EARTHQUAKE-RESISTANT DESIGN FEATURES OF DAMS IN JAPAN

JAPANESE NATIONAL COMMITTEE ON LARGE DAMS

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1. INTRODUCTION

The Japanese Islands are located in the Circum-Pacific Seismic Zone and they have been frequently affected by serious earthquakes. Therefore, in the design of almost all structures, it is a normal practice to take into accout the effect of earthquake which is

Table 1 Statistical Table of Large Dams in Japan

year	type	height in meter					total	
		15~30	31~60	61~100	101~150	over 150		
~1930	TE	718	7				725	781
	PG	35	13	2			50	
	CB	4	1				5	
	MV	1					1	
1931~1950	TE	249	13				262	376
	CB	1					1	
	PG	60	44	9			113	
1951・1968	TE	192	27				219	586
	ER	7	7	3	4		21	
	PG	107	131	55	7	2	302	
	CB		3	5	2		10	
	VA	4	5	14	8	1	32	
	MV		1	1			2	
1969~1978	TE	65	25	1			91	317
	ER	15	37	16	6	1	75	
	PG	36	75	20	3		134	
	CB			2			2	
	VA		1	8	5	1	15	
1979~1990.3	TE	33	23				56	281
	ER	14	51	20	3	1	89	
	PG	22	75	34	4		135	
	CB							
	VA				1		1	
	MV							
total	TE	1257	95	1			1353	2341
	ER	36	95	39	13	2	185	
	PG	260	338	120	14	2	734	
	CB	5	4	7	2		18	
	VA	4	6	22	14	2	48	
	MV	1	1	1			3	
grand total		1563	539	190	43	6	2341	

Note: TE—earth, ER—rockfill, PG—gravity, CB—buttress (incl. hollow gravity), VA—arch, MV—multi-arch
Large dam means 15 m or higher

an important external force. From olden days many earth dams for irrigation purpose have been constructed in Japan, and it was 1950' that many large modern dams have been built (see Table 1). Based on experinces and studies during those time, a general design procedure is established at present for all types of dams. This paper will deal with the recent trends in the earthquake resisting features in the design in dams, which are, damages to dams by earthquakes in the past, earthquake resistant design methods in practice and research activities to produce more rationalistic design methods.

2. SEISMICITY AND ITS INFLUENCES ON DAMS IN JAPAN

In Japan, in ancient times, agriculture developed primarily through cultivation of rice paddies. Many earth dams were built to assure supplies of water for irrigation, and among those existing the oldest dates back over 1,000 years. These old dams were constructed based on experience gained in those times. However, most dams approximately 50 meters and over in height, including dams for other purposes, were constructed in the last forty years. Therefore, at the time of the Kanto Earthquake of September 1, 1923, the number of high dams 50 meters or more in height existing was small. Since then, Japan has been affected by sevetal severe earthquakes. Investigations on dynamic behaviors of dams during these earthquakes were important factors in studies on earthquake-resistant features in the design of dams.

The probability of recurrence of earthquakes taken into account in Japanese design practice is considered to be once in 100 to 200 years and its validity is substantiated by statistical studies on the earthquake resistances of dams to past serious earthquakes. The engineering judgment on the recurrence probability of the design earthquakes as mentioned above may be correct. However, there is a report published recently by the Public Works Research Institute of Japan[34] concerning the design earthquakes worthy of due consideration, a summary of which is shown as Table 2.

In preparing this table, 60 dam sites were sampled at random from all over the Japanese islands and 611 historical earthquakes causing damage in and around Japan from 599 A.D.

Table 2 Classification of Historical Earthquakes Causing Maximum Acceleration at Sixty Dam Sites

Δ (km) \ M	$M<6.0$	$6.0\leqq M<6.5$	$6.5\leqq M<7.0$	$7.0\leqq M<7.5$	$7.5\leqq M$	Total
$\Delta<10$	1	2	1	4	0	8
$10\leqq\Delta<20$	0	2	4	3	1	10
$20\leqq\Delta<30$	0	2	4	6	1	13
$30\leqq\Delta<40$	0	2	6	2	0	10
$40\leqq\Delta$	0	3	5	8	3	19
Total	1	11	20	23	5	60

to 1973 A.D. were adopted from Usami's earthquake list, and the magnitudes and epicentral distances of these eatthquakes were classified and tabulated. From the table it can be seen that about 70% of the earthquakes producing maximum acceleration at the above-mentioned 60 dam sites had magnitudes between 6.5 and 7.5, while epicentral distances from the dam sites were mostly within 60 kilometers. Consequently, against such earthquakes, design earthquake motions must be determined without consideration of the probability of recurrence.

To add to the above cognizance of the influences of seismicity on dams, the following comments will be made concerning damage to dams in past earthquakes in Japan and the characteristics of the earthquake motions actually recorded at the dams.

The Ministry of Construction, in order to investigate earthquake resistances of dams in past earthquakes, carried out a survey by questionnaire in 1977 on 2,221 existing dams in Japan 20 meters or more in height. The earthquakes considered were the forty-four of magnitude (hereafter abbreviated M) of 6 or greater starting with the Tottori Earthquake of September 10, 1943 up to the earthquake of M of 6.4 which occurred in the Kyushu Region on April 21, 1975. Contrasted to these, with regard to earthquakes of $M<7$, dams within epicentral distances (hereafter indicated by $\varDelta$) of 100 kilometers, and with regard to earthquakes of $M\geqq 7$, dams within $\varDelta$ of 200 kilometers, were made objects of the survey. In investigations of damage caused the dams due to earthquakes, since there could be no disturbance if acceleration were to be small, out of all of the dams investigated studies were limited to those subjected to earthquake intensities 4 or 5 and over in case of fill dams, and 5 and over in case of concrete dams, but the cases studied numbered 655 as there were some dams subjected to earthquakes several times. The "disturbances" as defined in this investigation were the three kinds of ① case of cracks formed in dam body and spillway, ② case of distinct permanent deformation (settlement or horizontal displacement) occurring at body of fill dam, and ③ case of leakage increasing 10 l/min or more at a concrete dam, or for a fill dam, an increase due to earthquake taking into consideration the accuracy of measurements.

In the following, the occurrences or otherwise of disturbances are described for the principal earthquakes.

(A) Tottori Earthquake (September 10, 1943, M=7.4)

The number of dams surveyed was 156, but there was no damage even including Mitani Dam (concrete gravity type, hereafter abbreviated as PG, dam height, hereafter abbreviated as H, 27 m, $\varDelta=8$ km) which was close to the epicenter.

(B) Tonankai Earthquake (December 7, 1944, M=8.0)

There were no disturbances at all 86 of the dams surveyed.

(C) Mikawa Earthquake (January 13, 1945, M=7.1)

The number of dams included in the survey was 145, and with the exception of Oiike Dam (earth fill, hereafter abbreviated as TE, H=31 m, $\varDelta=27$ km), there were no disturbances at all. Whether or not there was damage at Oiike Dam is not clear either, all that exists being a record of repairs having been made on the dam body.

(D) Nankai Earthquake (December 21, 1946, M=8.1)

The number of dams made the objects of survey was 68 with no damage to any one of them.

(E) Fukui Earthquake (June 28, 1948, M=7.3)

The number of dams investigated was 124, and there was no damage even including Bushuko Dam (TE, H=20.3 m, Δ=18 km) close to the epicenter.

(F) Imaichi Earthquake (December 26, 1949, M=6.7)

The dams investigated numbered 18 and there were no disturbances including Nakaiwa Dam (gravity arch type, hereafter abbreviated GA, H=26.26 m, Δ=9 km) close to the epicenter.

(G) Tokachi-oki Earthquake (March 4, 1952, M=8.1)

There were 5 dams investigated with no damage at all.

(H) Yoshino Earthquake (July 18, 1952, M=7.0)

There were 177 dams surveyed including 14 earth dams within 30 kilometets of the epicenter such as Hongo Tameike Dam (TE, H=22.3 m, Δ=12 km), Shirakawa Tameike Dam (TE, H=27.6 m, Δ=14 km), Daimon-ike Dam (TE, H=32 m, Δ=17 km, built in Heian Period), but there was no damage at all.

(I) Kitamino Earthquake (August 19, 1961, M=7.0)

The number of dams investigated was 211, among which there were dams such as Miboro (rockfill, hereafter abbreviated as ER, H=131.0 m, Δ=18 km), Sasagawa (PG, H=76.0 m, Δ=29 km) and Soyama (DG, H=73.2 m, Δ=54 km) which were of large scale and close to the epicenter, but there was none subjected to damage. However, there is a record of repairs having been made later to the dam body in the case of Soyama Dam only.

(J) Miyagi-ken Hokubu Earthquake (April 30, 1962, M=6.5)

There were 25 dams surveyed with no damage even including Narugo Dam (arch type, hereafter abbreviated as VA, H=94.5 m, Δ=34 km) close to the epicenter.

(K) Niigata Earthquake (June 16, 1964, M=7.5)

The number of dams surveyed was 110. Damage occurred in the bodies of Minase Dam (ER, H=66.5, Δ=144 km) and Miomote Dam (PG, H=87.5 m, Δ=42 km), leakage changes were seen and damage requiring repairs of the dam bodies was sustained. Damage occurred at Hirusawa Dam (TE, H=24.2 m, Δ=100 km) and Sekishiba Dam (TE, H=31.8 m, Δ=102 km) also and repairs of the dam bodies were made, but it is not clearly known whether there were leakage changes. Damage occurred at the bodies of Kijiyama Dam (hollow gravity type, hereafter abbreviated as CB, H=46.0 m, Δ=68 km), Kanno Dam (PG, H=44.5 m, Δ=74 km), and Yanasawa Dam (TE, H=22.5 m, Δ=94 km), but repairs to the bodies were not required. Leakage changes occurred at Yakuwa Dam (PG, H=97.5 m, Δ=61 km), but dam body repairs were not required. Arasawa Dam (PG, H=61.0 m, Δ=52 km) was subjected to slight effects at the dam and surroundings of the reservoir.

(L) 1968 Hyuga-nada Earthquake (April 1, 1968, M=7.5)

The number of dams surveyed was 102, but there was none at which damage occurred.

(M) 1968 Tokachi-oki Earthquake (May 16, 1968, M=7.9)

There were 3 dams investigated and no damage occurred, but 7 dams built in ancient times and not objects of the survey (earth dams less than 20 meters in height) were broken.

(N) 1978 Miyagi-ken-oki Earthquake (June 12, 1978, M=7.4)

There were 198 dams (more than 15 meter high) in Tohoku district on the time. The items of the dams were PG 57, CB 2, VA 3, TE 133 and ER 6. According to the inspection following to the inspection guide after earthquakes, 192 dams had nothing wrong, but some comments had been reported on 6 dams. For some concrete dams, seepage water from lateral joints increased slightly at the moment, however, no damage had been found on the fill dams. Especially at Tarumizu dam (Rockfill, 43 meter high, epicentral distance 150 km) maximum acceleration observed on the foundation was 236 (gal), nevertheless nothing wrong in both foundation and dam body had been reported in the inspection result. It can be seen that rockfill dams have large resistivity against strong motion earthquakes.

(O) 1983 Nihon-kai Chubu Earthquake (May 26, 1983, M=7.7)

At more than 10 dam sites the earthquake motions were observed with the seismographs installed on dams. These dam sites are located at the distances of over 150 (km) from the epicenter, so that every maximum acceleration was not so large, though every record showed a specific response behavior of each dam. Some gravity dams were found to increase slightly the amount of leakage water from construction joints according to the inspections, however, it regained stable state of each dam within a year.

(P) 1984 Nagano-ken Seibu Earthquake (September 14, 1984, M=6.8)

Makio dam (Rockfill dam, 105 meter high) is situated at the distance of only 5 (km) from the epicenter. Some lateral cracks appeared at the crest (deepest one was 1.2 m in length) as well as a little settlement in the downstream shoulder was recognized, however, nothing except for them was found. There are several high dams within 50 (km) from the epicenter, nevertheless every one had nothing wrong after the earthquake.

(Q) Other

Although not named, the earthquake (M=6.6) which occurred at the western tip of Shikoku on August 6, 1968 caused disturbances in the body of Sekichi-ike Dam (TE, H=22.5 m, Δ=20 km) in Ehime Prefecture. It is unknown whether leakage change occurred, but repairs were required on the dam body. Ainono Dam (TE, H=40.8 m, Δ= 10 km) sufferred damage requiring repairs of the dam body due to the earthquake of October 16, 1970 with its epicenter at the inland part of Akita Prefecture, whereas at Ishibuchi Dam (ER, H=53.0 m, Δ=20 km) there was no damage to the dam body and repairs were not needed, but a leakage change occurred. At Yuda Dam (GA, H=89.5 m, Δ=20 km) there were no disturbances concerning the dam body, but effects were seen at the surroundings of the reservoir.

On the dams surveyed, those which showed even the slightest disturbances were 17 out of the 655 taken up as objects of investigation, with only 11 cases which fitted the definition given earlier. The breakdown consisted of 3 cases of concrete dams involving leakage changes accompanying opening of contraction joints of the gravity type dams, while the remaining 8 cases were fill dams. Of these 8 cases, except for Hirusawa Dam (TE) and Sekichi-ike Dam (TE), the disturbances were not such that it was thought the safety of the dams would be affected, and the two dams named above are considered to have been special cases. In short, they were old earth dams for which modern soil mechanics and construction methods had not been employed.

3. INSTRUMENTATION

(1) Purposes

One of the purposes of earthquake observation of dams and their foundations is safety supervision of the dam structures and reservoirs themselves. The administrative regulations concerning dam structures established by the Ministry of Construction of Japan places the owners of dams under the obligation to perform emergency inspections of all structures at dam sites when earthquake motions recorded at the sites exceed a third of design seismic coefficient. In the case of a dam with no seismic instrumentation, the emergency inspection must be carried out when the intensity of the earthquake exceeds 4 on the seismic scale of the Meteorological Agency of Japan. Other purposes of observation are further examination of the design seismic coefficients at dam sites, grasping of the behaviors of dams and their foundations during earthquakes, rationalization and improvement of design earthquake motions through gathering and analyzing of records of strong motion earthquakes, investigation of the mechanism of occurrences of earthquakes near dam sites, namely, induced earthquakes, etc.

(2) Measurement Items and Instruments

Measurement items may be divided into dynamic and static ones; the former records the dynamic behaviors of dams, and the latter the differences in characteristics of dams before and after earthquakes. The two types of measurement are summarized as follows:

(A) Dynamic Measurement Items and Locations

a.	Earthquake motions (acceleration, velocity, displacement)	Dam body, foundations, surroundings
b.	Stresses and strains	Dam body, foundations
c.	Pore pressure, uplift, hydrodynamic pressure	Dam body, foundations

(B) Static Measurement Items and Locations

a.	Displacements, inclinations, cracking	Dam body, foundations, surroundings
b.	Reservoir leakage	Dam body, foundations
c.	Stresses and strains	Dam body, foundations

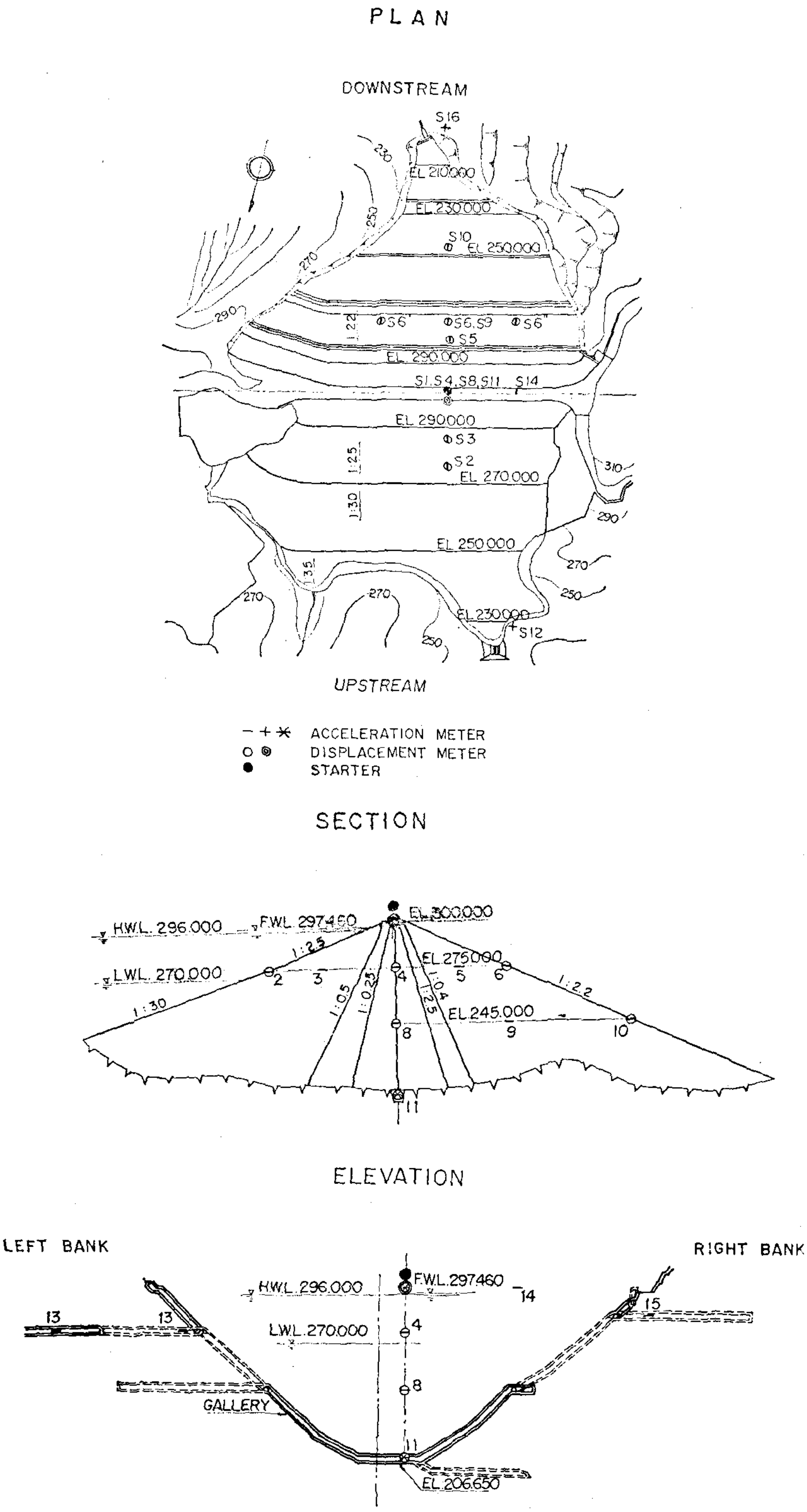

Fig. 1 Kisenyama Dam

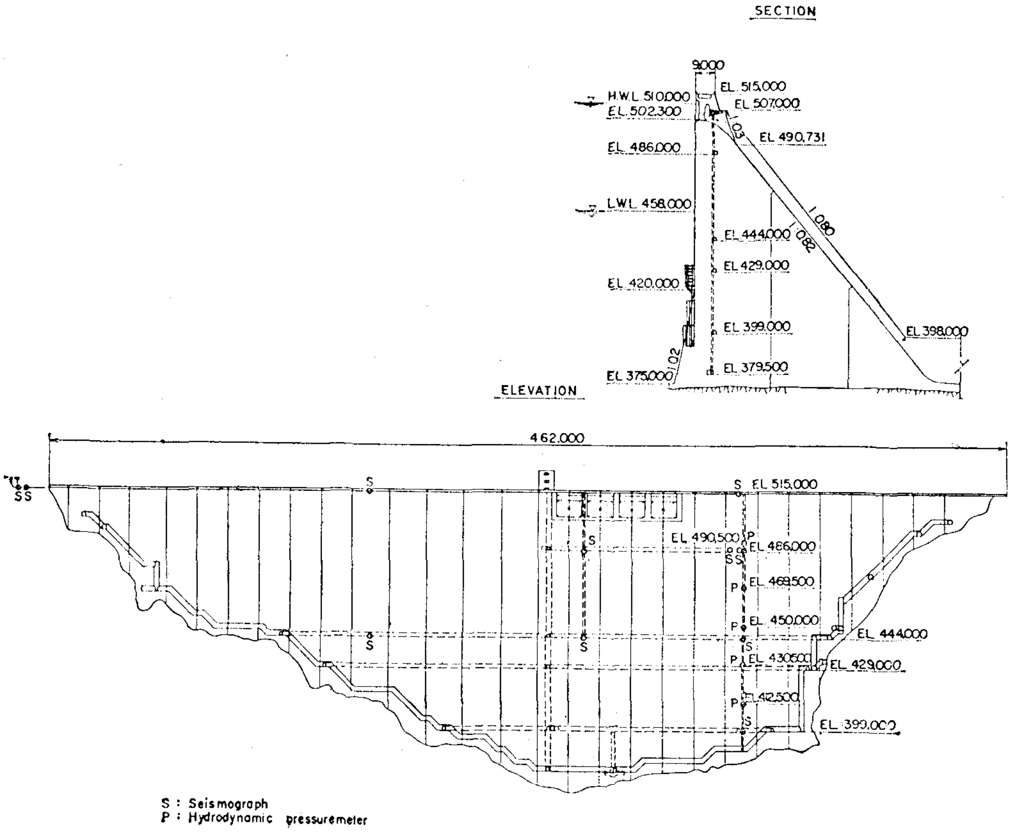

Fig. 2 Tagokura Dam

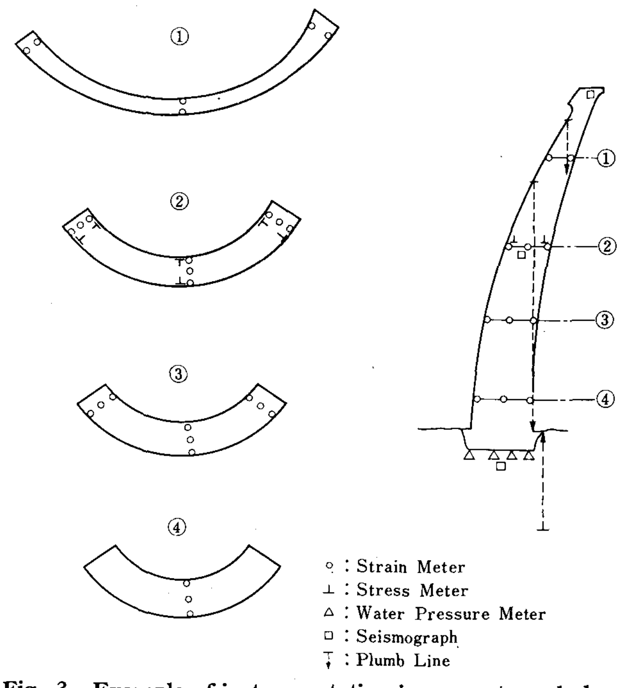

Fig. 3 Example of instrumentation in concrete arch dam

d. Pore pressure, uplift, ground water level — Dam body, foundations, surroundings

The instruments used include both dynamic and static ones. There are various types of transducers, and each type is based on a different principle.

Seismographs are used for measuring earthquake motions. In Japan, strong motion seismographs of SMAC and electromagnetic types are used very often. The SMAC type is suited to observation at sites of low frequencies of occurrence of strong motion earthquakes because of the simplicity of maintenance. The electromagnetic type can be laid under a dam body and is suited to observation of whole behaviors of dams during earthquakes at sites with high frequencies of occurrence of earthquakes although maintenance is not easy.

Instruments other than seismographs may be divided into electrical, mechanical, hydraulic, and optical types depending on the measuring mechanism. The types of the instruments must be selected according to the item to be measured. Generally, electrical and mechanical types are used for measurements inside the dam body such as of stresses, strains, pore pressures and deformations, while hydraulic, optical and mechanical types are mainly used for observations outside the dam body such as of deflections, displacements, and other factors which can be observed outside.

(3) Arrangement and Installation

The items, number, and distribution of measurements must be selected depending on the type of the dam, importance of the dam, seismicity of the site, and other factors. Some examples of instrumentation in fill dams and concrete dams are illustrated in Figs. 1 to 3.

4. BEHAVIORS OF DAMS DURING EARTHQUAKES

Studies concerning movements of dams during earthquakes have been carried out from a long time ago, and a great deal of results have been attained. These studies all have been made in attempting to obtain a rational earthquake-resistant design method, and a number of these results is being incorporated in actual designs. In the following, the vibration properties of dams during earthquakes are contemplated while touching upon the outlines of these study results.

(1) Concrete Gravity Dams

Since the series of studies by N. Mononobe regarding vibration of gravity dams[1], such dams were all handled as indicating rigid vibration. That is, design calculations were made considering that the various parts of a dam show vibrations identical to the dam base. However, with large gravity dams of 150-meter height and 400- and 500-meter crest length class as seen in recent years, it should probably be considered that such dams show a type of flexural vibration movement as pointed out by T. Hatano et al.[26] in papers and based on measurement results. It was learned from earthquake observation records of Tagokura Dam of height of 150 meters that, as shown in Fig. 4, the crest of the dam vibrated several times more than the dam base.

As seen from the results of many studies and measurements, the natural period of a high gravity dam of 250-meter class is around 0.4 sec, while the damping coefficient is thought to be about several percent. Consequently, it should be possible for resonance phenomena to occur depending on an earthquake within the range of the predominant period of the foundation rock, with the epicenter near by and waves of short periods numerous. Tagokura Dam is at a location approximately 100 kilometers from the epicenter of the Niigata Earthquake of 1964, and at the time of that earthquake a record was obtained based on which it could be thought vibrations close to resonance occurred. The maximum acceleration of the base of Tagokura Dam in this earthquake was about 70 gal, and there was no damage at all to structures at the dam site due to the earthquake.

As described above, vibrations of a gravity dam should be handled as those of an elastic

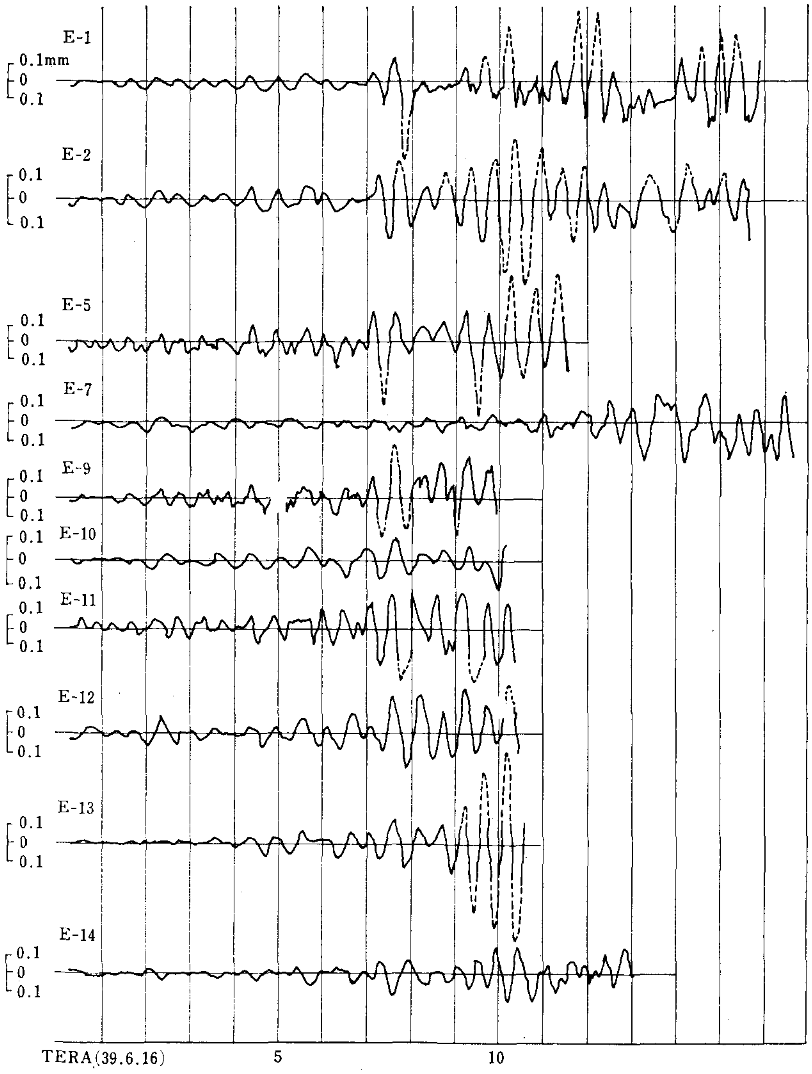

Fig. 4 Tagokura Dam Site Record (Niigata Earthquake)

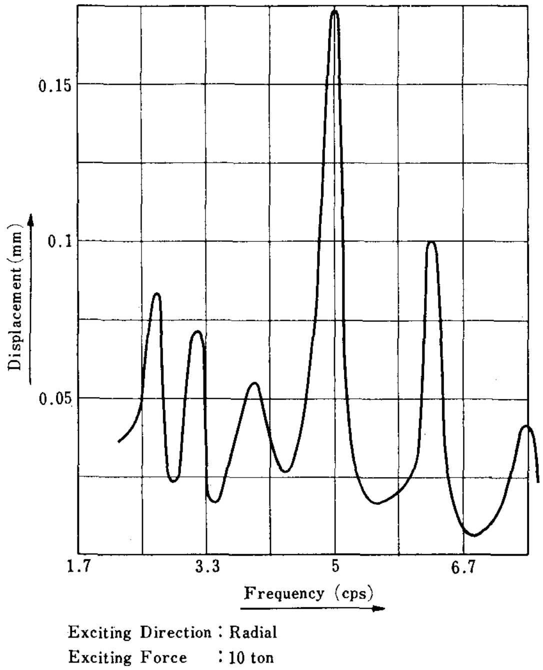

Fig. 5 Sakamoto Dam Frequency-Displacement Curve

body, but with a low dam under 50 meters, it is thought handling as rigid vibration as before will not be seriously erroneous. However, with regard to high dams of 50 meters and over, it is thought design calculations should be made considering seismic coefficient distribution corresponding to vibration deflection.

(2) Hollow Gravity Dams

The weakness of this type of dam against earthquake motion is said to lie in the vibration of buttresses against earthquake motions in the axial direction of the dam. According to the results of model vibration tests of Hatanagi No. 1 Dam and Ikawa Dam carried out by T. Takahashi[27] of the Central Research Institute of Electric Power Industry, the natural periods of buttresses were approximately 6.5 cps and approximately 10 cps, respectively, and damping coefficients were all extremely small. Further, from the results of tests on the actual dams, it was learned that vibrations in the direction of the dam axis should indeed be watched and that there is a tendency for stresses to be concentrated at parts around the crest of the dam where cross sections are thin.

(3) Arch Dams

Construction of arch dams was not seen in Japan until recent years in consideration of the country's natural conditions, namely, heavy flooding and frequent occurrence of earthquakes. However, since construction of Kamishiiba Dam, the first in Japan, there have been many arch dams built. During this time, there has been an extremely large number of studies made on the earthquake resistance of arch dams. Especially, precise model vibration tests have been widely performed to back up theory, while further, vibration tests of actual arch dams completed have been performed and actual movements during earthquake recorded with seismograph groups installed at dams and surrounding bedrock, so that it may be said the vibration characteristics of arch dams have been determined fairly accurately.

An arch dam, compared with dams of other types, is the most elastic structure, while individual designing is done in accordance with the shape of the valley and conditions of the foundation rock, and it is difficult to make a general statement. For example, an extremely sensitive frequency characteristic is indicated, while there is a number of distinct peaks in a frequency range of several cycles. Fig. 5 shows the resonance curve of Sakamoto Dam (Mie Prefecture, dome type, H=100 m), which was obtained from vibration tests carried out installing a shaker on the actual dam. Respective vibration modes corresponding to these resonance peaks are seen to appear. From an engineering standpoint, it is generally thought that a low-order mode is important, and symmetric first-order or asymmetric first-order modes are produced at fairly closely bunched frequencies.

Various measurements have also been made of the damping properties of arch dams, and generally, fairly small values of several percent to about 10 percent have been obtained.

(4) Fill Dams

The number of fill dams under earthquake observation is very large and amounts to more than 30. In spite of such a large number of sites undergoing observation, however, only a very small number of strong motion earthquakes have been recorded for dams compared with other structures in Japan. This may be attributable mainly to the fact that earthquake motions at dam sites are smaller than those at other types of ground because dam sites are usually located in mountains and their foundations are normally rock.

(A) Miboro Dam and Kuzuryu Dam[24),51)]

Miboro Dam is a rockfill dam with an inclined earth core and a height of 131 meters, located in the central region of Honshu, the main island of Japan. Its foundation consists of quartz porphyry and granite. Kuzuryu Dam is also a rockfill dam with an inclined earth core and height of 128 meters, located at the northwestern corner of the central region of Honshu. Its foundation consists of sandstone and slate. An earthquake occurred on September 9, 1969, shaking both dams. The epicentral distances of the earthquake from the two dam sites was nearly 40 kilometers for both, and both of the reservoirs happened to be full. The predominant frequencies observed at Miboro Dam were 1.5 Hz in the stream direction, and 2.0 Hz and 3.8 Hz in the direction of the dam axis. Those observed at Kuzuryu Dam were 2.0 Hz in the stream direction, and 2.4 Hz and 3.7 Hz in the direction of the dam axis. The predominant frequencies observed at the foundations of both sites were 1 to 2 Hz, 5 Hz, and nearly 10 Hz, and the component of the motion near the last frequency was fair-sized. Nevertheless, no component of motion of such high-range frequency was contained in the records of the earthquake motions observed at dam bodies. Such phenomena have often been recognized at a number of other dams. Therefore, it can be said that the components of the response motions of natural frequencies of the dam body are predominant during earthquakes although the components of shorter periods are predominant in the earthquake motions incident in the rock foundation of the dam. Moreover, it can be said from the above observation results that the natural periods of the dam body become shorter corresponding to the directions of vibration in such order as horizontal vibration in the direction of stream, horizontal vibration in the direction of dam axis, and vertical vibration.

(B) Kisenyama Dam[23)]

This is a rockfill dam with a central earth core, 95 meters in height, located in the southwestern part of Honshu. Its foundation consists of chert and slate. The maximum earthquake motion observed heretofore at the dam site was caused by the earthquake which occurred on September 9, 1969, the magnitude and epicentral distance from the dam site having been M=7.0 and Δ=145 kilometers, respectively. The maximum acceleration observed was 12.2 gal. At this dam site, other earthquake motions with maximum accelerations of 5.1 gal, 4.8 gal, and 4.3 gal were observed and analyzed.

A distinguishing feature of the response vibrations of this dam body is the abrupt increase in the response motions near the crest, such as the maximum response acceleration change from 12.2 gal at the base to 92 gal at the crest as shown in Fig. 6. The resonance curves were drawn up as the ratios of acceleration power spectra at the crest to those at the base. The natural frequency of 2.0 Hz for the vibration in the direction of the stream and damping constant of 0.15 were obtained with them. Field excitation

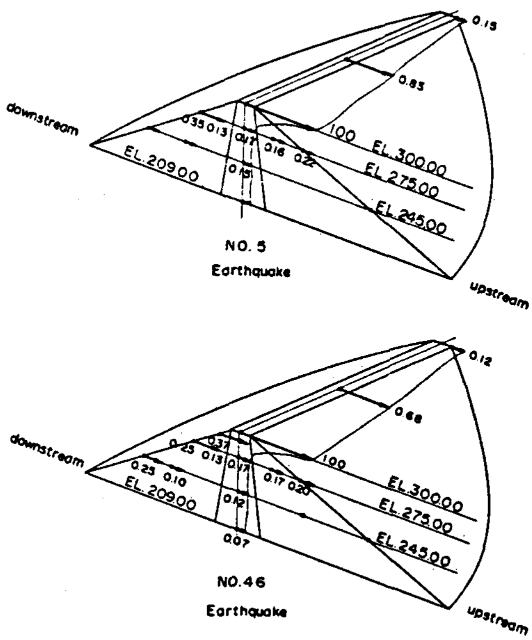

Fig. 6 Distribution of Response Accelerations at Observed Points in KISENYAMA Dam

Table 3 Natural Periods of Actual Fill Dams (after OKAMOTO)

Name	Type	Height	Natural Period [sec]			Remarks
			Perpendicular to Dam Axis	Parallel to Dam Axis	Vertical Direction	
Sannokai	Earth fill	37 m	large 0.42			Observation of Earthquake
			small 0.35	0.34	0.25	
Ainono	Earth fill	41	0.40			h=0.13~0.16
Nagano	Rock fill	128	0.44	0.45		Observation of Earthquake
Miboro	Rock fill	135	0.37	0.26		Observation of Earthquake
			0.42			
			0.59	0.57		
Yanase	Rock fill	115	0.42	0.42		Observation of Earthquake
			0.23			
Kisenyama	Rock fill	95	large 0.50	large 0.45	large 0.35	Observation of Earthquake
			small 0.37	small 0.40	small 0.26	
Bouquet Canyon	Earth fill	62	0.45			Vibration Test (nuclear explosion test)
			0.37			
			0.32			
Makio	Rock fill	85	0.38			Vibration Test
Tougo	Earth fill	31	0.4			Vibration Test

tests were carried out on this dam with an exciter of eccentric mass type, and natural frequency of 2.0 Hz and damping constant of 0.06 were obtained.

(C) Sannokai Dam, Ainono Dam, and Ushino Dam[19),20),22)]

These are earth dams close to 40 meters in height and located at the northeastern part of Japan. Through three-dimensional accelerograms obtained at the dam crests in several earthquakes it was confirmed that the dam body shows three-dimensional behaviors during earthquakes such as motion in the direction of the dam axis and motion in the vertical direction as well as shearing vibration, and that the predominant period in each direction is longest in the horizontal vibration along the stream, and shortest in the vertical direction, while the predominant period in the vibration along the dam axis coincides with either of the above periods. As results of the above observation of earthquakes, other facts were recognized as described below. The distinctive qualities of the spectra of earthquake motions recorded on a dam body are always constant and peculiar to the dam although those of eatthquake motions recorded at the dam base are different for every earthquake. The natural periods of the dam body have a tendency to increase as the maximum acceleration of earthquake motion increasess to exceed about 30 gal and vary distinctively according to the height of the dam independent of structure type if only the shape of the cross section is similar. The seismic response amplitude at the dam crest becomes larger in the order of horizontal vibration in the stream direction, horizontal vibration in the direction of the dam axis, and vertical vibration, respectively, similarly to the predominant period of the rockfill dam. The horizontal vibration in the stream direction increases at the crest and mid-height of the dam body. However, the rate of increase becomes lower as the earthquake motion increases because of the nonlinear material properties of the dam body.

S. Okamoto showed the observed natural periods of several fill dams as tabulated in Table 3. Also, he reported that the values of damping constants obtained from seismological observations on many fill dams were considerably large, being such as 0.13 to 0.16. These values were obtained from very small earthquake motions. Therefore, the damping in the seismic response motion of the dam body cannot be considered to come from friction of materials, and may be assumed to originate mainly in enetgy dissipation underground.

5. EARTHQUAKE-RESISTANT DESIGN METHOD FOR DAMS

(1) Fundamental Design to Determine Dimensions of Dams

The calculation method for earthquake resistance generally followed in design of dams is the so-called seismic coefficient method in which the weights of the dam body itself and a part of the reservoir water determined by the formula of dynamic water pressure are multiplied by the seismic coefficient, and the value obtained is treated as earthquake force. These forces of inertia are applied horizontally to the dam body to calculate its stresses and stability. This method has been employed in Japan from the time high dams began to be constructed. As time elapsed, improvements were made in the method, and with developments in research and studies of earthquake phenomena and the resistive properties of dams to earthquakes, the seismic coefficient taken into account in design of dams has come at present to be determined by various factors, such as, the type of dam, geological conditions, and occurrence of serious earthquakes in the past at the site where the dam is to be constructed. Table 4 is the design seismic coefficient applicable to the foundation of a dam given in the design criteria for dams established by Ministry of Construction. The values are classified by types of dams and the zones shown in Fig. 7 in which the dams are to be constructed. One half of the values shown in Table 4 may be taken when the reser-
values are classified by types of dams and the zones shown in Fig. 7 in which the dams are to be constructed. One half of the values shown in Table 4 may be taken when the reser-
voir is empty because under this condition the damage caused by earthquake would not be serious.

In case of an arch dam, the value of the seismic coefficient of the dam body must be more than two times the value applied to the dam foundation because the seismic load causes elastic vibration with some deflection on the dam body and the acceleration of the dam body is amplified as compared with that of the foundation.

Under earthquake load, the allowable stresses of a gravity dam as well as those of an arch dam may be increased by 30% of the value specified for static load.

In all of very high dams, the abovementioned amplifying effect is taken into consideration. Moreover, the seismic coefficient method is officially under reviewing its applicability.

Table 4 **Design Seismic Coefficient for Dam Body (Concrete Dam) and the one applicable to Foundation of Dam (Fill Dam)**

Type of Dam	Strong Zone A	Medium Zone B	Weak Zone C
Gravity & Hollow Gravity Dams	0.12~0.15	0.12	0.10~0.12
Arch Dam	0.24~0.30	0.24	0.20~0.24
Fill Dam	0.18	0.16	0.13

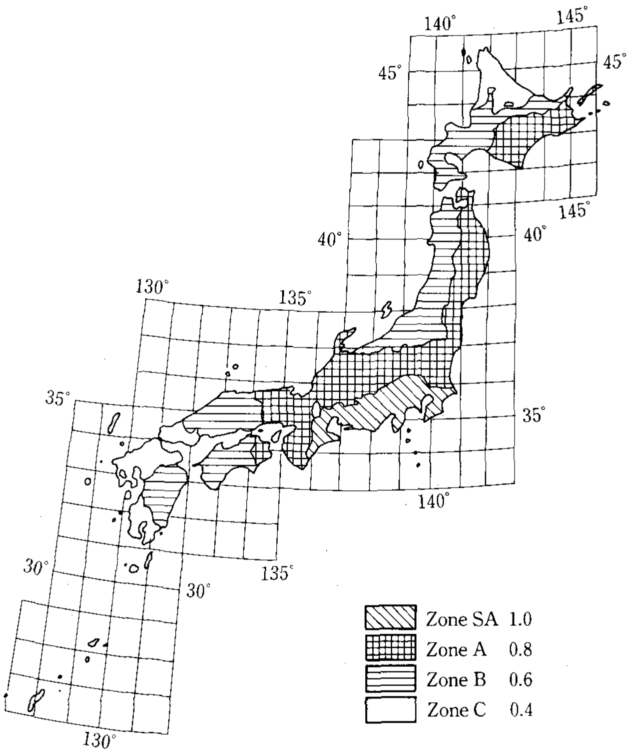

Fig. 7 Zoning of Design Seismic Coefficient for Dam Foundations

(2) Design by Modified Seismic Coefficient Method for Fill Dam

For seismic design of fill dams, new design guidelines have been prepared by Japan Institute of Construction Engineering under supervision of Ministry of Construction. The important points of it is as follows.

(a) Design Dam-Body Seismic-Force Coefficient

According to responses of dams during strong motion earthquakes, design dam-body seismic-force coefficient k is introduced and takes the place of the conventional seismic coefficient as follows.

$$0 < y/H \leqq 0.4 \ : \ k = k_F \times \{2.5 - 1.85 \times (y/H)\}$$

$$0.4 < y/H \leqq 1.0 \ : \ k = k_F \times \{2.0 - 0.60 \times (y/H)\}$$

where H is the height of dam, y is the length from dam crest to the lowest point of circular slip plane and k_F is the design seismic coefficient applicable to the foundation as shown in Table 4.

(b) Strength of Materials

Stability check of a dam-body should be carried on by the slice method on assumed circular slip planes determined by the conventional concept.

Conventional formula of soil materials is usable only for cohesive ones of impervious core. For cohesionless materials such as filter and shell other formulae are introduced as follows.

(1) For cohesive materials (conventional)

$\tau_f=(c+\sigma_n' \tan \phi)$

(2) For cohesionless materials (newly introduced)

$\tau_f=A(\sigma_n')^b$ or

$\tau_f=\sigma_n' \tan \phi_0$

where

τ_f: shearing strength of material

c, ϕ: constants of strength of material; cohesion and angle of friction, respectively

σ_n': mean effective principal stress in slip plane of every slice,

$\sigma_n'=\sigma_n-u$

here, σ_n: total stress

u: excessive pore water pressure

ϕ_0: angle of friction of cohesionless material, of which confined pressure reliance is as follows

$\phi_0=\phi_{\max}-a\cdot\log(\sigma_n'/\sigma_0')$; $(\sigma_n'>\sigma_0')$

$\phi_0=\phi_{\max}$; $(\sigma_n'\leqq\sigma_0')$

here, $\phi_{\max}$: maximum value of angle of friction at small confined pressure (defined from material tests)

a: coefficient of reduction rate of angle of friction as confined pressure increases (defined from material tests)

σ_0': critical stress under which angle of friction should be kept constant at maximum value of $\phi_{\max}$

(c) Factor of Safety

Factor of safety obtained from above stability check must be more than 1.2.

(d) Range of the Application of New Guideline

The new design guideline is applicable for fill dams of both zone type and homogeneous type under 100 meter high. In facing type dams the provision shall be applied correspondingly.

The stability of fill dams more than 100 meter high shall be checked by a method of seismic response analysis (dynamic analysis).

(3) Design by Dynamic Analysis

In the cases of dams of large scales, dams of high degrees of importance, dams of new types with no performance records of construction, and dams to be built in areas of high seismicities, safety is confirmed by means of dynamic analyses and model vibration tests in addition to fundamental design based on the pseudo-static method.

In carrying out a dynamic analysis the concrete details of the four items of earthquake input, physical properties, analysis method, and method of evaluating analysis results are determined as described below. Further, it is considered an important precondition for initial stress due to gravitational acceleration to be properly evaluated.

(A) Earthquake Input[35]

For the earthquake input used in dynamic analysis the strongest earthquake of those considered appropriate to be predicted as occurring in the vicinity of the dam site is taken. That is, in the case of Japan, the historical earthquake group extracted from the earthquake list is divided into submarine type (collosal earthquake) and land type, and the past maximum earthquake motion for each of the earthquake group classifications applied to the site is adopted.

Next, earthquake engineering knowledge presently available is utilized to the maximum extent, and the maximum value of earthquake motion, predominant period of acceleration, and duration are set. Further, investigations of earthquake damage hisroties are given overall consideration and the waveform of the design earthquake motion is determined applying engineering judgment. In concrete terms, two or three acceleration records of ground having charactetistics similar to those of the dam site are selected from among records of relatively strong earthquake motions, and a waveform with parameters such as maximum value of earthquake motion modified to fit the above values, or an artificial earthquake waveform prepared to fit the earthquake characteristics of the dam site is employed.

(B) Physical Properties[15),37)–43]

Physical properties broadly divided are those related to strength and those related to deformation. As the dynamic strength of concrete, strain velocity dependency is considered, and the standard test value increased by 30% is employed as the reference value. It is normal for dynamic strength of bedrock to be substituted with a static strength constant determined by block shear tests performed in the field. A strain velocity dependency should exist for the bedrock also, but there are extremely few cases of dynamic strengths having been measured, and the actual state is therefore unknown, so that to be on the conservative side, a static strength constant is used. The dynamic strengths of fill dam materials are being obtained by dynamic triaxial compression tests and totsional shear tests, and it is normal for the maximum stress reaching a certain cumulative strain quantity on applying a given number of loading cycles to the specimen to be larger than the static strength. However, since the present situation is that a clear-cut definition cannot be given to dynamic strength, static strength is substituted for dynamic strength to be on the conservative side.

The elastic modulus of concrete is in the range of 3.0×10^5 to 4.0×10^5 (kgf/cm^2) within the scope of allowable stress intensity. The dynamic modulus of elasticity is 20 to 30% higher than these values at around 10 and several Hertz. For the modulus of elasticity used in the fundamental design, 2.0×10^5 to 3.0×10^5 (kgf/cm^2) is employed considering creep since the principal loads of water pressure and dead weight are long-term sustained loads. Earthquake loads are of extremely short term and it is reasonable to use a dynamic modulus of elasticity obtained through experiments. There are cases when strain velocity dependency is considered and the dynamic modulus of elasticity of concrete is expressed by a four-element visco-elastic body. For the damping constant, a value of around 10% is employed considering energy losses at joints between concrete blocks and underground energy dissipation.

For the modulus of elasticity or deformation modulus of the bedrock, the static modulus of elasticity obtained by the jacking method or water chamber method is used in the fundamental design. Regarding the dynamic modulus of elasticity, what is done is to apply the results of the elastic wave method to the static modulus to determine the former. However, at hard bedrock it is elastic until the rock is broken, and with rock such as mudstone there is elasticity almost completely up to shear strain amplitude of 10^{-3}, so that it may be said the trend is for a value obtained from elastic waves to be used for the dynamic modulus of elasticity.

It is normal for the dynamic moduli of deformation and damping constants of fill dam materials to be expressed by equivalent linearization based on the results of dynamic triaxial compression tests and totsional shear tests, and for the respective strain amplitude dependency to be formularized by hyperbolic models and introduced in calculations. Detailed elastic wave velocity measurements have been made on a large number of fill dams in Japan, and accurate general velocity distribution models have been prepared in the forms of functions of depths from dam surfaces divided into rockfill dams and earth dams. Based on these, it is possible for the shear modulus in case the strain amplitude is extremely small to be set as an in-situ value, and it may be said that the precision of the constructive mood of design concerning dynamic modulus of deformation has been greatly improved. In case a fill dam is built on bedrock which is comparatively hard, the difference in velocity impedance between dam body and foundation will be marked, and the analytical base is often taken to be at the surface of the bedrock. In such case the underground dissipation of vibration enetgy of the dam body is considered by means of damping constant, and based on the theoretical and observed values a given damping constant value is added to the abovementioned experimental value in a range of 5 to 10%.

(C) Analytical Method[44)–47)]

Seismic response analyses of dams based on the shear wedge theory were performed in Japan as early as 1934 by M. Matsumura with earth dams as the objects. Today, numerical analysis methods using electronic calculators have become the mainstream, and in studies at the very beginning, an analysis was carried out by H. Ishizaki and N. Hatakeyama of a two-dimensional equation of motion of an earth dam by the finite differential method. It was clarified by this study that an earth dam shows not only shear vibtation but also stretching vibration, and this served as a guide to subsequent development of seismic response analysis methods.

At present, with the advent of FEM, seismic response analyses of dams in the form of numerical analyses have become established, and it has also become possible for three-dimensional analysis, nonlinear analysis, and further, analyses of discontinuous bodies and coupled vibrations of dams and reservoirs to be performed.

With gravity dams and arch dams, three-dimensional analyses including reservoirs and bedrock should be applied to design calculations, and even though handling of infinite boundaries has been introduced in the boundary element method, this still has not reached an adequately reliable stage, and neither can it be said to be at a practical stage from the aspect of economy. Therefore, an analysis method simplified by providing a hypothesis on the conservative side is being used in design. Since for fill dams it has been confirmed that two-dimensional analysis gives a more conservative response than three-dimensional analysis, detailed stress analyses are made by two-dimensional seismic response analysis. In case it is necessary to investigate the complexity of the topography at a dam site or the three-dimensional response characteristics of a fill dam with impervious facing, it is normal for studies to be made by model vibration tests.

Various seismic response analysis programs of contents described above have been developed in Japan also, and in two-dimensional analysis programs for fill dams, verifications by comparisons with results of earthquake observations of actual dams and results of model vibration tests are also made, while equivalent natures by comparative calculations between various programs of different analytical methods have also been verified, and the reliabilities of analytical methods have improved greatly.

(D) Method of Evaluating Analysis Results

The technique of evaluating the stability of a dam from the results of seismic response analysis differs according to the type of the dam. With a concrete dam the degree of tensile stresses is studied and the allowable level is judged within limits that cracking does not reach through the dam body. With a fill dam, the distribution of mobilized planes at each arbitrary instant during earthquake response is calculated, and the potential slip surface is obtained connecting the directions of mobilized planes in a range that the local safety factor drops very close to 1, and when this points outward from the dam body, it is judged to be momentarily unstable. However, if the duration of the unstable condition is short it is considered that sliding will not be produced in the dam body, and in order to make a quantitative evaluation, the method employing as a yardstick the sliding deformation quantity obtained with the dam body reaction resisting the unstable soil mass taken as the integrated quantity of the relative acceleration between soil mass and dam body, or a method employing as a yardstick the cumulative deformation quantity of dam body materials accompanying repetitive stresses during earthquake, etc., have been proposed.

6. RESEARCH ACTIVITIES

In any event, since a generally accepted method has not yet been established regarding a stability evaluation method, application to design is being attempted evaluating the abovementioned method on the conservative side.

Initial research activities in Japan regarding dams were concerned with concrete gravity dams, and a convenient method for earthquake-resistant design of concrete gravity dams was proposed in 1934 by N. Mononobe[1]. This method treated all forces due to earthquakes acting on dams as static loads, and it has been adopted as the standard earthquake-resistant design method for concrete gravity and earth dams in Japan. Several studies on dynamic behaviors of concrete gravity and earth dams were subsequently carried out[2)–7)].

At the time of designing Kamishiiba Dam, the fitst large arch dam in Japan completed in 1955, dynamic properties of the ground at the site and the dam were investigated, and since then, studies on the earthquake-resistant design of arch dams came to be actively conducted.

Lately, a number of very large rockfill dams has been constructed or planned in Japan and some of them are for pumped-storage power plants. With such a situation, research on earthquake resistance of fill-type dams has come to be carried out extensively[12),13)].

As for dynamic properties of dam materials, concrete and earth materials have been tested in laboratories at the design stages of high dams, while further dynamic properties of rock materials are also being studied by means of dynamic triaxial tests[14),15)]. The test results are applied to theoretical dynamic analyses of rockfill dams, which are being actively promoted nowadays. This contributes toward making dynamic analyses of the dams more accurate.

To describe the abovementioned matters further in detail, research on dynamic problems of dams is divided into research on analyses of dynamic responses of dams in earthquakes and research on the dynamic properties of dam materials.

As some examples of studies on the analyses of dynamic responses of dams in earthquakes, descriptions of numerical dynamic analyses of arch dams and fill dams are given in the following paragraphs.

With regard to arch dams, an approximate solution of initial value problems of partial differential equations by means of the finite difference method has been developed. This solution is a quite general three-dimensional one taking into account the shape of the valley and also the effect of coupled vibration of the dam and the reservoir[11),54)].

The shear vibration theory gives appropriate solutions for the fundamental vibrations of fill dams, and even at present, it is still an important method for earthquake-resistant design of fill dams.

More generally, a method solving the initial value problems of partial differential equations by means of the finite difference method, assuming the dam materials to be viscoelastic, has been developed for the response analyses of homogeneous dams with slight nonlinearities.

Currently, the most general method is the finite element method giving an equivalent linear solution for a nonlinear one. This method can be applied to zone type dams having systems of internal zones ranging from simple to intricate. This is a method for solving the vibration problem by an iterative procedure renewing the properties of materials for each time step[12),13)].

A three-dimensional response analysis by means of a mode superposition method using FEM has been recently introduced.

As an example of studies on the dynamic properties of dam materiais, a description of the present feature and the trend of studies on materials for fill dams is given below.

A number of detailed studies is being carried out on the effects of period and amplitude of load of sinusoidal oscillation upon the stress-strain relationship, viscosity, Poisson's ratio and other properties of soil materials[15)]. Furthermore, tests applying the load of random oscillation, besides sinusoidal oscillation, have become possible to perform. In addition, dynamic triaixial compression tests with phase differences between principal and lateral loads are about to be started. Studies on liquefaction of sand materials under vibration load have also been undertaken.

In parallel with these systematic studies at institutions, several tests using large specimens have been carried out for some actual dam projects in order to investigate the dynamic properties of dams.

Meanwhile, research works not only through laboratory studies, but also field observations, have been extensively carried out[16)–25)].

Field experiments on several dams of various types have already been conducted by means of vibrating machines, by which the vibration characteristics of the dams have been brought to light[26)–28)]. In addition, earthquake responses of dams have also been disclosed through observations of natural earthquakes.

As a recent trend of earthquake observation of fill dams, seismographs are being installed not only on the surfaces but also inside the dam bodies, and further, not only accelerations, but also displacements are observed.

One successful observation of dynamic water pressure during earthquake has been made at Tagokura Dam, where several dynamic water-pressure meters were installed at various heights on the upstream face of the dam along with seismographs[17)].

The number of dams on which observations of earthquake response have been conducted has recently increased, and Table 5 gives a list of those dams.

Research activities for earthquakeresistant design employing model experiments have made remarkable progress lately[29),30),54)–56)]. Large-capacity vibration apparatus have been made and their performances greatly improved[31)] (Table 6). Several types of driving mechanisms for shaking tables have been developed, namely, mechanical type, electromagnetic type, and electro-hydraulic type. Some shaking tables have capacities of more than 100 tons. Random vibration tests as well as sinusoidal tests have become possible with some of these shaking tables. In addition to tests of elastic behavior, those

Table 5 Seismographs Installed in Japanese Dams

Name of Dam	Type	Height (m)	Year of Completion	Type of Seismograph	
				Strong Motion	Electro-magnetic
Sannokai	TE	37.0	1954		○
Tonoyama	VA	64.5	1955	○	
Fujiwara	PG	94.5	1957	○	
Ikawa	EHG	103.6	1957		○
Narugo	VA	94.5	1957		○
Ogochi	PG	149.0	1957		○
Sarutani	PG	74.0	1957	○	○
Sasogawa	PG	76.0	1957		○
Sasanamigawa	VA	67.4	1959		○
Ayakita	VA	75.3	1960		○
Miboro	ER	131.0	1960		○
Tagokura	PG	145.0	1960		○
Arimine	PG	140.0	1961		○
Muromaki	VA	80.5	1961	○	○
Futatsuno	VA	76.0	1962		○
Hitotsuse	VA	130.0	1963		○
Minase	ER, PG	65.0	1963		○
Yokoyama	EHG	80.0	1963	○	○
Amagase	VA	72.0	1964	○	○
Ikehara	VA	110.0	1964		○
Kurobe No. 4	VA	186.0	1964	○	○
Yamakura	TE	22.5	1964		○
Saikawa	PG	72.0	1965	○	
Tsuruta	PG	118.0	1965	○	
Yanase	ER	115.0	1965		○
Yuda	VA	87.5	1965		○
Isaka	TE	34.4	1966		○
Kawamata	VA	120.0	1966		○
Kanayama	PG	57.3	1967	○	
Inekoki	VA	60.0	1968		○
Kuzuryu	ER	128.0	1968	○	
Shijushida	PG, ER	50.0	1968	○	○
Kisenyama	ER	91.0	1969		○
Koshibu	VA	105.0	1969	○	
Midono	VA	95.5	1969		○
Misakubo	ER	105.0	1969	○	
Nagawado	VA	155.0	1969	○	○
Shimouke	VA	98.0	1969	○	
Susobana	VA	83.0	1969	○	
Higashifuji	TE	22.0	1971		○
Shin-konoyama	ER	33.0	1971		○
Yahagi	VA	100.0	1971	○	
Kimigano	PG	73.0	1972	○	
Miyama	ER	76.0	1972		○
Numappara	ER	38.0	1972		○
Shintoyone	VA	117.0	1972		○
Shimokotori	ER	119.0	1973		○
Tataragi	ER	65.0	1974		○
Kurokawa	ER	98.0	1974		○
Seto	ER	111.0	1978		○
Asahi	VA	86.0	1978		○
Miura	PG	86.0	1973		○
Niikappu	ER	103.0	1974		○
Iwaya	ER	128.0	1976		○

Table 5 (continued)

Name of Dam	Type	Height (m)	Year of Completion	Type of Seismograph	
				Strong Motion	Electro-magnetic
Takase	ER	176.0	1978	○	○
Nanakura	ER	125.0	1978		○
Tamahara	ER	116.0	U.C.		○
Nabara	ER	85.0	1975		○
Aburadani	ER	82.0	1976		○
Tedori	ER	153.0	1979		○
Ohishi	PG	87.0	1978	○	
Managawa	VA	128.0	1978	○	
Tarumizu	ER	43.0	1976	○	
Miho	ER	95.0	1973	○	
Terauchi	ER	82.0	1978	○	
Ohkawa	PG	78.0	U.C.	○	
Urushizawa	ER	80.0	1980	○	
Shirakawa	ER	66.0	U.C.	○	○
Kamafusa	PG	45.0	1970		
Fujiwara	PG	95.0	1958		○
Daisetsu	ER	92.0	1975		○
Hoheikyo	VA	103.0	1972		○
Abugawa	VA	95.0	1975		○
Uchinokura	EHG	83.0	1972		○
Shiroyama	PG	75.0	1965		○
Koori	TE	38.0	1973		○
Haginari	PG	59.0	1966	○	
Hirose	ER	75.0	1975		○
Muromaki	VA	81.0	1961	○	○
Ohuchi	ER	102.0	U.C.		○

Note: Type of dam: VA—arch, PG—gravity, ER—rockfill, TE—earth

Table 6 Large Shaking Tables in Japan

Owner	Type of Vibration	Capacity (ton)	Size of Table	Waveform
National Research Center for Disaster Prevention	Electro-hydraulic	450	16 m×16 m	Sinusoidal, random earthquake
Central Research Institute of Electric Power Industry	〃	120	6 m×6.5 m	〃
Ministry of Construction	〃	40	4 m×4 m	〃
Institute of Industrial Science, University of Tokyo	〃	170	2 m×10 m	〃
Electric Power Development Co.	Mechanical	22	5 m×5 m	Sinusoidal

in the nonlinear region have been carried out by means of these apparatus. Moreover, rupture tests have also been undertaken[32].

The results of studies on numerical response analyses, on dynamic properties of dam materials, and on structural behaviors by means of model tests have been investigated, being compared with the observation records at a number of actual dams. These studies have contributed to the progress of earthquake-resistant design of dams in Japan.

In 1975, the Committee on Earthquakes of the International Commission on Large Dams recommended important items that should be intensively studied[33]. These items, given in "Present Recommendations," are consideration of seismic stability, use of dynamic analyses, improvements in earthquake-resistant design, stability of slopes at dam sites, and investigation of failure mechanisms and failure propagation. Complying with the above-mentioned "Recommendation," many studies concerned with seismic stability of dams have been carried out placing emphasis on fill dams. In these studies, 'dynamic analysis' is considered to be a system consisting of four definite items, namely, input earthquake motion, dynamic material properties, dynamic analysis procedures and evaluation of stability of slope.

Regarding the first item, a detailed survey of historical earthquake records has been made at many dam sites by the Public Works Research Iustitute[34] and design earthquake motions which must be taken into account have been pointed out. At the same time, an investigation on design earthquake motions on dams has been carried out with an interim report presented by the Japanese National Committee on Large Dams[35),36]. Design earthquake motions have come to be determined rationally based on the results of the above.

As for the second item[37)–43], many laboratory tests, such as dynamic triaxial compression tests have been carried out upon all kinds of materials for rockfill dams. The static and dynamic shear strengths under vertical strong accelerations have also been investigated. Through these tests, features of dynamic strength as well as dynamic moduli of granular materials have been revealed. With these results, it may be said generally that dynamic strengths of almost all kinds of materials for rockfill dams are larger than static ones. Therefore, if the design parameters of the strengths of materials such as c and ϕ are determined based on static tests, they would be on the conservative side.

Besides laboratory tests, experimental in-situ vibration tests have been performed on five rockfill dams and a fairly general model of velocity distribution in rockfill dams has been proposed. Its validity has been confirmed by comparing natural frequencies between calculated values of the velocity model and observation results from five other rockfill dams.

A so-called hyperbolic model was established taking into due consideration both the results of laboratory tests and experimental in-situ vibration tests. The design values of dynamic properties of materials such as shear modulus and damping constant can be formulated well by this model.

The third item may be classified according to two kinds of procedures: numerical analysis and model test. Regarding numerical analysis[44], three kinds of algorithms have been proposed in integration of the equation of motion, namely, mode superposition method, step-by-step integration method, and fast Fourier Transform method. In Japan, FEM is generally adopted as a useful method.

In connection with the mode superposition method, three-dimensional dynamic analyses have been applied to a number of rockfill dams[45),52),57),58]. An algorithm for non-elastic two-dimensional dynamic analyses of fill dams has been developed and applied to the design of many rockfill dams for pumpedstorage power plants. Its validity has been verified through both model vibration tests and observation of earthquake motions at an actual rockfill dam[46),49),50].

As for model vibration tests, a large number has been carried out by Tokyo Electric Power Co., Inc., the Electric Power Development Co., Ltd., the University of Tokyo, and the Central Research Institute of Electric Power Industry. With these results, the behaviors of fill dams subjected to earthquakes[47] have been clarified and the mechanism of seismic failure estimated.

Concerning the fourth item, several studies are now being carried out[48],[53],[59],[60], and will be completed in the very near future. These studies would contribute greatly to the progress of earthquake-resistant design of dams.

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