DESIGN SEISMIC COEFFICIENT IN SEISMIC COEFFICIENT METHOD AND ACTUAL EARTHQUAKE DAMAGE

by

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SUMMARY

The earthquake resistance actually possessed by a structure differs according to factors such as country, time of construction, design method and construction method and so indiscriminate conclusion cannot be drawn regarding the matter. However, it is shown from a macroscopic viewpoint based on behaviors during earthquakes that bridges and concrete gravity dams are capable of withstanding fairly great response accelerations compared with maximum accelerations simply calculated from design seismic coefficients. Useful data for estimation of earthquake resistance of actual structures have been presented here.

1. INTRODUCTION

Clarifying the relationship between earthquake resistance of a structure given as a condition in earthquake-resistant design and the earthquake resistance the structure possesses against an actual earthquake is a matter of considerable importance. This relationship is closely connected not only with conditions for design such as the character of the earthquake motion, foundation conditions, mechanical properties of the materials of the structure, design seismic coefficient, and allowable stress, but also the method of construction and the technical level of work execution. Consequently, this relationship will differ depending on the design of the structure, the era of construction, the country, and even the district. Clarification of this relationship is urgently required at this time when dynamic analyses are being made and earthquake-resistant design is being widely used.

This paper mainly takes up the Miyagi-ken-oki Earthquake of 1978 which caused much damage to facilities and structures designed based on earthquake-resistant design methods, and describes the relations of damage, earthquake motions and design seismic coefficients in regard to bridges and dams.

2. MAXIMUM ACCELERATIONS OF EARTHQUAKE MOTIONS

It is well known that the maximum acceleration of earthquake motion is not necessarily the direct cause of damage of a structure. However, the

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maximum acceleration; taking into account the earthquake magnitude, place of occurance, epicentral distance and ground conditions, but ignoring the predominant short period impulse, if there be any, may be taken as an important yardstick for indicating the intensity of an earthquake.

Fig. 1 shows the relations between maximum accelerations in the horizontal direction at bedrock, base rock, ground surface and underground obtained during the Miyagi-ken-oki Earthquake and the distances from the center of the aftershock area. The particulars of this earthquake are the following:

Time of occurrence:
17hr 14m 25.4 sec (JST), Jun 12, 1978
Magnitude: 7.4
Epicenter: 142°10'±1'E, 38°9'±1'N
Depth of hypocenter: 40 km
Aftershock area center assumed as 142°E, 38°10'N

In Fig. 1, numerals with • marks are numbers of measuring points, 1 to 5 being at bedrock, 6 at weathered bedrock, 27 at foundation ground, and 29 at bedrock 67.2 m underground. Among these measuring points, the maximum acceleration at 5 was 240 gal, whereas the second highest was approximately 130 gal, and only the maximum value is especially predominant. This trend is seen at 22 also, where the second highest value is 250 gal. The straight lines are the relations between the maximum accelerations and epicentral distances obtained in earthquake observations being carried out by the authors at an underground location in bedrock approximately 120 km north of Tokyo expressed with magnitude as the parameter. According to Fig.1, with 5 being an exception, the values at bedrock or foundation ground are seen to indicate the lower-limit values of maximum accelerations corresponding to the distances from the aftershock area center. The numbers 20 and 21 in Fig. 1 are measuring points at the ground floor and the second basement of a building on a diluvial table in the city of Sendai with 240 gal and 253 gal, respectively. The number 7 is for a measuring point at the ground surface in a case of hard rock existing 2 to 3 m underground. It is estimated that on diluvial ground around Sendai approximately 100 km from the aftershock area center, maximum accelerations of about 250 to 300 gal were reached. During the earthquake of M = 6.7 which occurred on February 20, 1978 several tens of kilometers north of this earthquake, it was seen that the lower limits of maximum accelerations were similarly expressed by straight lines.

Fig. 2 shows response acceleration spectra (according to the Public Works Research Institute, Ministry of Construction) calculated from acceleration records (Tarumizu Dam) obtained at measuring point 5. In case of damping coefficients of 0.1 to 0.2, taking into consideration high-order vibrations also, it may be seen that the maximum accelerations will reach roughly 350 to 500 gal at the surfaces of surface layers having predominant vibration periods of 0.4 to 1.0 sec.

Fig. 3 is an example of distribution of acceleration estimated from surveys of gravestones toppled in the earthquake. Points of 250 to 300 gal are large in number in districts about 100 km distant from the aftershock area center, and these roughly agree with the previously-mentioned values of maximum accelerations at diluvial ground. In districts such as the vicinity of the mouth of the Natori River where there was heavy damage to bridges, levees and houses, it is seen that there is a correspondence with the high accelerations estimated from overturning of gravestones — 400 to 450 gal or even higher. In general, it is known that heavy damage occurred to structures in areas of alluvial ground overlying diluvial tableland.

3. DAMAGE TO STRUCTURES AND DESIGN SEISMIC COEFFICIENTS

In earthquake-resistant design the seismic coefficient method is generally employed for such reasons as simplicity in use, and the earthquake resistances of structures are considered by design seismic coefficient k. However, k is closely related with kind of structure, type, size, design method, allowable stress, etc., and the earthquake resistance of a structure is not something that is decided solely by high or low value of k. Knowledge about the relations between the earthquake-resisting strengths and k's of various types of structures is important for carrying out rational earthquake-resistant design. The relation between damage and k for two kinds of structures is examined from such a viewpoint.

A. Bridges There apparently have been few cases in which so many large bridges designed by earthquake-resistant methods were subjected to strong earthquake motions and damaged as in the Miyagi-ken-oki Earthquake. There are bridges among these which were designed for earthquake resistance based on the most recent standards. On overall observations of damage, in addition to settling of approaches, failure of shoes, damage to embedded portions of anchor bolts, and inclinations of bridge abutments of girders, buckling of lateral-structure and sway bracing members, and ruptures of chord members in superstructures, at substructures, cracking and tilting of piers occurred to indicate that strong earthquake motions had been sustained.

Major cases of bridge damage are listed in Table 1 with the locations of damaged bridges shown in Fig. 4. The numbers in Fig. 4 correspond to the bridge numbers in Table 1. These bridges had been designed for earthquake resistance by the seismic coefficient method, and except for two bridges which had been completed very recently, the values of k were $k_H = 0.2$ with k_V one half of k_H. At the four exceptions 0.22, 0.24 and 0,25 had been taken for k_H because of ground conditions and height of piers.

The damage to bridge pier bodies in the table consisted of roughly horizontal or slightly diagonal cracks thought to be due to shearing and bending deformation, buckling of main reinforcement and tilting in case of massive reinforced concrete bridge pier bodies, cracking in case of relatively flexible rigid structures and wall-type structures, and movement and crushing at construction joints of concrete in case of nonreinforced concrete bridge piers. Foundations were pile foundations, and well or caisson foundations reaching down to bearing layers through soft ground of thickness of ten and several meters.

At some of steel bridges buckling of members such as cross beams and sway bracing occurred near supports, while at bearing parts breaking of joints between web plates and cover plates, buckling of web plates and twisting deformation of cover plates were seen, and pins of shoes also failed. At one steel through truss bridge there were breakages of top chord members, and judging by the condition of the ruptured surfaces, it is surmised that fatigue had been fairly advanced.

At reinforced concrete and prestressed concrete bridges there were many cases of shoes breaking and girders moving while there was a number of bridges where cracks occurred near fixed bearings at ends of girders.

In both reinforced concrete and steel bridges, it appears that either damage consisting of failure of fixed bearings, pull-out of anchor bolts, and breaking of concrete around bolts, or damage to girders near fixed bearings occurred in most cases.

The conditions of damage described above indicate that very great earthquake forces acted on the superstructures of all of these bridges.

According to earthquake records, a maximum horizontal acceleration of 475 gal was registered on the bridge piers of Bridge 15 approximately 140 km distant from the aftershock area center. At Kaihoku Bridge at a point about 70 km distant, the acceleration was more than 500 gal and ran off the recording paper. With the former, the web plates and cover plates of steel girders near fixed bearings were deformed. There was no damage with the latter where oil dampers had been provided at bearing portions.

As previously described, bridges which sufferred damage were all constructed at diluvial ground and according to investigations of gravestone damage, it is estimated that accelerations of 350 gal to 400 to 450 gal occurred at the ground surface. Consequently, as seen in earthquake observations, it may be estimated that maximum accelerations at superstructures reached about 500 gal or even more. On the other hand, in case of bridges built on diluvial ground, there was no damage other than settlement of approaches and buckling of handrails at joints.

At the time of the Izu-hanto-oki Earthquake (M = 6.9) in 1974, at Tengu Bridge (two-span continuous deck bridge, length 128.5 m, pier height 40 m) and Isuzu Ohashi Bridge (three-span, continuous-curve plate girder, length 105.0 m, pier heights 31.0 m and 34.0 m) were both cases of damage only of the degree of breaking of attachment bolts for upper shoes and girders, and damage to the connectors for preventing falling of girders and abutment walls. According to the estimates of the authors, the fundamental vibration periods of the bridges were about 0.4 to 0.5 sec, and it is thought there was acceleration in excess of 700 gal at the tops of bridge piers during the earthquake. The k_H values used in earthquake-resistant design of these bridges were 0.415 (Tengu) and 0.345 (Isuzu) at pier tops with extra allowances made in view of the heights of the piers.

Based on the above, it may be considered that real damage to the functions of a bridge will not occur even though response accelerations double or even more than double the acceleration calculated from k_H occur at the superstructure. However, it will be necessary to study this further upon collecting data which can be used for accurate dynamic analyses.

B. Concrete Gravity Dams Regarding concrete gravity dams, an extremely small number of cases have been reported of cracks being formed in the bodies of old dams of heights about 10 m which happened to be in epicentral areas. Among high dams designed by modern methods, it is thought Koyna Dam is the only one that suffered damage to its body. However, this dam does not possess a customary cross-sectional shape for reasons of circumstances during construction, and moreover, it was located near the epicenter of an earthquake of M = 7.

In Japan, concrete gravity dams have been designed by the seismic coefficient method since 1925, and although k_H varies according to the district, 0.1 to 0.12 is generally used. According to earthquake observations of the authors, dams of this type also have natural frequencies, and vibrations have been recognized to be considerably amplified at their crests.

Of recent earthquakes, the one causing the most damage to dams was the Niigata Earthquake (M = 7.5) of 1964. From the condition of distribution of damage and earthquake observation records at two dams, the intensity of the earthquake motion was considered to be roughly equal to that of the Miyagi-ken-oki Earthquake, and damage was surveyed referring to the maximum acceleration diagram of Fig. 1.

Fig. 5 shows the locations, types and whether or not damage was suffered with regard to dams (as of 1978)) 15 m or higher in an area approximately 150 km north and south of the epicenters of the Miyagi-ken-oki and Niigata earthquakes. The marks o and ● in the figure indicate locations of dams with the latter denoting changes or damage occurring in the Miyagi-ken-oki Earthquake. The mark ▲ denotes change or damage occurring in the Niigata Earthquake. The marks ∇, ▽, □, ⊠, ⌓ and ⌓⌓ indicate dam types, they being concrete gravity dam, hollow concrete gravity dam, fill dam, combination of fill dam and concrete dam, arch dam, and multiple arch dam, respectively.

In the case of the Miyagi-ken-oki Earthquake, there were no dams within a radius of 100 km from the aftershock area center. The only dam suffering real damage to its body was a 24-m high earth dam at a distance of 160 km, with damage at other dams being slight such as temporary increase in leakage.

At the time of the Niigata Earthquake, concrete gravity dams of height 15 m or more located within a radius of 100 km from the aftershock area center were those listed in Table 2, and these were designed with k_H as 0.12. As shown in the table, changes occurred at three dams all of which had heights of 40 m or more. Of these three, the dams No. 110 and No. 76 indicated temporary changes in leakage. At No. 111, leakage from grout pipes occurred, and further, there was leakage from joints of blocks and repair works were carried out to stop the water. Changes occurred at dams other than these three in the Niigata Earthquake, but all were of the degree of temporary variation in leakage, damage to appurtenant facilities such as administration buildings and falling of rocks from cliffs in the surroundings.

An example of a gravity dam in Japan having been close to an epicenter is Mitani Dam. The epicenter of the Tottori Earthquake ($M = 7.4$) of 1943 was only 8 km from the dam. Only a temporary change in leakage occurred due to this earthquake. Taking amplification of earthquake motion by the dam body to be double and considering the maximum acceleration on bedrock of Fig. 1, the abovementioned fact shows there was no real damage to the dam body even if the response acceleration at the dam crest was 4 or 5 times compared with the acceleration calculated from k_H.

Table 1. Major Bridges Suffering Damage

No.	Type	Length (m)	Width (m)	k_H	Year Completed	Damaged Portion: Superstructure	Damaged Portion: Shoe	Damaged Portion: Shoe Seat, Anchor Bolt	Damaged Portion: Bridge Pier Body
1	Through truss steel plate girder	571	6	0.2	1932			x	x
2	Composite girder	309	19	0.2	1965			x	x
3	Continuous steel plate girder	243	9.5	0.22	1974	x			x
4	Gerber steel plate girder	108.8	10.9	0.2	1957	x	x		
5	Langer steel plate girder	367.7	6	0.2	1959		x	x	x
6	Continuous steel box girder	441.55	10	0.24	1974		x		
7	Prestressed concrete (PC) girder	303.6	7.5	0.2	1972		x		
8	Steel truss	450	8.5	0.2	1974	x	x		
9	Gerber reinforced concrete T-girder	306	5.3	0.2	1945	x			x
10	Gerber steel truss girder	181.4	5.5	0.2	1928	x			
11	Truss steel plate girder	575.5	6	0.2	1956	x	x	x	x
12	H-steel plate girder	155	10	0.2	1932				x
13	PC T-girder, PC box girder	541.7	8	0.2	1972	x		x	x
14	H-steel plate girder	236	4.5	0.2	1931		x	x	x
15	Continuous steel truss	288	7.0	0.2	1966	x	x		
16	Steel plate girder	247.3	5.5	0.2	1936		x	x	x
17	Continuous PC girder	525	DTRR	0.25	UC		x	x	x
18	PC box girder, PC girder	179	DTRR	0.25	UC	x			x
19	Steel plate girder	160.2	STRR	0.2	1941				x
20	Reinforced concrete frame	225.1	DTRR	0.2	1959				x

Note: DTRR: Double track (railroad) STRR: Single track (railroad) UC: Under construction

Table 2. Concrete Gravity Dams of Height 15 m or More Within 100-km Radius of Aftershock Area Center of Niigata Earthquake

Distance from Aftershock Area Center (km)	Dam No.	Height (m) 15-20	20-30	30-40	40-50	50-70	70-100	Purpose	Year Completed	Change or Damage
40-50	77					x		F,N,P	1966	
	110				x			P	1965	x
	111						x	F,N,P	1952	x
	115				x			P	1959	
50-75	74				x			P	1933	
	75	x						P	1958	
	76						x	P	1958	x
	91		x					P	1938	
	98			x				P	1958	
	106				x			P	1954	
	107			x				P	1954	
	113			x				P	1962	
	117			x				P	1962	
	118				x			P	1962	
75-100	78	x						P	1938	
	99		x					P	1962	
	125	x						P	1963	
	186	x						P	1959	

Purpose F: Flood control N: Recreation P: Electric power

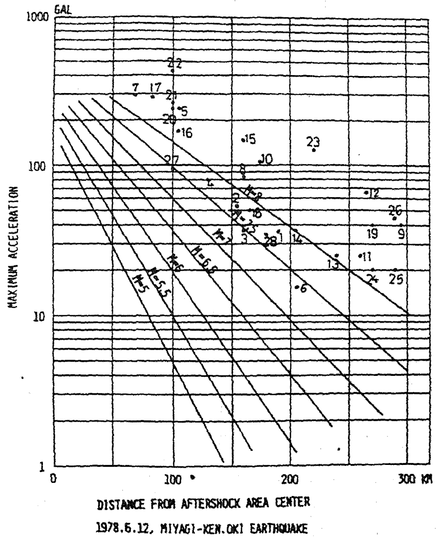

Fig. 1

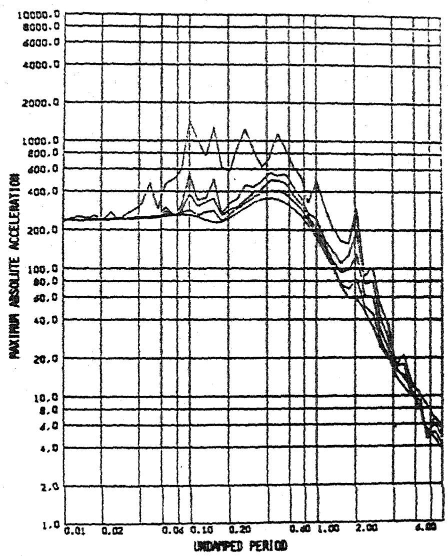

Fig. 2
(from P.W.R.I.,Ministry of Construction)

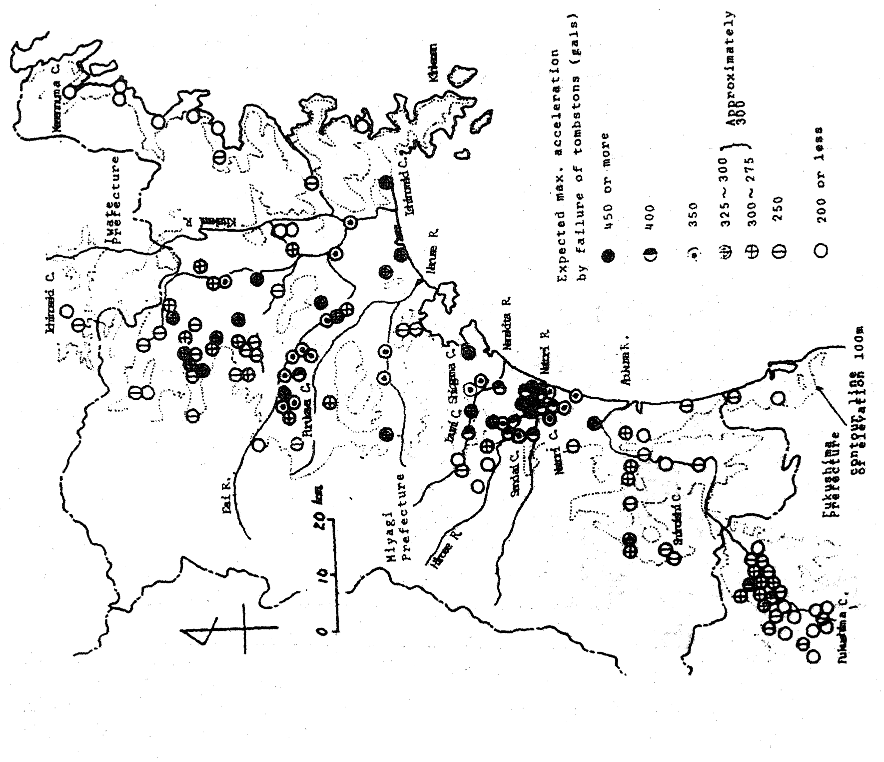

Fig. 3
(from Mr.Kunii et al)

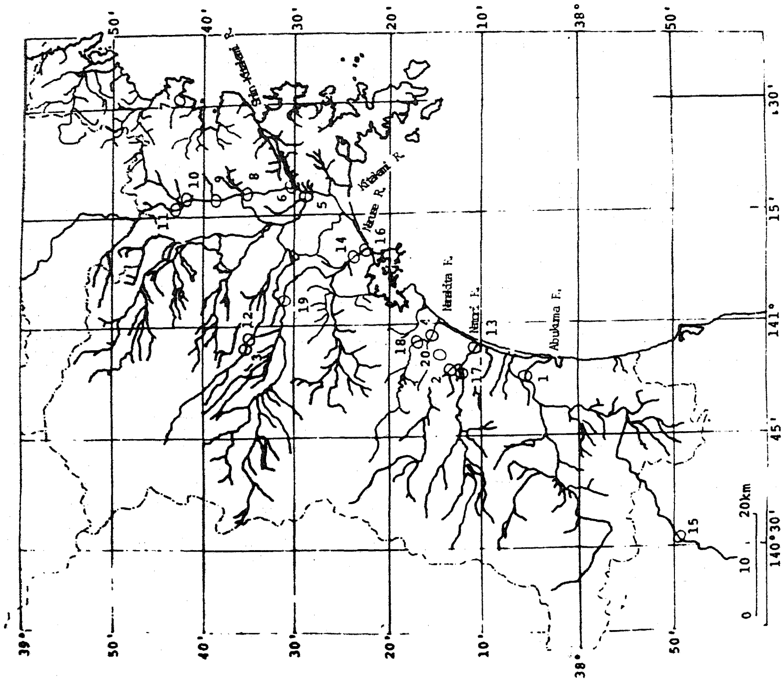

Fig. 4

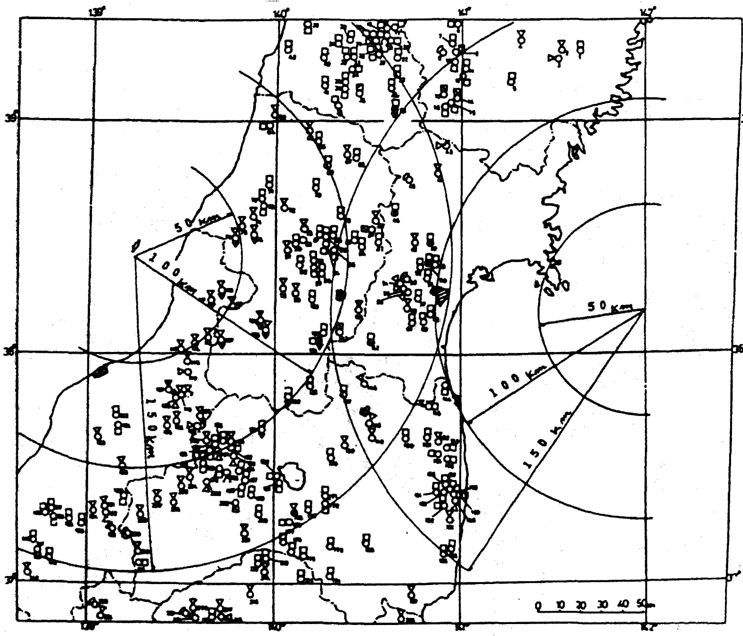

Fig. 5