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EVALUATION METHOD FOR LIFETIME SEISMIC RELIABILITY OF REINFORCED CONCRETE STRUCTURES

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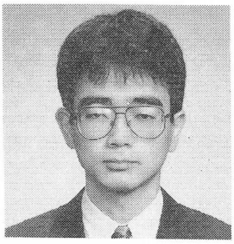

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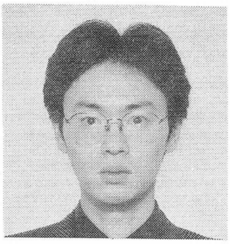

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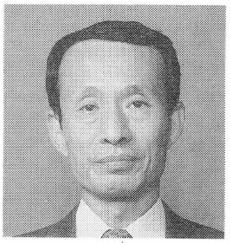

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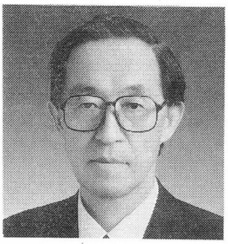

Yoshio OZAKA

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.

Keywords: reinforced concrete structure, seismic design, lifetime, seismic risk, damage index, damage probability matrix

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1. INTRODUCTION

The Hyogo-ken Nanbu Earthquake of January 17, 1995 claimed many victims, caused severe damage, and at the same time taught numerous lessons and left many problems to be solved concerning seismic design.

After the Hyogo-ken Nanbu Earthquake, the Japan Society of Civil Engineers issued the paper "Proposal Concerning Seismic Resistance Standards for Civil Engineering Structures"[1] (hereafter referred to as, "Proposal"). It stated the need to account for extremely strong earthquakes and that the assumed state of damage to structures must take into account the degree of importance of each structure, and proposed seismic resistance diagnoses of existing structures followed by immediate retrofitting as needed.

It indicated the necessity of reviewing existing ideas about earthquake motion intensity and the necessity of including the effects of earthquakes with very long return periods, but it did not clearly stipulate to what extent this, "extremely strong earthquake motion intensity with a low probability of occurrence" can be determined, so further study is needed.

The concept that the state of damage and importance of structure should be assumed is incorporated in "Road Bridge Guidelines"[2] as a correction coefficients for the standard design horizontal seismic coefficient. And in the "Specifications for the Restoration of Road Bridges Damaged by the Hyogo-ken Nanbu Earthquake" [3] (hereafter referred to as, "Restoration Specifications"), doubling the design seismic coefficient for more important bridges is applied with necessary modifications.

The plastic factor is often used to assume damage, but because its relationship with the degree of damage to members or structures is unclear, currently damage indices are widely used to estimate the degree of damage. The existing damage indices are all proposed for members with a dominant bending behavior. But a common belief is that during the Hyogo-ken Nanbu Earthquake, shear damage was an important determining factor in the overall damage to structures[4]. In short, in order to evaluate the extent of damage to members suffered during an earthquake, it is essential to use an index which accounts for damage caused by shearing in addition to bending to obtain comprehensive estimate of the degree of damage.

Here 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 are occur during the structure's lifetime.

In recent years, a rise in the cost of maintaining structures has been accompanied by a demand for design that optimizes the cost - benefits of structures. In the field of seismic design, this has meant efforts to minimize both initial construction costs and the costs of repairing earthquake damage. Another view point holds that future structures must have lower maintenance costs than existing structures. The foundation of both approaches is the concept of a structure's lifetime, and both require the establishment of a method enabling designers to forecast the state of damage to a structure during its lifetime and at the end of its lifetime (hereafter referred to as "after lifetime") during the design phase.

This paper looks into the estimation of the probability of earthquake motion occurrence with a long return period based on seismic risk analysis, and the evaluation of the comparison between a structure's seismic resistance performance and the occurrence of earthquakes. To do this, damage indices which consider the state of damage due to shearing or flexure were used to verify the damage indices and the value of the degree of the damage, and to compare the value of the damage indices and the degree of the actual damage for a reinforced concrete structure damaged by past earthquakes. The study also took into consideration the differences in the

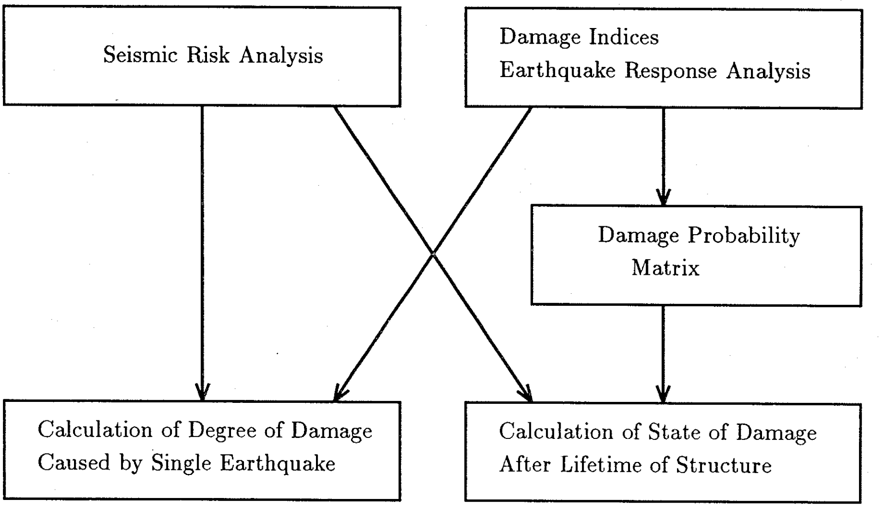

Fig. 1 Flow Chart of Research Project

degree of damage depending on the differences between the new and existing design methods and the absence or presence of reinforcement, and hypothesized at what value the degree of damage should be considered.

Further, to study the seismic resistance reliability of a structure throughout 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. Consideration was given to earthquake motion strength in a case where the effects of cumulative damage caused by multiple earthquakes were substituted for the damage caused by a single earthquake, and based on the result, the values of earthquake motion strength which should be considered in the design of structures was defined. A flow chart of this research project is shown in Fig. 1.

The authors have previously conducted research on seismic resistance evaluations throughout the lifetime of a structure[5], but in this paper, they used a new index[6] which can account for both bending and shearing to evaluate the damage caused by a single strong earthquake to multiple structures, to verify the degree of damage evaluation throughout a structure's lifetime for a number of cases, and as they demonstrated its usefulness, at the same time they considered problems related to earthquake resistance engineering based on these results.

2. SEISMIC RISK ANALYSIS

2.1 Past Research

Ebisawa et al.[7] combined data concerning past earthquakes and active faults, performed seismic risk analysis treating the occurrence process as a non-Poisson process, actually applied this evaluation method, and by varying the past earthquake's reference period evaluated the way the reference period influenced the risk. Their results revealed that the calculation points could be categorized in two groups, points where past powerful earthquakes caused by active faults are dominant, and points where, due to no occurrence of extreme earthquakes, there was no problem even when forecasting earthquakes with a long return period from earthquake data over the past 100 years. Based on these results, it is possible to demonstrate a certain degree of suitability, even when using the conventional risk analysis method, for forecasting earthquake motion with an extremely long return period.

2.2 Analysis Method

The earthquake data used for this research project was comprised of earthquake records covering the 400 year 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. The decay by distance formula used was a formula for standard ground represented by the following equation[8].

$$Acc_{\max} = 18.4 \cdot 10^{0.382M} \cdot \Delta^{-0.8} \tag{1}$$

Where $Acc_{\max}$: Maximum acceleration (gal), M : Magnitude, Δ : Epicentral Distance (km).

Refer to the author's past research[5] for a detailed explanation of this analysis method.

2.3 Analytical Results

It was assumed that, it is also possible to forecast earthquake motion with a long return period from earthquake data covering a relatively short period of time as in the case of the research by Ebisawa et. al.. 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 in Table 1. In this table, the value with a return period of 800 years at Niigata is far higher than those obtained for other locations, but this is believed to be a result of reduced precision because only a small amount of historical earthquake data was available for the Niigata vicinity. It must, therefore, be used carefully.

Table 2 shows 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. DEGREE OF DAMAGE EVALUATION

3.1 Past Research

Park et. al.[9] represented the degree of damage as the linear sum of the maximum deformation rate and the consumed energy rate, and after applying this to about 400 experimental results, demonstrated that this damage index closely follows a normal logarithmic distribution. The authors[10] took the loading hysteresis into account to define a damage index for reinforced concrete members which suffered bending failure subsequent to bending yield, and to verify the damage index itself experiments were conducted with specimens which suffered bending failure after bending yield. Furthermore, they applied it to earthquake response analysis to indicate

Table 1 Anticipated Value of Maximum Acceleration in Cities During Various Return Periods

Calculation Location	Return Period				
	50yr	100yr	200yr	400yr	800yr
Sapporo	127	160	202	255	321
Sendai	195	254	329	428	556
Tokyo	241	322	430	574	766
Niigata	222	377	641	1088	1846
Nagoya	261	361	499	690	954
Kyoto	178	228	292	373	477
Osaka	191	250	328	430	564
Hiroshima	136	187	256	350	479
Takamatsu	156	197	250	317	402
Fukuoka	120	154	198	255	327

(Unit: gal)

Table 2 Certain Return Period/Return Period of Seismic Force Double the Maximum Acceleration During That Period Ratio

Calculation Location	Multiplier	Calculation Location	Multiplier
Sapporo	7.96	Kyoto	7.08
Sendai	6.28	Osaka	5.92
Tokyo	5.28	Hiroshima	3.05
Niigata	2.48	Takamatsu	7.61
Nagoya	4.40	Fukuoka	6.82

the relationship of the damage index to the failure probability in order to position the damage index in seismic design.

3.2 Definition of the Damage Index

a) Ultimate Limit State Caused by Bending and the Damage Index
The ultimate limit state caused by bending was defined based on a phenomenon with the following three aspects: "protective concrete separation, the bulging outward of the axial steel reinforcement caused by this separation, the point where the bulging causes the load on the load - deformation curve to decline or the loop shape to change."

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

$$D_{\mathrm{M}} = \frac{H}{R} \tag{2}$$

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.

b) Ultimate Limit State Caused by Shearing and the Damage Index
The ultimate limit state caused by shearing was defined based on a phenomenon with the following three aspects: "protective concrete separation, the yielding of the shearing reinforcement

Table 3 Material Properties of the Concrete and Reinforcement

Concrete	Compressive strength (kgf/cm^2)	240
	Tensile strength (kgf/cm^2)	32
	Strain at time of max. compressive stress	0.002
	Ultimate strain	0.0035
Steel Reinforcement	Yield strength (kgf/cm^2)	3500
	Tensile strength (kgf/cm^2)	5000
	Yield strain	0.002
	Strain at beginning of strain stiffening	0.02
	Ultimate strain	0.1

caused by this separation, the abrupt decline of the load caused by this yielding."

The shearing damage index D_S for this ultimate limit state is defined as shown below.

$$D_S = \frac{R_{wn}}{R_{wu}} \tag{3}$$

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[6] for details.

4. THE SEISMIC RESPONSE ANALYSIS MODEL AND SIMULATED SEISMIC WAVES

4.1 Seismic Response Analysis Model

Analysis was performed on a single column reinforced concrete pier modelled as a single mass system. An elasto-plastic response analysis was performed using the β method of Newmark and based on linear acceleration for which $\beta = 1/6$. The damping constant was set at 0.02. 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. The quantity of axial reinforcement pull out was determined through the use of proposed by Shin et. al.[11]. To determine the hysteresis loop, the bending was basically represented by the Takeda Model[12], while the results of cyclic box shear testing of the concrete were corrected and used for the shearing (see Reference[6]).

Calculations were performed assuming that the bridge pier was divided into 20 elements along the axis, each cross section was divided into 20, and the time was divided into periods of 1/500 seconds. The material properties used are shown in Table 3. Specific calculation was done by using the cross section division method to calculate the $M - \phi$ relationship based on the material properties and the specifications of the structure, determining its cracking point, yielding point, and other characteristic points as inflection points of the hysteresis loop, and based on this, the seismic response of the single mass system was analyzed. The seismic wave input direction was, in principle, the bridge axis direction, whose seismic resistance properties are low.

4.2 Method of Preparing the Simulated Seismic Waves

In addition to actual seismic waves, simulated seismic waves represented by the following equation were used for this seismic response analysis[13].

$$\ddot{y_0}(t) = g(t) \cdot z(t) \tag{4}$$

$z(t)$ was obtained by applying and superimposing a random phase on equal frequency, equal interval harmonic wave forms within a given power spectrum. In this phase of the research a model of the power spectrum concentration of earthquake motion $S(\omega)$ the Kanai - Tajimi spectrum was used. The envelope function $g(t)$ which represents the irregularity was a formula represented by the linear sum of the e function.

5. VERIFICATION OF THE DAMAGE INDEX BASED ON ACTUAL DAMAGE AND SEISMIC RESISTANCE EVALUATION OF REINFORCED CONCRETE STRUCTURES

5.1 Past Research

Almost no research has been conducted on the use of a damage index to verify actual earthquake damage to a structure, nor to verify a forecast of earthquake damage to an actual design.

Concerning the plastic factor, the Concrete Standard Guidelines[14] categorize the maximum response displacement - degree of damage relationship of a structure during an earthquake based on actual data from previous damage and experimental results. Ikeda et. al.[15] have verified the state of damage in the Concrete Standard Guidelines using the state of cracking as the criterion. Park et. al.[16] used a damage index they proposed themselves (see section 3.) to define the degree of damage to an overall structure derived from the degree of damage to its members, and verified the damage index by performing a seismic response analysis of nine damaged buildings.

5.2 Verification of Damage Index Based on Actual Damage

a) 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[17], the Kushiro-oki Earthquake of 1993[18], the Hokkaido Nansei-oki Earthquake of 1993[19], or the Hyogo-ken Nanbu Earthquake of 1995[4]. These 10 piers were selected by eliminating those impossible to analyze, their height ranged from 400 cm to 1,510 cm, with their shear - span ratios between 2.75 and 9.11. The seismic wave data used was actual seismic wave forms corresponding to the damage seen in each of the structures[18–21]. Table 4 presents the seismic wave data used. Because acceleration records pertaining to the locations of the damaged structures were not obtained, 4 to 6 types of wave forms observed during the earthquakes near the locations of the structures, and believed to be relatively appropriate were input, their averages determined from the results and used to verify the damage index itself.

b) Analysis Results
Table 5 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"

Table 4 Actual Seismic Waves Used for Analysis

No.	Name of Earthquake	Observation Point	Direction	Acc_{max}
STK	Miyagiken-oki	Sendai Railway Administrative Bureau (B1F)	NS	432.42
			EW	232.61
SUM		Sumitomo Building (B2F)	NS	250.90
			EW	240.90
KSR	Kushiro-oki	JMA Kushiro Observatory (GL)	063	711.40
			153	637.24
HRO		Hiroo Town Office (1F)	320	518.05
			050	403.68
SCH	Hokkaido Nansei-oki	Shichihou Bridge (GL)	TR	386.21
			LG	379.10
ISO		Isoya Bridge (GL)	TR	157.89
			LG	117.71
JMA	Hyogo-ken Nanbu	JMA Kobe Observatory (GL)	NS	817.83
			EW	617.14
JRT		JR Takatori (GL)	EW	666.20
			NS	641.73
EKB		Higashi Kobe Bridge (GL)	N12W	327.31
			N78E	280.72

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

which is the definition of bending damage. Those whose degree of damage was expressed as "cracking" were distributed from 0.2 to 0.5. The value for the Motosaka Bridge was small, but because its site was much closer to the hypocenter than the other two bridges damaged by the Hokkaido Nansei-oki Earthquake and because the seismic waves used for the analysis were not obtained near the hypocenter or the site of the Motosaka Bridge, the value is believed to be smaller because the analysis value was smaller than the actual damage.

For the four bridge piers where the shearing damage exceeded 1.0, all damage was marked by prior shearing. In the case of the Matsunoe Bridge, it can be assumed that the shear - span ratio of the bridge pier was the smallest (2.75) of all those analyzed and that the shear behavior tended to be dominant. 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, bending damage increased sharply after reaching the shear ultimate. 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. This will be explained in greater detail later.

It is not always possible to accurately reproduce actual damage using the bending and shearing damage indices, but it is believed that in actual use, they could reproduce with relative accuracy earthquake damage inflicted on a reinforced concrete structure, and can therefore, be used to estimate earthquake damage.

5.3 Evaluating the Seismic Resistance of Reinforced Concrete Structures Based on Various Design Methods

a) Analysis Method
An evaluation of reinforced concrete bridge piers either newly constructed or reinforced based on recent design methods was carried out to determine the degree of damage which would be

Table 5 Results of Analysis on Actual Damage Cases

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

The degree of damage based on various earthquake reports is indicated in the parentheses under the name of each bridge.

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

inflicted on them by relatively powerful earthquake motion. A comparison of the earthquake resistance performance was also obtained using old and new design methods, including the assessment of reinforcement work effectiveness.

Included in the structures analyzed were an example of a new design based on the 1990 edition of the Road Bridge Guidelines[2] (hereafter "1990 Guidelines") and the Restoration Specifications[3], and an example of a design based on guidelines earlier than 1990, which was later reinforced using both the steel plate lining method and the reinforced concrete lining method based on the Restoration Specifications. In this analysis 16 reinforced concrete single column bridge piers were used with heights ranging from 500 cm to 1,350 cm and shear - span ratios between 2.43 and 5.40. Table 6 shows the specifications of 8 typical bridge piers which were used to illustrate the following explanation. Three types of seismic waves were used: Miyagiken-oki Earthquake (B1F, NS, at the Sendai Railway administrative Bureau), Hyogo-ken Nanbu Earthquake (GL, NS, at the JMA Kobe Observatory), and standard wave forms used for time history response analysis in the Road Bridge Guidelines (For category II ground, damping constant of 2 %). The verification was performed by expanding and contracting their maximum acceleration between 600 gal and 800 gal.

b) Analysis Results

Table 7 shows the analysis results when a maximum acceleration of 800 gal was input. The comparison of conditions before and after reinforcement reveals that for both steel plate and reinforced concrete types of reinforcement, the shearing damage exceeded 1.0 in all cases prior to

Table 6 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
Before Reinforced Concrete Reinforcement	250	190	950	490	1.54	0.106
After Reinforced Concrete Reinforcement	300	240	950	490	1.26	0.258
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) H : Bridge pier depth (cm)
a : bridge pier height (cm) N : Overburden load (tf)
p_l : Axial steel reinforcement rate (%) p_w : Hoop tie reinforcement rate (%)

Table 7 Results of Analysis of Actual Design Cases

Pier Type	Damage Index	Miyagiken-oki	Hyogo-ken Nanbu	Std. Wave Form
		800gal		
Before Steel Plate Reinforcement	D_M	**1.282**	**2.824**	**4.284**
	D_S	**1.775**	**4.510**	**5.782**
After Steel Plate Reinforcement	D_M	0.067	0.946	**2.026**
	D_S	0.463	0.092	0.138
Before RC Reinforcement	D_M	**2.907**	**2.918**	**7.185**
	D_S	**1.355**	**1.247**	**2.594**
After RC Reinforcement	D_M	0.646	**1.348**	**1.771**
	D_S	0.387	0.624	0.751
1990 Guidelines	D_M	**3.200**	**3.525**	**6.388**
	D_S	**1.847**	**1.924**	**3.292**
Restoration Specifications	D_M	**2.803**	**4.047**	**6.188**
	D_S	0.702	0.911	**1.315**

reinforcement and in the case of those specimens which are forecast to reach extreme damage or failure, after reinforcement their shear damage reached a maximum of 0.75, showing the remarkable effectiveness of steel plate or reinforced concrete reinforcement.

A comparison of D_M and D_S produced by input seismic waves at the same maximum acceleration revealed that the standard wave forms were larger for the Hyogo-ken Nanbu Earthquake and smallest for the Miyagiken-oki Earthquake. According to a comparison of the 1990 Guidelines and Restoration Specifications designed under identical conditions, although not much bending damage improvement was obtained, the shearing damage improved substantially. Taking into consideration the concept that shearing damage governs the amount of damage, it is desirable to improve the resistance to shearing damage.

A comparison of old and new design methods under identical design conditions, shows no clear difference in the degree of damage such as those under "prior to reinforced concrete reinforcement" and "1990 Guidelines" in Table 7 for example. It is assumed that due to big differences in the design conditions, design method differences were not very apparent in the case of strong earthquake motion. It is also assumed that in the case of strong earthquake motion, differences in the weight of the superstructure have much greater effect on the degree of damage than differences in the design methods.

5.4 Consideration of the Results

a) Seismic Resistance Performance Diagnosis Formula

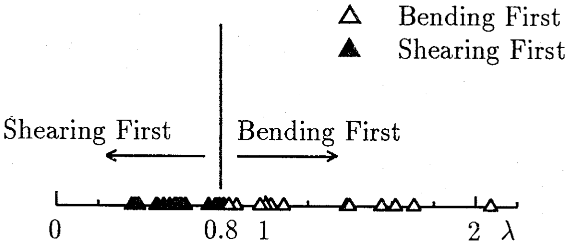

Fig. 2 Correspondence of the Seismic Resistance Diagnosis Equation and the Damage Indices

The "Proposal Concerning the Seismic Resistance Evaluation and Seismic Resistance Reinforcement of Existing Reinforced Concrete Columns and Bridge Piers"[22] issued by the Japan Society of Civil Engineers Concrete Committee after the Hyogo-ken Nanbu Earthquake proposes that seismic reinforcement performance be diagnosed using the following equation.

$$\lambda = \frac{V_{\mathrm{y}} \cdot a}{M_{\mathrm{u}}} \tag{5}$$

Where λ : Seismic resistance performance diagnosis index, V_{y} : Shear bearing force of a member, a : Shear span, M_{u} : Bending bearing force of a member.

This section examines the effectiveness of this seismic resistance diagnosis formula and its relationship with the damage indices proposed by this research. Fig. 2 presents the survey results of a total of 26 bridge piers analyzed in subsection 5.2 and 5.3. The words "Bending First" and "Shearing First" in the figure indicate cases where the respective damage index reached 1.0 first; it does not indicate the structure's mode of failure. The figure clearly shows that the degree of bending damage tends to reach 1.0 first when the seismic resistance diagnosis index λ is greater than about 0.8, and the degree of shearing damage reaches 1.0 first when λ is lesser than about 0.8. The results of a comparison of the seismic resistance performance diagnosis index and the damage index proposed by this research reveals the conclusion that this seismic resistance performance diagnosis index is convenient, and is capable of representing the seismic resistance performance of a structure.

b) Simple Formula of the Damage Index

Because the amount of axial reinforcement bulging, which is the basis for the bending damage index in this research project, is calculated from the plastic factor, the relationship between the bending damage index and the plastic factor was investigated. The following formula was obtained by verifying the results from a total of 44 cases, obtained by inputting 4 to 6 kinds of seismic waves to the bridge piers in subsection 5.2 which verified the damage inflicted.

$$\begin{aligned} D_{\mathrm{M}} &= 0.608\mu - 0.742 \\ r &= 0.884 \end{aligned} \tag{6}$$

Where D_{M} : Bending damage index, μ : Bending plastic factor of a member, r : Coefficient of correlation.

In the Railway Structure Etc. Design Standards[23], the plastic factor during design is assumed to be about 4, and in the Restoration Specifications, the criterion is set at a maximum of about 8. Because the value for plastic factor $\mu = 8$ in formula (6) is $D_{\mathrm{M}} = 4.12$, it can be concluded from this result that if a structure has sufficient strength, even if the bending damage index exceeds the ultimate of 1.0, the structure will not fail between bending damage levels of 3 and

4. But it is assumed that if the shearing damage index exceeds 1.0, it will have a significant effect on the bending damage index, and resulting in a tendency towards severe damage.

As was shown in subsubsection 5.4 a), the correlation between the seismic resistance performance diagnosis formula and the damage index used for this research, the following formula was obtained by attempting to represent the shearing damage index by using both this seismic resistance performance diagnosis index and the plastic factor.

$$\begin{aligned} D_S &= \lambda^{-3.40}(0.12\mu - 0.065) \\ r &= 0.665 \end{aligned} \tag{7}$$

Where D_S : Shearing damage index, λ : Seismic resistance performance diagnosis index, μ : Bending plastic factor of a member, r : Coefficient of correlation.

This formula itself is not necessarily the optimum form, and the value of the coefficient of correlation is not necessarily a high value. But it is useful when used to calculate the shearing damage from the bending plastic factor and the seismic resistance performance diagnosis index, which can be obtained easily. Because the simplified bending degree of damage formula is represented by the plastic factor, this formula itself is considered vague. But because this bending damage index is where the limit stage has been defined, the degree of damage is a clear index, unlike the plastic factor, and the simultaneous study of it and the simplified formula for the degree of shearing damage when taking the seismic resistance performance diagnosis index into consideration reveals that it is extremely practical. In formulae (6) and (7) for example, in cases where the shearing damage index D_S is 0.7, the hypothesized design plastic factor should be no greater than $\mu = 8$, the minimum level of seismic resistance performance diagnosis index which a structure requires is $\lambda = 1.07$. Consequently, it is necessary to reinforce structures which do not satisfy this requirement.

These two formulae are approximate equations, and the probability exists that the number of bridge piers analyzed was not sufficient. In fact, the value of the plastic factor calculated in the earthquake response analysis is believed to be highly dependent on the bending or shearing model, and as long as the model can not be applied as it is, it is insufficiently accurate. In order to improve the accuracy of the calculations and verify their usefulness, it is necessary to verify that they conform with the results of alternating loading testing.

6. EVALUATION OF THE DAMAGE STATE AFTER LIFETIME

6.1 Past Research

Shinozuka et al.[24] have proposed a future reliability evaluation method for structures damaged by past earthquakes. The structure studied was a single mass point system reinforced concrete column and the damage index was the quantity of stiffness decline or maximum cracking width (test values or analytical values) with high correlation to the quantity of stiffness decline. The damage could be categorized into three levels, serious, moderate, and light; two matrices, the "initial damage probability matrix" and "state damage probability matrix" were introduced, and their probability was calculated based on 50 Monte Carlo simulations. It is assumed possible to assess the degree of reliability under a subsequent earthquake occurring on structures which have suffered past damage.

This research contains two strong points: the damage index can be considered linked to the maximum cracking width obtained through observation and because two kinds of matrices are used, the damage index accurately reflects the properties of reinforced concrete structures whose behavior differs sharply according to loading conditions. But problems remain: the

Table 8 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

Monte Carlo simulation frequency is too low, and the damage state is broadly classified into three categories.

6.2 Analysis Method

a) Damage Categorization
To prepare the damage probability matrices, damage states were categorized according to the value of the damage index. For this research, different indices were used for bending and shearing, and although identical damage index values do not always represent the same degree of damage, in this case, 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 8 presents this categorization. The degrees of damage shown in this table are approximations based on the results of the analysis, they are not actual values and were not evaluated using test results. Because even if it is assumed that there is a difference between the categorization and actual degree of damage, this will not effect the calculation of the final results, therefore the purpose of this damage categorization should be seen as a categorization of convenience used to prepare the damage probability matrices rather than as a representation of a relationship between the damage index values and actual damage.

b) Preparation of the Damage Probability Matrices
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 example of one of the damage probability matrices used.

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

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. Because the level of damage does not decline, the part below the diagonal line of the matrix is zero.

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

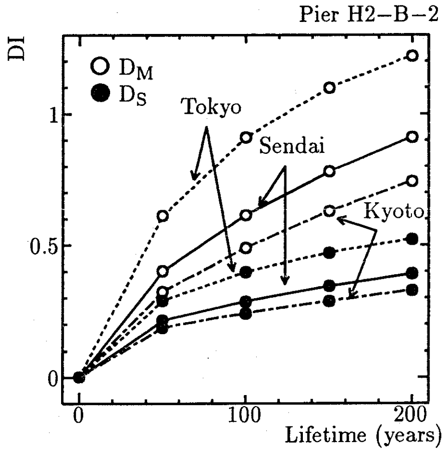

Fig. 3 Examples of Anticipated Values of the Degree of Damage Over Various Lifetimes

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.

c) State of Damage at End of Lifetime Calculation Method
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. Because the state for a structure when it is constructed is "no damage," only the first row of this state probability matrix was used to calculate the anticipated after lifetime values, and based on this, the damage state of the structure after its lifetime was calculated. Refer to the literature[5] for the detailed calculation method. A structure which has been damaged by an earthquake is usually repaired and reinforced according to its state of damage values. But because the focus of this research project was determining just how a structure is effected throughout its lifetime, it was assumed that the structure would neither be repaired nor reinforced.

6.3 Results

Figure 3 shows examples of anticipated damage levels to a structure after its lifetime of 1990 Design Example B-2 in Table 6 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 6 is shown in Fig. 4. 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. Such a bridge pier would have to be reinforced, and this reinforcement work should have first priority in high seismic risk regions, but in areas where there are many piers with such damage indice values a determination must be made on how quickly the reinforcement work can be done. If for example, it is assumed that all bridge piers will be reinforced in 100 years, as the anticipated values of all the bridge piers, the curve

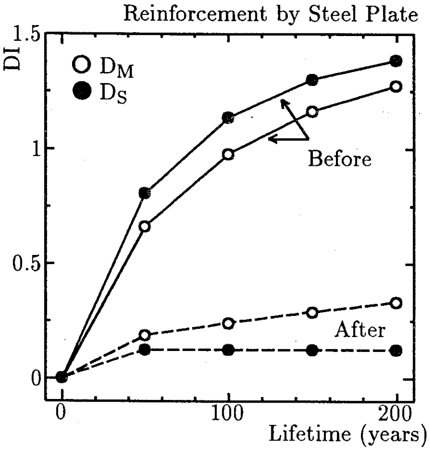

Fig. 4 Examples of Effects of Steel Plate Reinforcement During Various Lifetimes (Sendai)

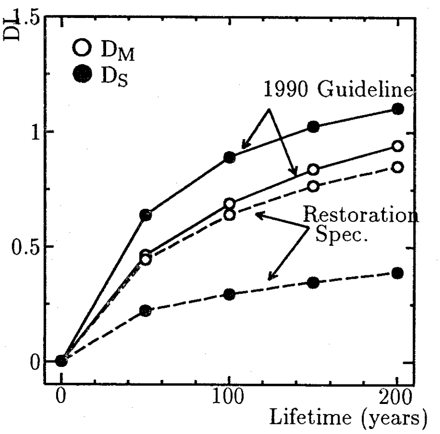

Fig. 5 Example of a Comparison Between the New and Old Design Methods During Various Lifetimes (Sendai)

for the anticipated values of the damage indices in the figure will be a curve oriented towards a median value between "before reinforcement" and "after reinforcement" after 100 years of service, and in this case, the value will be about 0.7. If for example, the anticipated value after 100 years is less than 0.4, the prompt reinforcement of the bridge piers will be necessary.

Figure 5 compares examples of design based on the 1990 Guidelines and on the Restoration Specifications in Table 6. 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.

6.4 Considerations

The concept of the lifetime of a structure can be discussed from a variety of perspectives. For example, the legally stipulated lifetime of a bridge pier according to the provisions of an order from the Ministry of Finance is between 40 and 50 years[25]. But there are many examples of structures which continue to be used and which fully perform their required functions even though they are much older than this stipulated lifetime. Considering the necessity for future repair work and changes in maintenance and social conditions, the future projected lifetimes of structures must be longer than those calculated up to now. In recognition of this fact, 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 6 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. 6 means a multiplier of 6.28 times the return period of an earthquake which produces motion twice as strong as the earthquake motion with a return period identical to the lifetime in Sendai (See Table 2). 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.

Figure 7 is an example of a similar comparison of this relationship for Tokyo and Kyoto. It reveals a distribution sequence which conforms to the earthquake risk. And as in Fig. 6, this figure shows the multiplier of the return period of an earthquake whose motion is twice as strong as an earthquake with a return period equal to the lifetime of a structure for each city. When a lifetime of about 100 years has been accounted for, the results for Tokyo and Kyoto are almost identical to those for Sendai.

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." For example, in a case where the lifetime of a structure in Sendai is considered to be 100 years, the maximum acceleration of a single earthquake which is forecast to occur during that period is 254 gal based on seismic risk analysis. The return period of an earthquake with a maximum acceleration of 508 gal, or one twice as strong, is 628 years, and by designing the structure taking this maximum acceleration into account, it is possible to satisfy the following requirements.

- No damage from an intermediate earthquake with a return period of a few decades.
- Slight damage or less from a strong earthquake with a return period of about 100 years.
- No failure from an extremely strong earthquake with a return period of about 600 years.
- Only minor damage from many intermediate earthquakes occurring during its lifetime of 100 years.

This does not mean that the standard design horizontal seismic coefficient should be doubled. Judging from the results in section 5., increasing the resistance of a reinforced concrete structure is a more effective way of improving the seismic resistance of such a structure than simply increasing its strength. Therefore this must be reflected in the form of improvements to the verification of the ultimate lateral strength during an earthquake or verification using the seismic coefficient in dynamic analysis, or the input acceleration in the Road Bridge Guidelines[2].

And in contrast to the 1990 Guidelines in which the seismic coefficient used to verify the ultimate lateral strength during an earthquake is 1G, in the Restoration Specifications this figure is nearly double. But because the basis for the calculations differs, the Restoration Specifications are not identical to those considering the cumulative damage. This approach is also an effective way of responding to the widely held fear of extremely powerful earthquakes and to reduce long term maintenance costs.

7. CONCLUSIONS

The following conclusions have been reached based on the results of this research.
(1) Damage indices for bending and shearing have been defined by a simplified formula using the plastic factor and an earthquake resistance diagnosis equation, and their correspondence

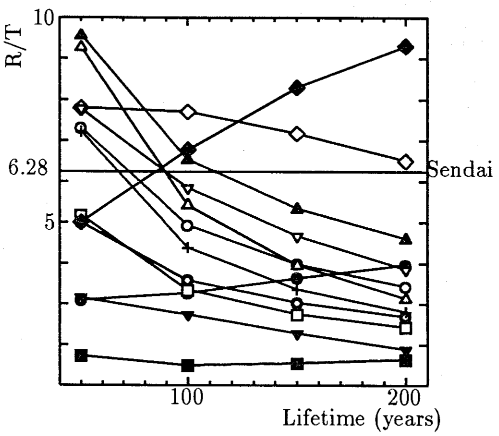

Fig. 6 Multipliers for Sendai (10 Damage Index Cases)

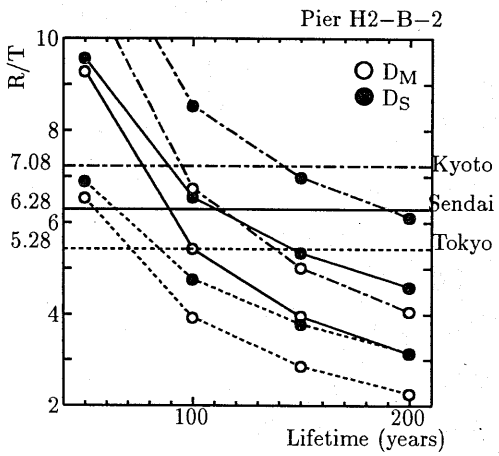

Fig. 7 Examples of Multipliers at Sendai, Tokyo, and Kyoto

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 insufficient 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 hypothesize a subsequent seismic force double that of a single earthquake forecast at the location of the structure during its lifetime.

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