STUDY ON DYNAMIC BEHAVIORS OF ROCKFILL DAMS

by

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SYNOPSIS

The elastic constant of high rockfill dams recently constructed in Japan can be expressed as a function of the depth from the dam surface. The damping factor of any existing rockfill dam is several per cent within Moderate strain range. Though the natural frequencies of the dams slightly decrease due to the reservoir water, the earthquake behaviors of rockfill dam are scarcely affected. The vibration modes in stream direction largely depend upon the shape and the width of the valley. It is found that the rockfill dams behave linearly to large response.

INTRODUCTION

Study on the dynamic behaviors of fill dams during earthquakes has been conducted since Dr. Mononobe. Recently, with development of application techniques of an electronic computer, complicated numerical calculation have been become possible, so that there have been many theoretical studies based upon a finite element method involving various assumption and many arguments presented. However, as there have been very few observational studies and also physical properties of dam body materials cannot be correctly estimated yet, it should be neccesarry to excecute observations and field tests and to establish the ground of theoretical treatments.

Dynamic behaviors of some high rockfill dams recently constructed in Japan have been studied by earthquake observations and various field tests. These dams were constructed by modern construction techniques at the valley with V-shape or U-shape. The outlines of for rockfill dams examined especially in detail are in Table 1.

It is bolieved that this report answers the important problems concerning with eathquake proof design of rockfill dams.

FIELD TESTS

Investigating the Kisenyama Dam, the field vibration test with a vibration machine was conducted. In this test, the max. acceleration near the crest center was about 40 gals. This test was conducted two times, in full water level and low water level of the reservoir to investigate whether the vibration characteristics changes or not due to the difference of the reservoir water level. The resonance curves and the vibration modes from the test are shown in Fig. 2 and Fig. 3. As shown in these figures, the amplitude remarkably grows near the crest of the dam. As the reservoir water level rises, the natural frequencies in x-direction slightly lower. The equivalent viscous damping factor of the 1-st natural vibration about 5-6 per cent to critical damping on every water level.

After this test, the blasting observations and micro-tremor observations were conducted and the results were compared with those of the field vibration test. Thereafter the both observations were applied to investigate the dynamic characteristics of the other rockfill dams.

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The tail of the Vibration of the dam by blasting is similar with viscously damped free vibration. so that in addition to the natural frequencies of the dams, the equivalent viscous damping factors can be evaluated. The micro tremors were analysed by means of spectral analysis. These observations have been made once every year after completion of the dams. to examine the effect of reservoir water and passing the time on the predominant frequencies and the vibration modes. But no change is recognized at all. These results are summarized in Table 2 and Fig. 4.

WAVE VELOCITY DISTRIBUTION IN DAMS

The longitudinal wave velocity Vp and the transversal wave velocity Vs in dam body were investigated in detail after completion and under construction of the dams for the relationship between the wave velocity and the dam height and for the effect of the velocities due to reservoir water. From the results, it is shown that both Vp and Vs greately increase according to the construction height, so that they are greater at deeper portion of the dams. As for the change of the velocities due to the reservoir water, Vs in rock and core Zone were scarecely changed, but Vp in the upstream rockfill zone greatly increased due to the reservoir water.

In general, the relationship between Vs and depth z can be represented by the following formula, although different coefficients should be used in the upper part, from the crest to about 25 m. below, and in the lower part ;

$$Vs = a\ Z^{b}$$

The coefficients, a and b, of the upper part widely vary according to different positions of the dam and different dams. On the other hand, these in the lower part of any rockfill dam are approximately the same values (Fig. 5, Fig. 6 and Table 3).

From the resluts, it may be quite all right that the physical property of lower part is hardly effected due to dam body materials and construction condition, although that of the upper part widely changes due to these. The fact is very important to pay great attention for the theoretical study on behaviors of rock fill dams during earthquakes.

EARTHQUAKE OBSERVATIONS

Many transducers are installed not only on the surface and the foundation, but also within the dams. These observation locations are well arranged for the earthquake behaviors of the dams (Fig. 1).

A great number of earthquake records have been so far obtained. In analysing earthquake records, the predominant frequencies and the vibaration modes were investigated in detail by spectral analysis (Fourier spectral analysis and Power spectral analysis and Max. Entropy Method). These results are shown in Table 2 and Fig. 4. The 1-st vibration mode was predominant on each dam during earthquakes. Fig. 4 shows the 1-st vibration mode obtained from the spectral analysis. As shown in these fi figures, the vibration mode of the Tataragi Dam with U-shape valley grows linearly large from the foundation to the dam crest, but those of the other dams with V-shape valley grow especially large near the crest.

The equivalent viscous damping factors evaluated by the spectral analysis are shown in Table 2. It is found that the equivalent dam damping factor on any dam is equally about 5-6 per cent to critical damping.

CONCLUSION

The earthquake behaviors of rockfill dams were analysed by various methods, a finite element method, a shear vibaration theory and a multiple reflection theory, and the analysed results were compared with the observed results for actual treatments in the earthquake proof design of rockfill dams. In these analyses, elastic constants of dam body materials obtained from the observed wave velocities were employed.

As shown in these results, in any method the natural frequencies of the dams calculated were approximatly coincident with the observed one. But the 1-st vibration mode disagreed with the observed at the upper part of the dams.

The relationship between the max. response acceleration of the dam crest and the max. acceleration of the foundation during earthquakes are shown in Fig. 7. The earthquake responses of the dams are very different according to earthquakes. The responses of the Kisenyama Dam are the Greatest of the other dams. As shown in Fig. 4, this is because the mode shape of the Kisenyama Dam become greater near the crest than that of the other dams.

Up to date, the observed max. responses of rockfill dams was about 100 gals on the top of the Kisenyama Dam. The earthquake (Magnitude 7) occured at the central part of Japan on the 9-th 1969 Sep.. The mean value of strain between the crest and 25 m below from the dam crest due to this earthquake was 1.5×10^{-4}, and it is shown from the previous results that the dam linearly behaved during earthquake.

The difference of water levels of the Kisenyama Dam is about 26 m per a day. It is observed that the displacement of the dam due to water load is proportional to the difference of the water level and the difference of the displacement between the crest and 25 m below from the crest is 1.5 cm (Fig. 8.) The mean value of the strain estimated between the crest and 25 m below is 6×10^{-4}.

It may be quite all right to consider that the dam linearly behaves within the above strain range during earthquakes. Suppose that the dam induces the 1-st natural vibaration and the amplitude of the crest is 1.5 cm, the acceleration of the crest is estimated 260 gals. It is convinced that the rockfill dam linearly behaves over considerably great responce.

ACKNOWLEDGMENT

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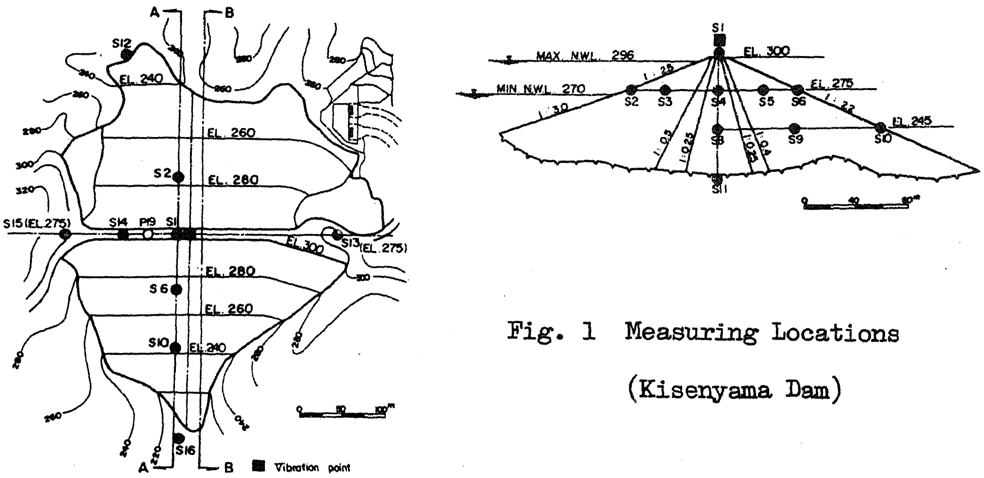

Fig. 1 Measuring Locations

(Kisenyama Dam)

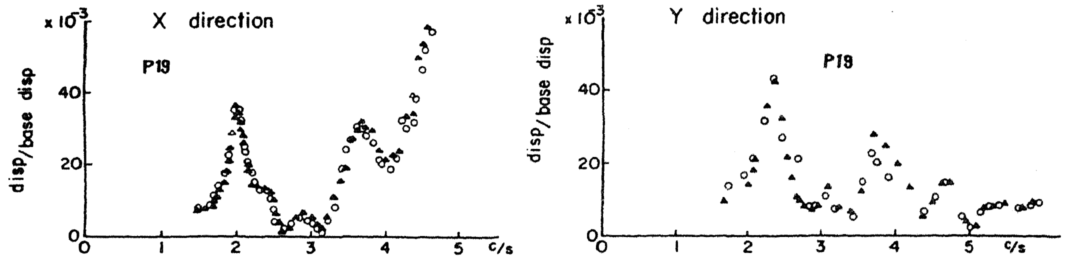

Fig. 2 Resonance Curves (Kisenyama Dam)

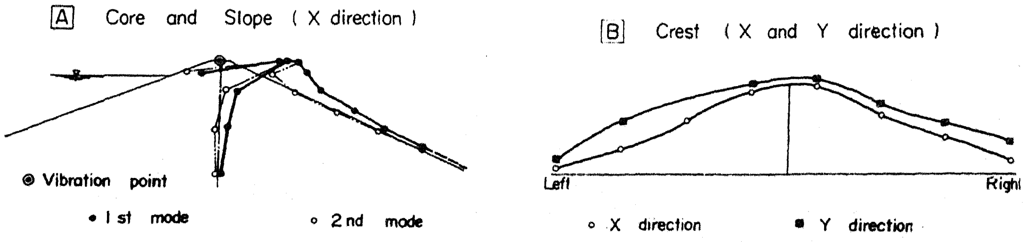

Fig. 3 Vibration Modes (Kisenyama Dam)

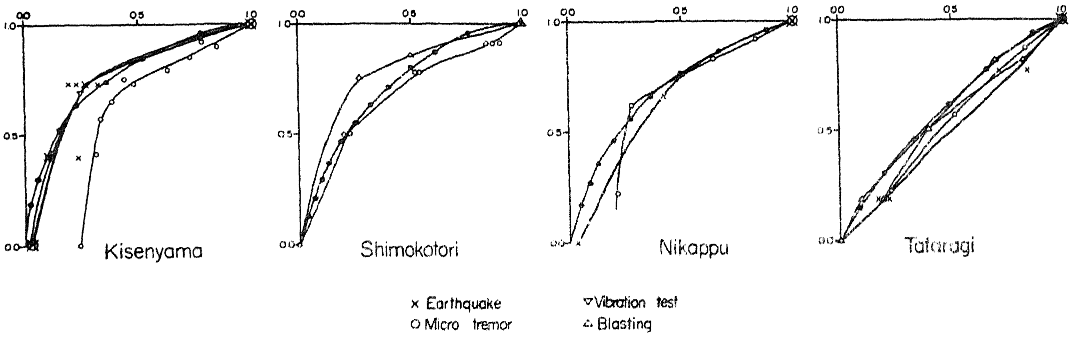

Fig. 4 The 1-st Vibration Mode in X-direction

(along the vertical axis in the Core zone)

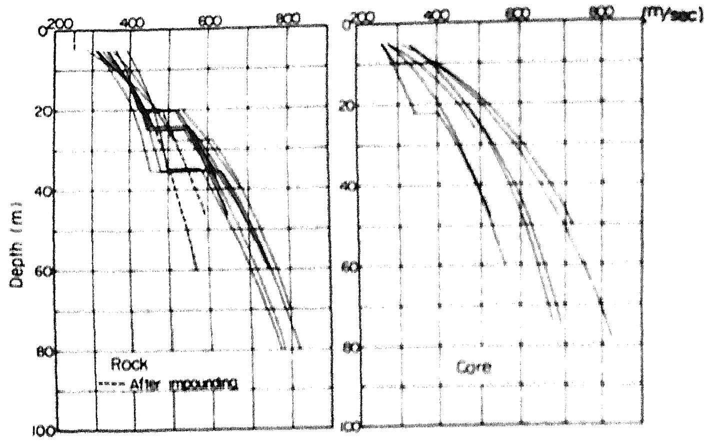

Fig. 5 Shear Wave Velocity Distribution

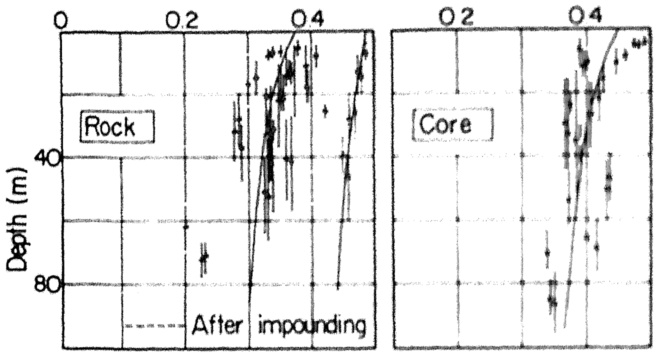

Fig. 6 Poisson's Ratio Distribution

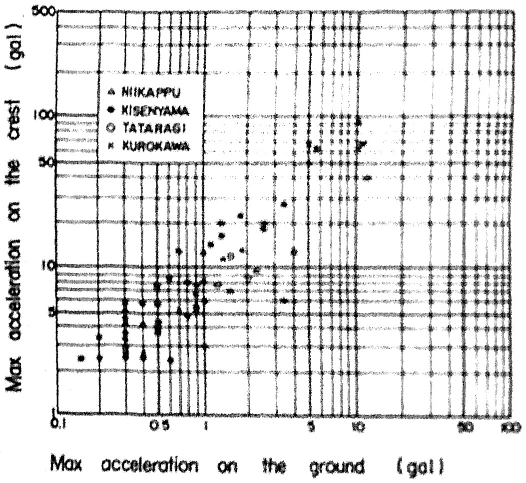

Fig. 7 Max. Acceleration of the Crest vs. the Foundation

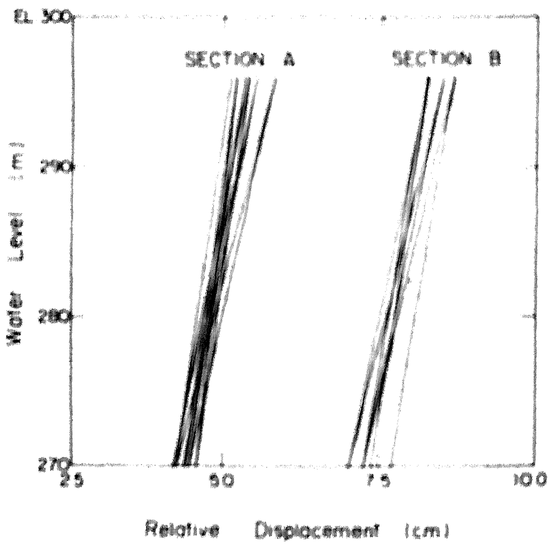

Fig. 8 Relative Displacement due to Water Load

(Crest - EL. 275m = 25m)

Table 1 Outline of the Dams

Dam Name		Kisenyama	Shimokotori	Tataragi	Niikappu
Type		Center Core	Center Core	Asphalt Facing	Center Core
Height(m)		95	119	64.5	102.8
Crest Length(m)		267	289	283	326
Slope	Up-Str.	1 : 2.5	1 : 2.4	1 : 1.8	1 : 2.3
	Down-Str.	1 : 2.2	1 : 1.85	1 : 1.75	1 : 1.9
Dam Volume (m^3)		2.34×10^6	3.53×10^6	1.46×10^6	3.06×10^6
Valley Shape		V-Shaped	V-Shaped	U-Shaped	V-Shaped

Table 2 Predominant Frequencies and Damping Factors

Dam Name	Height(m)	Mode	Predominant Frequency(Hz)			Damping Factor(%)	
			Micro Tremor	Earth-quake	Vibration Test	Earth-quake	Blasting
Kisenyama	95	X 1st	2.0	1.9~2.1	2.1	4.5~7.6	
		X 2nd	3.5		3.7		
		Y 1st	2.4	2.1~2.4	2.4		
		Z 1st	2.9	2.8~3.0	2.9		
Tataragi	64.5	X 1st	2.6	2.5~2.6		4.3~4.6	6.9
		Y 1st	2.8	2.8			
		Z 1st	4.5	4.0~4.5			
Shimo-Kotori	119	X 1st	2.2~2.4	2.2~3.2			5.0~5.4
		X 2nd	3.5				
		Y 1st	2.4~2.6	2.2~3.2			5.0
		Z 1st	2.9	3.6			
Kurokawa	98	X 1st		2.3~2.5			
		Y 1st		2.5~2.8			
		Z 1st		3.6~3.7			
Niikappu	102.8	X 1st	2.2	2.2		4.3~7.3	
		Y 1st	2.6	2.6			
		Z 1st	3.7~4.2				
Kuzuryu	128	X 1st	3.0	1.9			
		Y 1st	2.5	2.4			
Miboro	135	X 1st	2.7	1.7			
		Y 1st	2.7~3.1	3.8			
Honzawa	73	X 1st		2.78			
Miyama	75	X 1st		2.5			
Yanase	115	X 1st		2.38			

Table 3 Typical Shear Velocity and Poisson's ratio

Zone Depth Z(m)	Rock		Core	
	Non-Saturated	Saturated	High	Low
0 ~ 5	Vs = 245		Vs = 210	
5 ~ 30	$V_s = 250\ Z^{0.20}$	$V_s = 250\ Z^{0.20}$	$V_s = 180\ Z^{0.35}$	$V_s = 140\ Z^{0.34}$
30 ~	$V_s = 200\ Z^{0.315}$			
Poisson's Ratio	$\nu = 0.375 - 0.006\ Z^{0.58}$	$\nu = 0.49 - 0.01\ Z^{0.95}$	$\nu = 0.45 - 0.006\ Z^{0.60}$	