was assessed using numerical model analysis for both concrete dams and embankment dams.

KEYWORDS

Hyogoken-Nambu Earthquake, concrete dam, embankment dam, site investigation, acceleration records, safety of dams

INTRODUCTION

About 50 dams exist within 50 km from the epicenter of the Hyogoken-Nambu Earthquake, which were built on rock foundations except some old small dams. Some dams in Kobe city are very old, and the Gohonmatsu Dam built in 1900 is the first concrete gravity dam (rubble masonry structure) in Japan. The nearest dam is located as close as 800 m to the Nojima earthquake fault. Dams near the epicenter were shaken strongly during the earthquake. Site offices within about 200 km from the epicenter conducted safety inspections of dams immediately after the main shock, and specialists of the PWRI investigated the safety of dams in the region. This paper presents the findings of these inspections. In this paper, dams are defined as structures with a height of more than 15 m.

SPECIAL SAFETY INSPECTION BY SITE OFFICES

Special safety inspections of dams were carried out just after the earthquake by site offices under the jurisdiction of the Ministry of Construction (MOC) within the river reaches administered under the River Law. The special safety inspection consisted of primary and secondary inspections. The former was a visual inspection of the condition of the dam immediately after the earthquake, while the latter included both a detailed visual inspection of the dam and safety checks of the data recorded by instruments. Special safety inspections were conducted at a total of 251 dams. Figure 1 shows the location of inspected dams.

The special safety inspections (primary and secondary inspections) of 251 dams were completed by January 21, 1995, and no damage requiring emergency protective countermeasures was reported. However, some slight damage to dams, such as minor cracks in the pavement on the crest, was reported. The amount of drainage water increased slightly after the earthquake at nine dams, but the total amount was small and stabilized later.

ON-SITE INSPECTION OF DAMS BY PWRI

Summary of Inspection

The Dam Department of PWRI dispatched teams to conduct detailed safety inspections of dams, categorized into two groups: dams where slight damage was reported in the special safety inspection by site offices, and dams located in areas where a seismic event of intensity 5 or greater was reported. Figure 2 shows the locations of the 28 dams that were investigated, and Table 1 lists the investigated dams within 50 km from the epicenter and summarizes the inspection results.

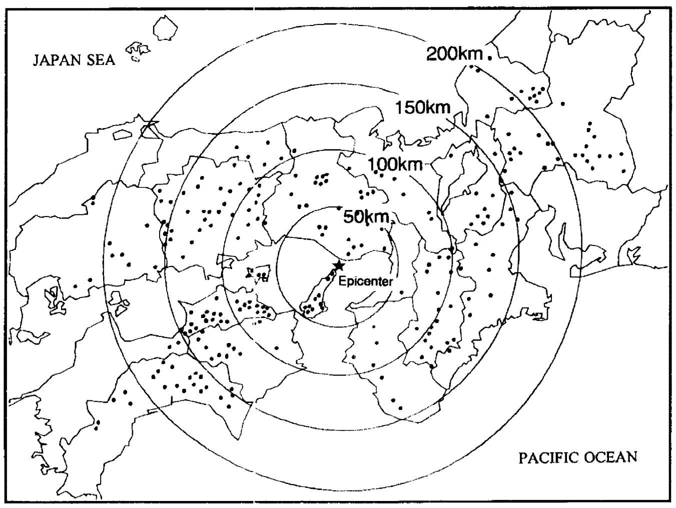


Figure 1 Location of dams inspected by site offices

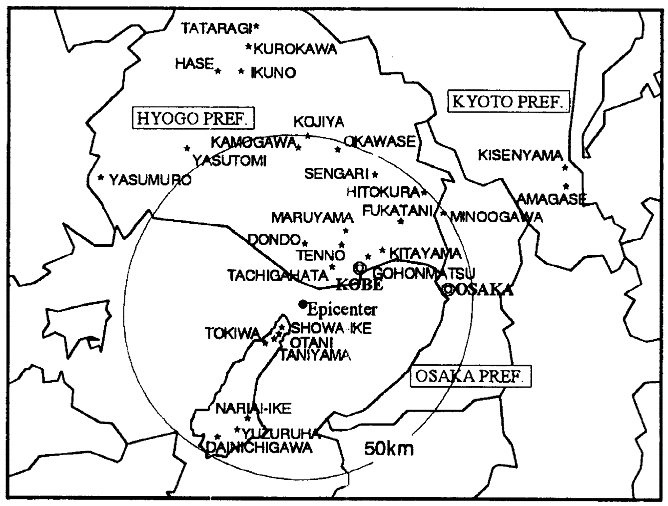


Figure 2 Location of dams investigated by PWRI

Details of Inspection

Yuzuruha Dam - The Yuzuruha Dam is a 42 m-high concrete gravity dam, located approximately 43 km from the epicenter. Figure 3 shows the cross section of the dam. The special safety inspection by the site office reported slight spalling of the concrete on the upstream face, but the PWRI investigation found that some mortar placed for a surface finish had peeled off, and no damage of the dam body was observed (Photo. 1). Water from the foundation drainage holes became slightly turbid after the earthquake, but the turbidity disappeared after several days. The amount of drainage water and uplift pressure both increased slightly after the earthquake, but stabilized later.

Tokiwa Dam - The Tokiwa Dam is a 33.5 m-high zoned earthfill dam, located about 10 km from the epicenter. The dam is as close as 800 m to the Nojima fault, which moved during the earthquake. Figure 4 shows the cross section of the dam. 25 mm wide and 7 m long transverse cracks were observed in the pavement at the end of the crest of the dam near both abutments (Photo. 2). Subsequent excavation by the dam owner showed that one of the cracks reached the top of the core materials, but remained within the freeboard. Rock fragments from the right abutment fell down, but the upstream and downstream slopes of the dam body were not damaged.

Table 1 Main features and results of inspection of the dams investigated by PWRI within 50 km from the epicenter

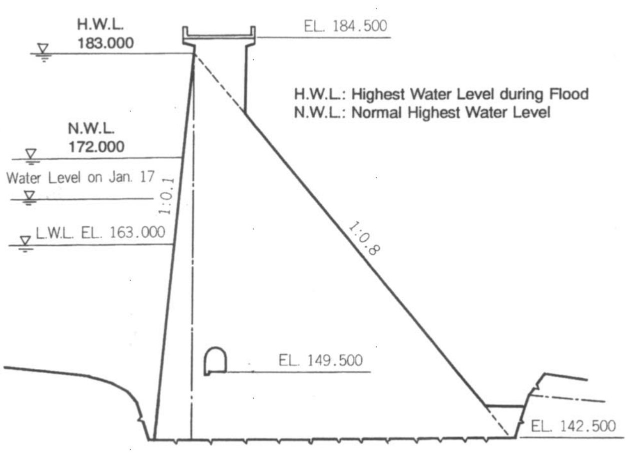
Dams within river reaches administered under the River Law

Name of dam	Name of river	Type of dam	Height of dam (m)	Depth of reservoir water during earthquake (m)	Year of completion	Damage	Distance from Epicenter (km)	Foundation *
Hitokura	Ina	Concrete gravity dam	75.0	38.4	1983	· No damage to dam body · Small rock-falling from reservoir shore	47	Sandstone, slate 1,2
Minoogawa	Minoo	Rockfill dam with central core	47.0	24.5	1983	· No damage	49	Sandstone, slate 1
Yuzuruha	Yuzuruha	Concrete gravity dam	42.0	25.1	1974	· Slight spalling of mortar of upstream face finish	43	Sandstone, shale 2
Dainichigawa	Dainichi	Concrete gravity dam	43.5	18.0	1964	· No damage	48	
Tenno	Tenno	Concrete gravity dam	33.8	13.9	1980	· No damage to dam body · Fall of rock fragments at abutment	16	Granite, granodiorite 2
Tachigahata	Ishii	Masonry dam	33.3	24.2	1905	· No damage	15	Granite 2
Sengari	Hatsuka	Masonry dam	42.4	27.6	1919	· No damage	39	Liparite 2
Maruyama (No.1)	Funasaka	Concrete gravity dam	31.0	18.4	1977	· No damage	31	Sandstone, conglomerate 3
Nariai-ike	Nariai	Masonry dam	33.0	19.4	1950	· No damage	42	Sandstone 2
Tokiwa	Nojima	Zoned earthfill dam	33.5	20.9	1974	· Transverse cracking at crest near abutments	10	Granite 2
Taniyama	Kusumoto	Zoned earthfill dam	28.2	19.8	1974	· Minor transverse cracking at crest	7	Granite 2
Dondo	Yamada	Concrete gravity dam	71.5	42.8	1989	· No damage	19	Tuff breccia
Kojiya	Shidehara	Rockfill dam with central core	44.1	16.7	1991	· No damage	48	Liparite
Kamogawa	Kamo	Concrete gravity dam	43.5	16.3	1951	· No damage	37	Quartz porphyry, tuff
Okawase	Tojyo	Concrete gravity dam	50.8	36.0	1990	· No damage	40	Tuff breccia

Dams not within river reaches administered under the River Law

Name of dam	Name of river	Type of dam	Height of dam (m)	Depth of reservoir water during earthquake (m)	Year of completion	Damage	Distance from Epicenter (km)	Foundation *
Otani	Kusumoto	Earthfill dam	16.6	—		· Minor cracking at crest near spillway contact	7	
Syowa-ike	Uzaki	Earthfill dam	16.0	—		· Slight bulging of upstream slope protection · Hairline longitudinal cracking on crest	4	
Gohonmatsu	Ikuta	Masonry dam	33.3	24.6	1900	· No damage to dam body · Fall of rock fragments at abutment	19	Granite 2
Fukatani	Sakase	Rockfill dam with inclined core	41.0	24.0	1971	· Slight settlement near spillway · Cracking on splitter of spillway · Hairline transverse cracking at crest	33	
Kitayama (No.1)	Shuku	Homogeneous earthfill dam	24.5	16.2	1968	· Shallow sliding of upstream surface	31	Granite 2

* Geological era of foundation 1: Palaeozoic, 2: Mesozoic, 3: Cenozoic (Tertiary)



(unit : m)

Figure 3 Cross section of Yuzuruha Dam

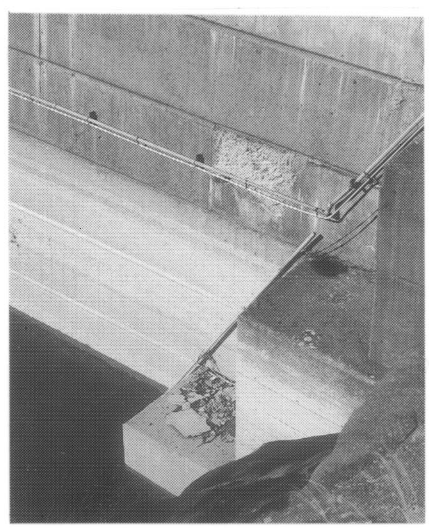


Photo. 1 Spalling of mortar on the upstream face of Yuzuruha Dam

Gohonmatsu Dam - Completed in 1900, the Gohonmatsu Dam is a concrete gravity dam (rubble masonry structure) located about 19 km from the epicenter. The height of the dam is 33.3 m. Figure 5 shows the cross section of the dam. Rock fragments at the left abutment fell into the spillway, and hairline cracks were found on the capping concrete on the crest railing, but there were no cracks on the dam.

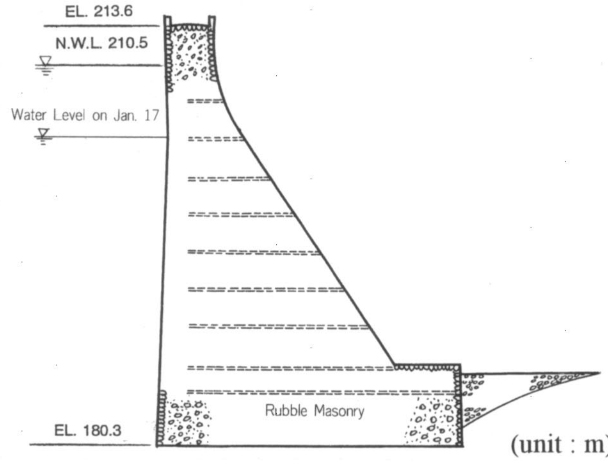


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Figure 5 Cross section of Gohonmatsu Dam

Kitayama Dam - The Kitayama Reservoir is formed by five homogeneous earthfill dams — dam No. 1 to No. 5. The reservoir is located about 31 km from the epicenter. Figure 6 shows the cross section of dam No. 1. The dam has a height of 24.5 m with decomposed granite soil used as fill material and is embanked on a foundation of weathered granite. A shallow sliding of the upstream surface of the dam occurred (Photo. 3), but the stability of the dam was not affected.

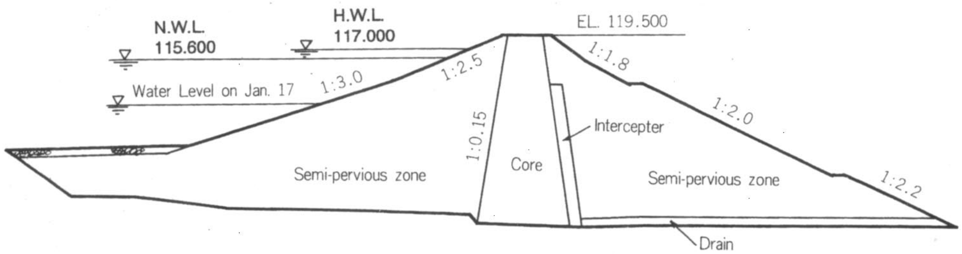

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Figure 4 Cross section of Tokiwa Dam

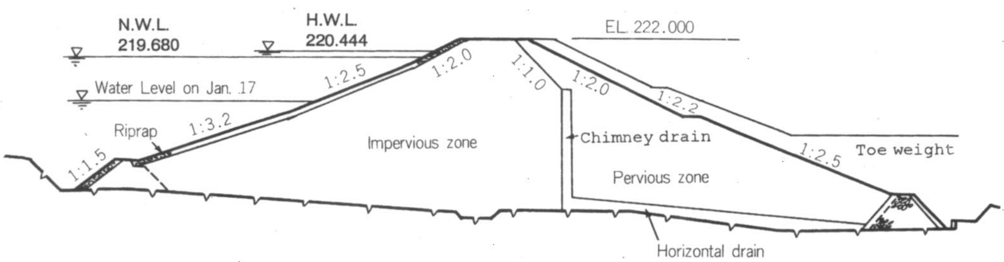

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Figure 6 Cross section of Kitayama Dam No.1

Photo. 2 Cracking at the crest of Tokiwa Dam

Photo. 3 Shallow sliding at the upstream slope of Kitayama Dam No.1

CHARACTERISTICS OF OBSERVED EARTHQUAKE MOTIONS

Peak Acceleration at Foundations of Dams

Attenuation of Peak Acceleration - The attenuation of the horizontal peak accelerations observed at the dam foundations during the earthquake is shown in Figure 7, and that of the vertical peak accelerations in Figure 8. These values include the values obtained at the gallery under the embankment dam or at the lowest gallery in the concrete dam. Figures 7 and 8 show peak accelerations measured at dam sites and those at soil sites. In Figure 7, the data with white squares show peak accelerations at dam sites in the stream direction, the black squares show those in the dam axis direction, and the white circles show those of soil sites (Committee on Measures for Bridge Damage for Hyogoken-Nambu Earthquake, 1995). Figure 8 compares peak vertical accelerations between dam sites (black squares) and soil sites (white circles). The foundations of dams consist of hard rock before Tertiary period (40 dam sites: pre-Tertiary period; 5 dam sites: Tertiary period), whereas soil sites are of alluvial or diluvial deposits. Figures 7 and 8 indicate that the peak accelerations at the dam sites were substantially smaller than those at the soil sites. The maximum observed horizontal peak acceleration at a dam foundation was 183 gal. This site is located about 47 km from the epicenter, and about 10 km from the earthquake source faults which have caused the main shock. The attenuation of horizontal peak accelerations at dam sites with distance from the earthquake source faults is shown in Figure 9. The maximum acceleration during the earthquake at a rock foundation was estimated to be about 220 gal from Figure 9 (Committee on Evaluation of Earthquake Resistance of Dams, 1995).

Vertical Accelerations and Horizontal Accelerations - The relationship between the vertical peak accelerations and the horizontal peak accelerations at dam foundations is shown in Figure 10. The ratio of the vertical to the horizontal peak accelerations is 1/3 to 1/1. The figure shows that the larger the horizontal acceleration is, the smaller the ratio is.

Acceleration Response Spectrum

The response spectrum of accelerations recorded at 25 dam foundations is shown in Figure 11. The spectrum is normalized so that the maximum acceleration is 1. The mean value of the spectrum is about 2 for the period between 0.1 and 0.6 seconds. For periods larger than 0.6 seconds, the value decreases rapidly.

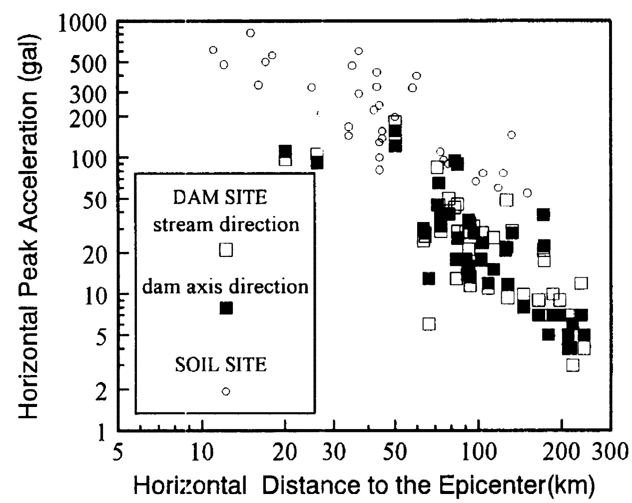

Figure 7 Attenuation of peak accelerations with distance to the epicenter (Horizontal acceleration)

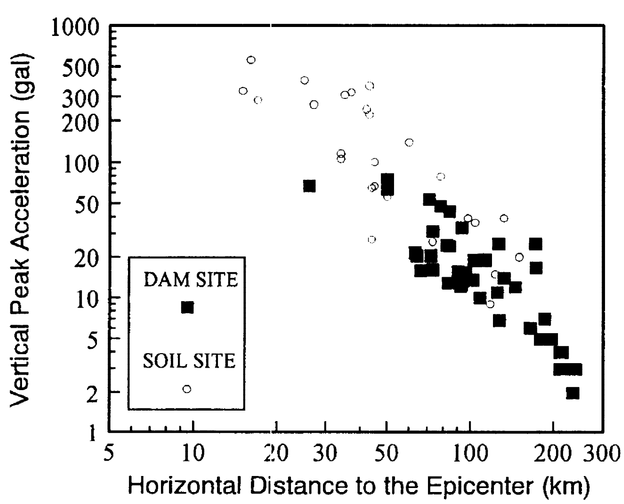

Figure 8 Attenuation of peak accelerations with distance to the epicenter (Vertical acceleration)

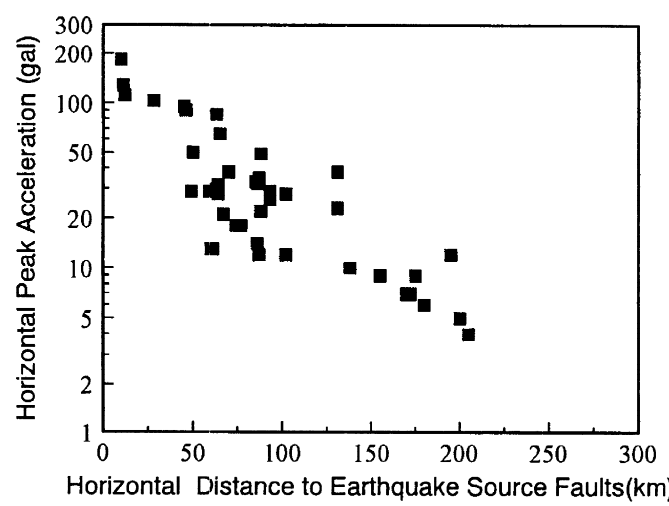

Figure 9 Attenuation of peak accelerations at dam foundation with distance to the earthquake source faults (Horizontal acceleration)

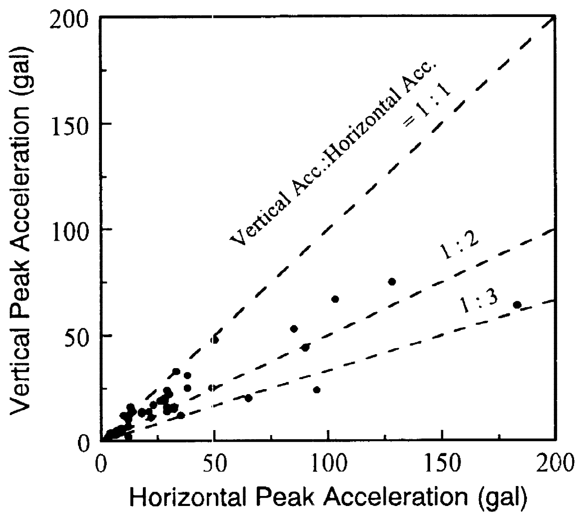

Figure 10 Relationship between horizontal peak accelerations and vertical peak accelerations

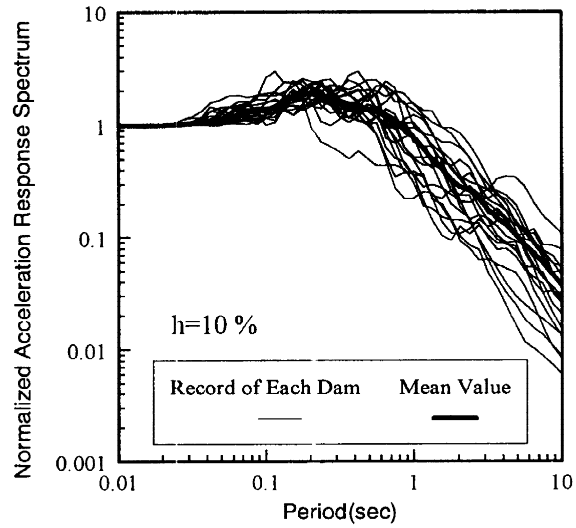

Figure 11 Acceleration response spectrum at dam foundations (Damping ratio h=10 %)

EVALUATION OF EARTHQUAKE RESISTANCE OF DAMS

Seismic Design for Dams in Japan

Dams in Japan are generally designed using the seismic coefficient method. The values of the seismic coefficient in the Japanese design criteria are shown in Table 2. The coefficient varies with the type of dam and seismic activity zone. Dams in Japan designed using the seismic coefficient method have suffered no serious damage during earthquakes so far. The earthquake resistance of dams, however, was reviewed (Committee on Evaluation of Earthquake Resistance of Dams, 1995) since many other large structures were severely damaged during the Hyogoken-Nambu Earthquake.

Acceleration Records for Analyses

As noted already, the maximum horizontal peak acceleration at dam foundations in the Hyogoken-Nambu Earthquake is estimated to have been 220 gal. The horizontal peak acceleration of 250 gal was, however, used in the analysis taking account of adequate margin of safety. The analysis used four records (ACC-1, 2, 3, 4) obtained in dam sites near the epicenter, the peak acceleration of which was increased to 250 gal. The horizontal acceleration response spectrum with the damping ratio of 10% is shown in Figure 12. The vertical acceleration was also included in the analysis.

Concrete Gravity Dams

Analysis Condition - The FEM model of a concrete gravity dam is shown in Figure 13. The height of the dam varies from 25 m to 150 m. The reservoir water is assumed to be an incompressible fluid in the analysis; the physical properties of the dam are shown in Table 3. The dynamic response of the dam is calculated using the response spectrum method.

Results of Analysis - The maximum dynamic tensile stress calculated in the dam body is shown in Figure 14. The stress increases in proportion to the height of the dam, but even the stress in the 150 m-high dam is less than 2.5 MPa. It is supposed that concrete gravity dams can withstand such dynamic stress. The maximum shear force along the contact plane between the dam and the foundation was also analyzed and the force was found to be less than the shear resistance along the contact plane. Concrete gravity dams thus have resistance against sliding.

Table 2 Seismic coefficient in Japanese design criteria for dams

Dam Type	Seismic Coefficient (Lowest Limit)		
	Low*	Medium*	High*
Concrete Gravity Dam	0.10	0.12	0.12
Concrete Arch Dam	0.20	0.24	0.24
Rockfill Dam	0.10	0.12	0.15
Earthfill Dam	0.12	0.15	0.15

* Low : Low Seismic Activity Zone
Medium : Medium Seismic Activity Zone
High : High Seismic Activity Zone

Table 3 Physical properties of material (Concrete gravity dam)

Elastic Modulus (MPa)	3.0×10^4
Poisson's Ratio	0.2
Density (kg/m^3)	2300
Damping Ratio (%)	10

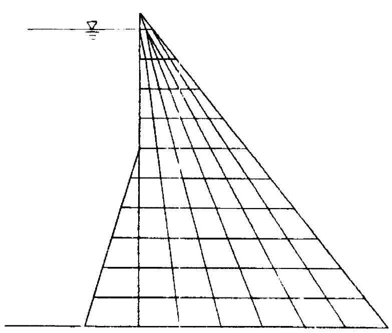

Figure 13 FEM model of concrete gravity dam

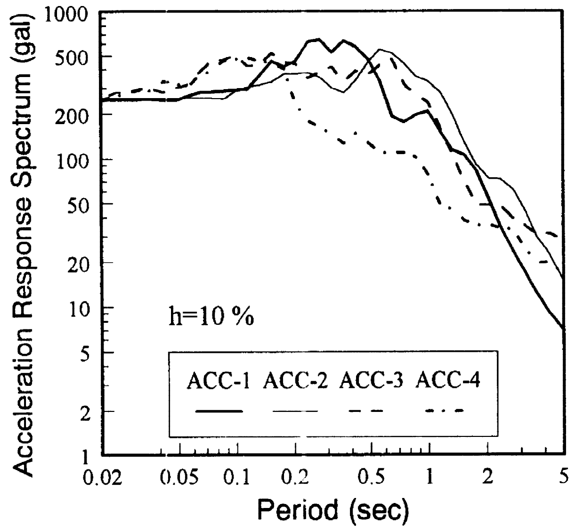

Figure 12 Acceleration response spectrum of input motions (Damping ratio h=10 %)

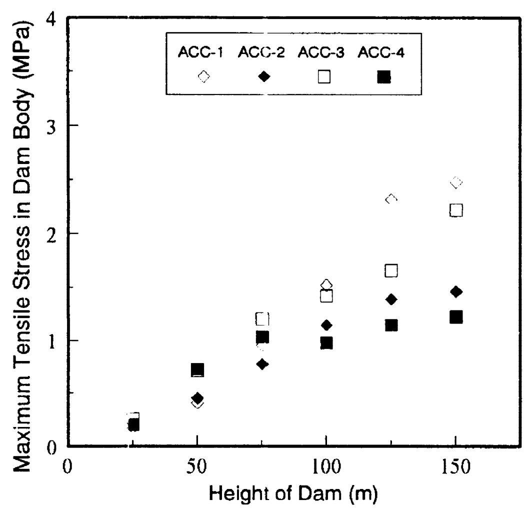

Figure 14 Maximum tensile stress in dam body with height of dam

Embankment Dam

Analysis Condition - The FEM model of a rockfill dam is shown in Figure 15. In the dynamic analysis, the physical properties of the model is assumed to be only rock whose wet and saturated density is 1880 and 2080 kg/m^3, respectively. The height of the model dam is assumed to be 63, 110 and 150 m. The relationship between the initial shear modulus Go and the depth D from the surface of dam is shown in Figure 16, and the shear strain dependent curves of the shear modulus ratio and hysteresis damping ratio of the rockfill materials are shown in Figure 17. The total damping ratio is assumed to be hysteresis damping of rockfill materials plus 15%. The poisson's ratio is 0.35. The peak seismic inertia force of the potential circular slide mass (see Fig.15) is obtained by the direct time integration using the equivalent linear method (QUAD-4), and the peak average acceleration of each slide mass is calculated as the peak seismic inertia force moment divided by the mass moment of each slide mass.

Results of Analysis - The analysis showed that the margin of safety of the 110 m-high dam is the smallest. The peak average acceleration of the slide mass for a given y/H (see Figure 15) is shown in Figure 18 with the resisting acceleration. Resisting acceleration is defined as the acceleration that would cause the potential slide mass to begin to slide, and is calculated from stability analysis using the physical properties of materials shown in Table 4. At the upper part of the dam, the peak average acceleration during the earthquake is very large, but the values are within the resisting acceleration for any y/H, so the rockfill dam is safe against sliding.

Table 4 Physical properties of materials for stability analysis (Rockfill dam)

Zoning	Wet Density (kg/m^3)	Saturated Density (kg/m^3)	Strength Parameter* A (or c)	b (or Φ)
Rock	1880	2080	A=1.128	b=0.804
Filter	2130	2240	A=0.808	b=0.908
Core	2220	2230	c=0.098 MPa	Φ=35°

* $\tau = Po \cdot A \cdot (\sigma/Po)^b$ or $\tau = c + \sigma \tan\Phi$, where τ is the shear strength, Po is 1 MPa and σ is the normal stress on the sliding plane.

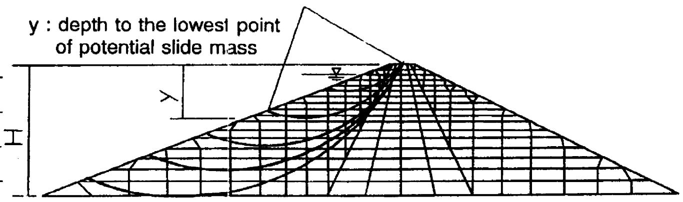

Figure 15 FEM model of rockfill dam and definition of potential slide masses

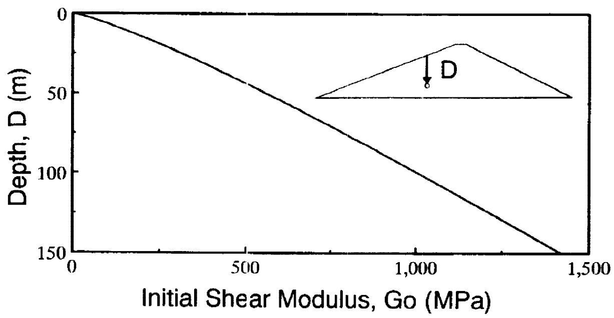

Figure 16 Initial shear modulus versus depth from the surface of the dam

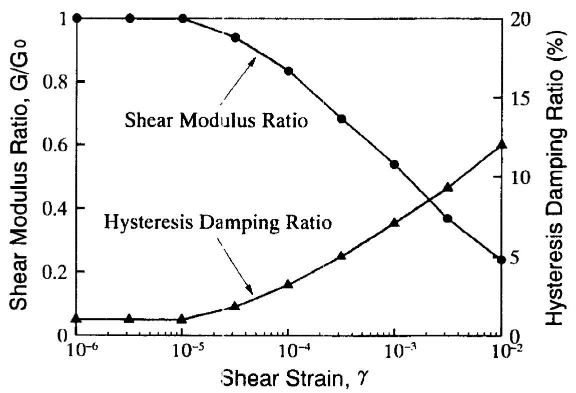

Figure 17 Shear strain versus shear modulus ratio / hysteresis damping ratio

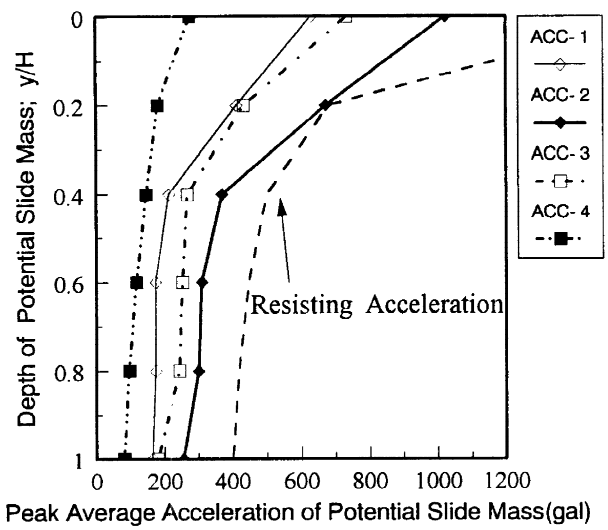

Figure 18 Peak average acceleration of potential slide mass with depth of potential slide mass for 110m-high dam

CONCLUSIONS

The special safety inspections by site offices and detailed investigations by PWRI engineers confirmed that there was no serious damage affecting dam safety or requiring immediate protective countermeasures.
The dams were constructed on the rock foundations where the earthquake motion were substantially smaller than those at soil sites. It is one of the reason why dams were safe during the Hyogoken-Nambu Earthquake. Careful geological investigation and site location, adequate safety factor in designing dams, high-quality construction were also important factors to ensure the safety of dams.

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