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LIFETIME SEISMIC RELIABILITY OF REINFORCED CONCRETE STRUCTURES

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Abstract
An evaluation method for the lifetime seismic reliability of reinforced concrete structure is proposed. In this method, a seismic risk analysis is proposed and the damage indices for RC members are defined which take into consideration bending and shearing behaviors. Furthermore, their damage indices are examined by the analysis of several damage reports, and the seismic reliability of RC piers which were designed and reinforced by recent design methods are estimated.
Key words: Reinforced concrete structure, seismic design, lifetime, seismic risk, damage index, damage probability matrix

1 Introduction

In Japan where earthquakes are a common event, structures are often damaged by not only one but by two or more relatively powerful earthquakes during their lifetime. For this reason, in order to provide sufficient seismic safety throughout the lifetime of a structure, its designers have to consider the effects of not only one earthquake, but of many earthquakes which may occur during the structure's lifetime. To study the seismic resistance reliability of a structure throughout

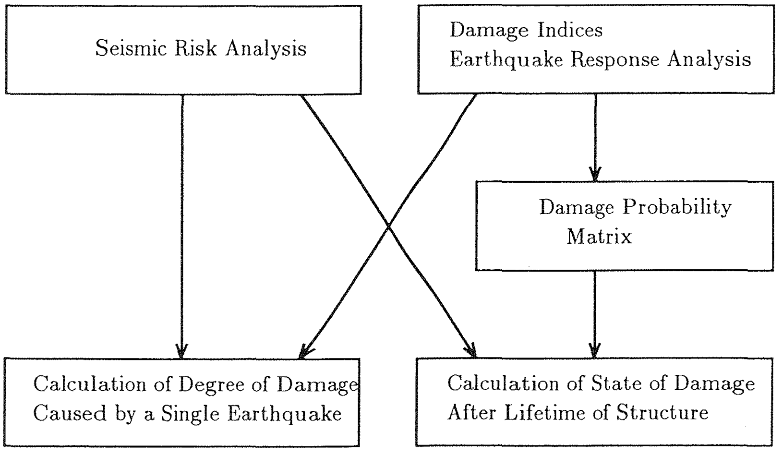

Fig. 1. Flow chart of research project

its lifetime, an attempt was made to forecast the state of damage after lifetime using seismic risk analysis, damage indices, Monte Carlo simulations, and damage probability matrices.

2 Seismic risk analysis

2.1 Analysis method

The earthquake data used for this research project was comprised of earthquake records covering the 400 years period from 1600 to 1988 weighted according to the reliability of the data, and while no particular use was made of active fault data, that included in historical earthquake records was taken into consideration. The earthquake occurrence time distribution model used was a simple Poisson process, and the earthquake motion intensity measurement points were seismic observation facilities at meteorological stations. Refer to the author's past research (Suzuki et al. (1996)) for a detailed explanation of this analysis method.

2.2 Analytical results

Because this research project was based on earthquake data covering a period of about 400 years, the value of maximum acceleration up to a return period of 800 years, which is twice as long as 400 years, was calculated for various calculation points. These results are shown

Table 1. Anticipated value of maximum acceleration in cities during various return periods

Calculation Location	Return Period					Multiplier
	50yr	100yr	200yr	400yr	800yr	
Sendai	195	254	329	428	556	6.28
Tokyo	241	322	430	574	766	5.28
Kyoto	178	228	292	373	477	7.08

(Unit: gal)

in Table 1. And "Multiplier" means multipliers of the earthquake return period for various cities which provide a maximum acceleration twice the maximum earth tremor acceleration during a certain return period. Because of the nature of this risk analysis method, this value is a constant unrelated to the return period. For example Sendai has a value of 6.28, when the return period of a certain earthquake has been set at 100 years, it indicates that the return period of earthquake motion which would cause maximum acceleration double that of this earthquake would be 628 years. Refer to section 6. for this value.

3 Definition of the bending and shearing damage index

The bending damage index D_{M} for this ultimate limit state is defined as shown below.

$$D_{\mathrm{M}} = \frac{H}{R} \tag{1}$$

Where H : The force causing the bulging of the axial direction steel reinforcement (buckling force), R : The force with which the covering concrete resists the bulging of the axial steel reinforcement.

The shearing damage index D_{S} for this ultimate limit state is defined as shown below.

$$D_{\mathrm{S}} = \frac{R_{\mathrm{wn}}}{R_{\mathrm{wu}}} \tag{2}$$

Where R_{wn} : Shearing resistance of the shearing reinforcement in a certain state, R_{wu} : Shearing force resistance of the shearing reinforcement at the shearing reinforcement yield time.

This is based on the fact that after a member suffers bending yielding, cyclic loading reduces the shearing resistance of the concrete and increases the strength after the bending yielding of the member, which in turn gradually increases the shear force which is shared by the shearing reinforcement. See past research by the authors (Akakura et al.(1996)) for details.

4 The seismic response analysis model and simulated seismic waves

Analysis was performed on a single column reinforced concrete pier modelled as a single mass system. The skeleton curve of the load - deformation curve accounted for the rotational deformation caused when the axial reinforcement was pulled out of the footing and also influenced the amount it was pulled out. To determine the hysteresis loop, the bending was basically represented by the Takeda Model (Takeda et al.(1970)), while the results of cyclic box shear testing of the concrete were corrected and used for the shearing (see Reference (Akakura et al.(1996))).

5 Verification of the damage index based on actual damage

5.1 Analysis method

The objects of the analysis were 10 reinforced concrete single column bridge piers on either road bridges or railway bridges. These piers had suffered damage from either the Miyagiken-oki Earthquake of 1978, the Kushiro-oki Earthquake of 1993, the Hokkaido Nansei-oki Earthquake of 1993, or the Hyogo-ken Nanbu Earthquake of 1995. The seismic wave data used was the actual seismic wave forms corresponding to the damage seen in each of the structures. Table 2 presents the seismic wave data used.

5.2 Analysis Results

Table 3 shows the analysis results including a comparison of them with the actual damage data. Five of the seven examples having actual damage consisting of bulging or greater damage are believed to accurately express the condition, "bulging of the axial reinforcement of 1.0" which is the definition of bending damage. Those whose degree of damage was expressed as "cracking" were distributed from 0.2 to 0.5. As for the two piers damaged by the Hyogo-ken Nanbu Earthquake, the bending/shearing damage indices both exceeded 1.0, and during analysis, either the shear ultimate was reached first or the shear ultimate immediately followed the bending ultimate, and in both cases,

Table 2. Actual seismic waves used for analysis

No.	Name	Observation Point	Dir.	Acc_{max}	Dir.	Acc_{max}
STK	Miyagiken-oki	Sendai Bureau (B1F)	NS	432.4	EW	232.6
SUM		Sumitomo Building (B2F)	NS	250.9	EW	240.9
KSR	Kushiro-oki	JMA Kushiro (GL)	063	711.4	153	637.2
HRO		Hiroo Town Office (1F)	320	518.1	050	403.7
SCH	Hokkaido Nansei-oki	Shichihou Bridge (GL)	TR	386.2	LG	379.1
ISO		Isoya Bridge (GL)	TR	157.9	LG	117.7
JMA	Hyogo-ken Nanbu	JMA Kobe (GL)	NS	817.8	EW	617.1
JRT		JR Takatori (GL)	EW	666.2	NS	641.7
EKB		Higashi Kobe Bridge (GL)	N12W	327.3	N78E	280.7

LG : Bridge axis direction TR : Right angles to bridge axis
Unit of maximum acceleration : gal

bending damage increased sharply after reaching the shear ultimate limit. This indicates that shear damage plays an unusually large role when determining damage to the entire bridge pier. Inversely, if the shear damage index remained within 1.0 even when the bending damage index exceeded 1.0, the behavior of the structure showed slight change and resulting in relatively light damage.

6 Evaluation of the damage state after lifetime

6.1 Analysis method

To prepare the damage probability matrices, damage states were categorized according to the value of the damage index. For this research, based on the results of section 5., damage could be categorized into four levels with values ranging from 0 to 1.0, for convenience, based upon the values of the bending and shearing damage indices, then an additional level representing values higher than 1.0 was set to establish a total of five levels. Table 4 presents this categorization.

Bending and shearing damage probability matrices which indicate the probability that the level of damage to a structure will move from a certain condition to the subsequent condition under earthquake motion force with a certain width were prepared. The following is an

Table 3. Results of analysis on actual damage cases

Miyagiken-oki	DI	STK		SUM				Averages
		EW	NS	EW	NS			
Nanakitagawa Bridge	D_M	0.22	0.56	0.30	0.04			0.28
(Separation/cracking)	D_S	0.67	2.03	1.08	0.51			1.07
Kushiro-oki	DI	HRO		KSR				Averages
		050	320	063	153			
Yoda Bridge	D_M	0.27	0.83	1.48	1.47			1.01
(Breakage/bulging)	D_S	0.10	0.22	0.28	0.30			0.22
Matsunoe Bridge	D_M	6.44	16.8	13.7	9.62			11.6
(Bulging/separation)	D_S	5.39	9.19	4.88	7.16			6.65
Shin-Tawa Bridge	D_M	0.00	0.00	0.11	0.83			0.24
(Bending cracking)	D_S	0.00	0.00	0.00	0.03			0.01
Hatsune Bridge	D_M	0.10	0.35	0.75	0.99			0.55
(Bending cracking)	D_S	0.11	0.16	0.27	0.53			0.27
Hokkaido Nansei-oki	DI	ISO		SCH				Averages
		LG	TR	LG	TR			
Motosaka Bridge	D_M	0.00	0.06	0.00	0.00			0.01
(Bulging/separation)	D_S	0.02	0.03	0.03	0.02			0.03
Motouriya Bridge	D_M	0.00	0.00	6.10	10.1			4.05
(Bulging/separation)	D_S	0.07	0.05	1.49	1.98			0.90
Shin-Shiriuchi Bridge	D_M	0.00	0.00	0.41	0.68			0.27
(Bulging/separation)	D_S	0.05	0.06	0.23	0.34			0.17
Hyogo-ken Nanbu	DI	EKB		JMA		JRT		Averages
		NW	NE	EW	NS	EW	NS	
Kobe-P138	D_M	7.82	1.24	1.62	3.40	8.02	9.98	5.35
(Failure(shearing))	D_S	19.6	5.44	6.90	9.60	21.1	24.9	14.6
Nishinomiya-P167	D_M	4.48	0.32	1.36	3.74	6.00	9.16	4.18
(Failure(shearing))	D_S	6.17	0.86	2.52	4.49	7.35	11.3	5.45

Separation : Separation of protective concrete layer
Breakage : Breakage of axial steel reinforcement
Bulging : Bulging of axial steel reinforcement

Table 4. Categorization of damage states

Damage State	Damage Indices (Values of D_M, D_S)	Degree of Damage
I	0 – 0.25	Slight
II	0.25 – 0.5	Minor
III	0.5 – 0.75	Moderate
IV	0.75 – 1.0	Serious
V	1.0 –	Ultimate

Table 5. Specifications of the bridge pier sample designs

Bridge Pier Type	B	H	a	N	p_l	p_w
1990 Design Example B-2	400	170	500	364	0.889	0.096
Before Steel Plate Reinforcement	280	280	1100	470	1.32	0.071
After Steel Plate Reinforcement	280	280	1100	470	1.32	0.645
1990 Guidelines	350	300	1050	1050	0.912	0.073
Restoration Specifications	370	320	1050	1050	0.810	0.183

B : Bridge pier width (cm) N : Overburden load (tf)
H : Bridge pier depth (cm) p_l : Axial steel reinforcement rate (%)
a : Bridge pier height (cm) p_w : Hoop tie reinforcement rate (%)

example of one of the damage probability matrices used.

$$\boldsymbol{M}_{\mathrm{M}}(300;500) = \begin{bmatrix} 0.148 & 0.170 & 0.193 & 0.057 & 0.432 \\ 0.000 & 0.222 & 0.333 & 0.333 & 0.111 \\ 0.000 & 0.000 & 0.250 & 0.625 & 0.125 \\ 0.000 & 0.000 & 0.000 & 0.833 & 0.167 \\ 0.000 & 0.000 & 0.000 & 0.000 & 1.000 \end{bmatrix}$$

It is an example of a bending damage probability matrix, and 0.333 in row two, column three of this matrix indicates the probability that a structure whose initial condition is state II will change to state III under the effects of earthquakes with a motion force between 300 gal and 500 gal.

The damage probability matrices were prepared as follows. Two kinds of simulated seismic waves were continuously administered to the structure to perform seismic response analysis, and by inputting earthquake motion of random size as the first and earthquake motion with a certain stipulated size as the second, the number of changes in degree of damage state were calculated in order to compute the probability. The bending damage probability matrix $\boldsymbol{M}_{\mathrm{M}}$ and the shearing damage probability matrix $\boldsymbol{M}_{\mathrm{S}}$ were independently prepared by performing 200 Monte Carlo simulations for each matrix in order to prepare four levels of matrix from 200 gal to 800 gal at intervals of 200 gal.

Based on the damage probability matrices prepared and the results of section 2., a state probability matrix was finally obtained by combining the annual average probability of earthquake motion for each city with all probability elements to calculate the overall probability. Based on this, the damage state of the structure after its lifetime was calculated. Refer to the literature (Suzuki et al.(1996)) for the detailed calculation method.

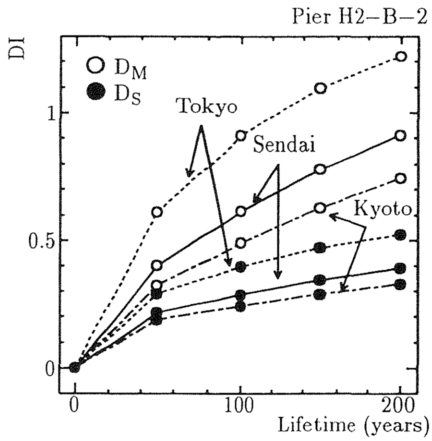

Fig. 2. Examples of anticipated values of the degree of damage over various lifetimes

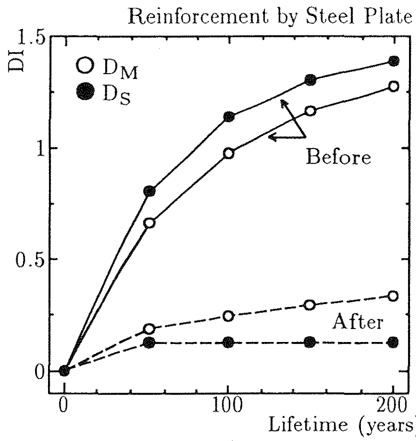

Fig. 3. Examples of effects of steel plate reinforcement during various lifetimes (Sendai)

6.2 Results

Figure 2 shows examples of anticipated damage levels to a structure after its lifetime of 1990 Design Example B-2 (Japan Road Association 1990) in Table 5 if it was located in Sendai, Tokyo, and Kyoto. It reveals that the anticipated levels are distributed from Tokyo, with the highest earthquake risks, to Kyoto with the lowest. In this case the bending damage index for Tokyo exceeds 1.0 after 150 years. And a comparison of the state "Before Steel Plate Reinforcement" and "After Steel Plate Reinforcement" in Table 5 is shown in Fig. 3. From this, it is assumed that when the cumulative damage to a structure has been accounted for, after 100 years of use both bending and shearing damage will be close to 1.0, thus being in a ultimate state, but after reinforcement is completed, even after 200 years of use, the values of both damage indices will be less than 0.3, resulting in only slight damage at the end of its lifetime.

Figure 4 compares examples of design based on the 1990 Guidelines and on the Restoration Specifications (Japan Road Association (1995)) in Table 5. In this case, although the degree of bending damage of the Restoration Specifications case does not change as much as that of the 1990 Guidelines case, a big improvement was achieved in the degree of shearing damage, revealing an improvement in the shear strength. In this 1990 Guidelines case, the anticipated value of shearing damage after 150 years of use exceeds 1.0, and it is assumed that it would reach ultimate shearing before ultimate bending.

6.3 Considerations

Consideration has been given to the question of just what relationship exists between the degree of damage at the end of structure's lifetime when cumulative damage has been accounted for, and a single earthquake which inflicts the same degree of damage on the structure.

Figure 5 presents the degree of cumulative damage to a structure in Sendai during its lifetime (T) / return period of a single earthquake which inflicts the same degree of damage (R) ratios R/T with the horizontal axis representing the lifetime of the structure. This figure presents 10 examples calculated for Sendai. It reveals that when cumulative damage is accounted for, consideration must be given to a single earthquake with a return period occurring from 2 to 9 times the lifetime, and demonstrates that cumulative damage must be accounted for in the design of a structure. It also shows that overall, there is a tendency for the multiplier to become smaller as the lifetime increases.

But the value of this multiplier naturally varies according to the seismic risk at the calculation location. Because analysis must be performed for each calculation location, it must be considered from a separate perspective. The lateral axis ($R/T = 6.28$) in Fig. 5 means a multiplier of 6.28 times the return period of an earthquake which produces a motion twice as strong as the earthquake motion with a return period identical to the lifetime in Sendai (See Table 1). This means that when the lifetime of a structure is considered to be approximately 100 years, if an earthquake force of about the same strength is accounted for, the result obtained will be identical to that of the case where cumulative damage was accounted for.

In other words, it is possible to conclude that, "In a case where the lifetime of a structure is assumed to be 100 years, in order for it to be able to withstand the cumulative damage, seismic force double that of the maximum earthquake which is forecast to occur during that period at the location of the structure must be accounted for."

7 Conclusions

(1) Damage indices for bending and shearing have been defined and their correspondence with the actual degree of damage and the criterion for the failure of a structure have been presented.

(2) Damage to an existing building is effected far more by shearing than by bending, so in order to limit damage, the shear strength of the structure must be increased to improve its resistance to damage.

(3) It is necessary to promptly reinforce structures having insuffi-

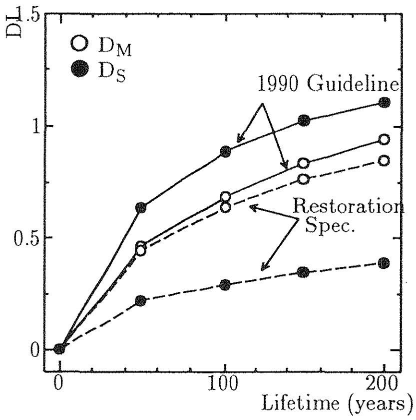

Fig. 4. Example of a comparison between the new and old design during various lifetimes (Sendai)

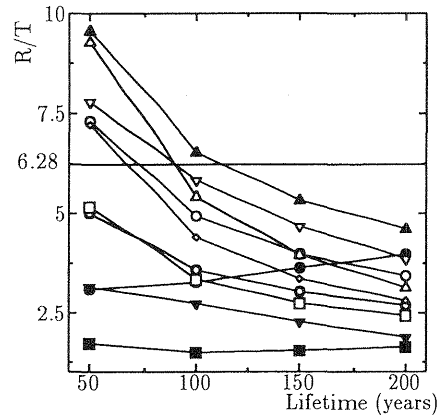

Fig. 5. Multipliers for Sendai (10 damage index cases)

cient seismic resistance, as deemed necessary, based on calculations of the anticipated values of cumulate damage.

(4) When the accumulation of damage is not considered, it is necessary to assume a subsequent seismic force double that of a single earthquake forecast at the location of the structure during its lifetime.

8 References

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