METHOD OF EVALUATING EARTHQUAKE SAFETY OF RC BRIDGE SYSTEM BASED ON RELIABILITY THEORY

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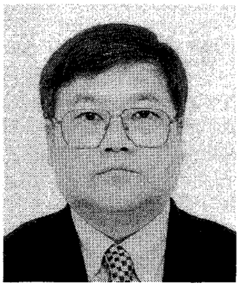

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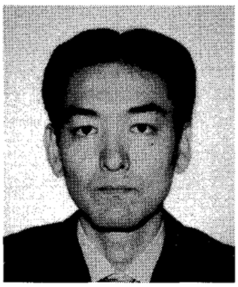
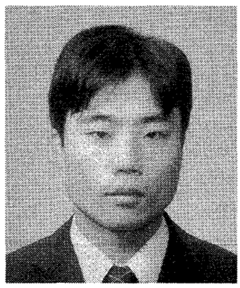

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This study proposes a method of calculating the failure probability of a structure and its structural members during an earthquake. Reliability theory and a Markov process are used to calculate the failure probability of a structure and the probability of damage to its structural members from the time the earthquake commences up to any arbitrary time. The characteristics of these techniques mean they are able to quantitatively evaluate changes in damage over time based on failure probability and damage probability. On the basis of the proposed method, a safety evaluation is carried out on a bridge system consisting of bearings, RC piers, and pile foundations during an earthquake. This evaluation elucidates the effects of failure of and damage to bearings, bridge piers, and pile foundations on the safety of the bridge system as a whole.

Key Words : *system reliability, RC bridge, failure probability, damage probability, seismic design.*

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1. INTRODUCTION

The 1996 Design Specifications for Highway Bridges [1] stipulate seismic performance requirements for the bridge foundations. They are expected to maintain a level of performance such that, in the case of a strong earthquake motion with an extremely low probability of occurring during the structure's lifetime (Level 2 earthquake motion), bridge functionality can be restored soon after the earthquake without the need for reinforcement (Seismic Performance 2). No earthquake should cause overall failure of the structure (Seismic Performance 3). However, present-day seismic design methods provide only for safety verifications of each individual member of the structure. As a consequence, it is difficult to accurately evaluate the safety of a bridge as a complete system because a bridge may be influenced not only by inertial forces of the superstructure, but also by dynamic interactions between the ground and the structure [2], [3]. For example, in the case of a bridge pier supported by a pile foundation, it is impossible to account for the effect of the many other members of the bridge in verifying an individual member.

For this reason, the safety of a structure during an earthquake should be evaluated by an earthquake response analysis that considers a total system model and accounts for each member. Further, it is necessary to predict a bridge's damage state after a hypothetical seismic event in order to allow for importance-based design. A number of investigations have attempted to clarify the state of a bridge system after an earthquake. For example, Takeda et al. mapped the locations of viaducts damaged in the Hyogo-ken Nanbu Earthquake in a computer, and then used GIS to study the structures, ground conditions, and earthquake motion. In this way, they were able to carry out comprehensive bridge damage factor analysis by evaluating damage from a variety of perspectives [4]. They also developed a seismic resistance diagnosis method that can be used to evaluate the seismic resistance of existing bridges by a relatively simple method, and proposed a diagnosis process aimed at guaranteeing the seismic resistance of a total bridge system. However, the Takeda study does not explicitly account for various uncertainties interposed in the bridge system in the case of a certain hypothetical seismic external force, and cannot clarify to what extent an analyzed bridge will remain safe in the event of the hypothetical earthquake.

The uncertainties generally relate to material strengths, structural modeling, the equations used for yield strength calculations, etc. when verifying the safety of a structure. Structural modeling, for example, should yield a model that is simple and precise, yet simplicity and analytical precision are contradictory aims [5]. For this reason, the uncertainties involved need to be rationally considered when constructing a design system. That is, RC bridge system should be designed through the quantitative evaluation of these uncertainties based on probability theory. This will ensure that not only individual parts and members, but also the overall bridge system, maintains its stipulated margin of safety.

In order to develop a design method that ensures the stipulated level of safety for an entire bridge under some arbitrary earthquake motion, the state of the system with respect to failure of and damage to its components must be evaluated over time. In order to achieve such probabilistic evaluations, probability values for the overall system should be derived from probability values calculated for each component of each member. This would then allow for a quantitative clarification of various possible states: (1) the bridge system is undamaged and can be used, (2) it is damaged and must be repaired, or (3) it has failed and must be rebuilt.

This study begins by establishing these three possible bridge system states, as shown in Table 1, and calculating the probability of each state arising. A comparative study of the probability of each state is implemented to clarify the likely state of the bridge system after an earthquake. Based on these results, the failure mode of the bridge system is clarified using probability values obtained from each member.

On the basis of this study, a method of applying reliability theory to take into account various uncertainties interposed in a bridge system is proposed. Using this method, the safety of a bridge system and changes in failure and damage probability over time, both for the bridge system and for its component parts, can be quantified. As a test, the proposed method is applied to the pile foundation yield strength and bearing condition of a bridge system consisting of bearings, RC bridge piers, and pile foundations in order to look into their effects on the safety of the bridge system as a whole.

Table 1. State of Bridge Systems by Damage Category

Damage Category	State of Bridge System
Undamaged (safety probability)	Repair unnecessary
Damaged (damage probability)	Repair necessary
Failed (failure probability)	Repair necessary

2. PROPOSED SAFETY EVALUATION METHOD

2.1 Outline

The earthquake safety of a bridge system is evaluated by successively calculating the probability of it reaching the ultimate state (the "failure probability") and the probability of it being damaged (or reaching a limit state prior to the ultimate state) (the "damage probability") during an earthquake. A method of evaluating safety based on changes over time in these probabilities is also proposed.

2.2 Failure Probability Calculation Method

a) Modeling the Bridge System

The bridge system considered here is a system consisting of bearings, bridge piers, and pile foundations. The ultimate state of the bridge system was defined as the state when any one of its parts reaches the ultimate state. The ultimate state of individual parts is any one of the conceivable ultimate states for that part, as indicated in Chapter 3. Considered this way, it becomes possible to model the bridge system as a serial system comprising three parts, as shown in Figure 1.

Figure 1. Serial Model of a Bridge System

b) Instantaneous Failure Probability Calculation Method

First, the secondary moment method is used to calculate the probability of failure at a certain time for the set of limit states applicable to each part (the "instantaneous failure rate").

Then a method of structural system reliability evaluation [6] able to simply and accurately compute a safety index while accounting simultaneously for a number of the limit states proposed by the authors is used to calculate the instantaneous failure rate of a part from these instantaneous failure rates.

Finally, Equation (1) is used to calculate the instantaneous failure rate of the bridge system from the instantaneous failure rates of each part.

$$P_{fsys}(t) = 1 - \prod_{i=1}^{3}[1 - P_{fi}] \quad (1)$$

Where, $P_{fsys}(t)$: instantaneous failure rate of bridge system, $P_{fi}(i = 1,2,3)$: instantaneous failure rates of each member or part.

c) Successive Failure Probability Calculation Method

If it is assumed that failure does not occur by time t and that the instantaneous failure rate $P_f(t)$ at time t is the conditional probability of failure during the next time increment, $P_f(t)$ can be represented by Equation (2) below [7].

$$P_f(t) = \frac{\phi(t)}{R(t)}$$

$$= \frac{d\Phi(t)}{dt} \cdot \frac{1}{R(t)} \tag{2}$$

$$\text{or} \quad = \frac{\dot{\Phi}(t)}{R(t)} (\Phi(t) + R(t) = 1)$$

$$= -\frac{\dot{R}(t)}{R(t)}$$

Where, $\phi(t)$: probability density function, $\Phi(t)$: cumulative density function, $P_f(t)$: instantaneous failure rate, $R(t)$: probability of failure not occurring at time t ("reliability").

Integrating this equation by time to obtain the reliability at any time t_i $R(t_i)$, and then substituting this into the failure probability $PF(t_i)$ versus reliability relationship represented by $PF(t_i) + R(t_i) = 1$ yields equation (3) [7].

$$PF(t_i) = 1 - \exp\left[-\int_0^{ti} P_f(t)dt\right] \tag{3}$$

This demonstrates that it is possible to calculate the failure probability PF of the bridge system, of each part, and each limit state based on the instantaneous failure rate for any time t_i after the start of an earthquake.

2.3 System Successive Damage Probability Calculation

a) Method of Calculating Bridge System's Successive Damage Probability

Takahashi et al. [8] used the Markov process to categorize the state of a structure into three categories: undamaged, damaged, and failed. This is shown in Figure 2. The undamaged state was assumed to be the state before member stress reaches the yield point. The damage state was assumed to be the state when the member stress has passed the yield point but has not yet reached the ultimate strength. The failed state was assumed to be when the member stress has reached the ultimate strength.

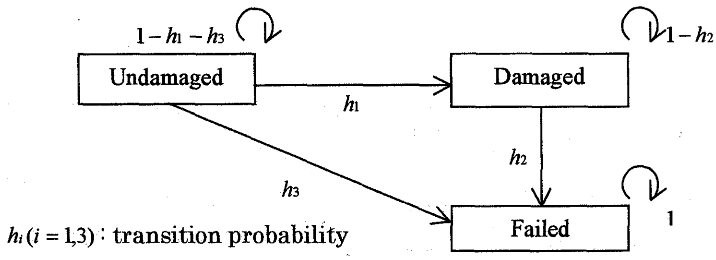

Figure 2. Change of State

In this study, the same concept is applied to a bridge system during earthquake motion. In brief, the model was expanded to include five categories of damage to a bridge system, as shown in Figure 3: the undamaged state, a state where one part is damaged ("1-component damage"), a state where two parts are damaged ("2-component damage"), a state where all parts are damaged ("total damage"), and the failure state. The undamaged state corresponds to Seismic Performance 1 (where repairs are unnecessary), while the failure state corresponds to a state which fails to satisfy Seismic Performance 2 (where reinforcement is necessary). The other damage states correspond to states that satisfy Seismic Performance 2 (where repair is necessary but reinforcement is unnecessary).

In a case where a member being analyzed changes from the undamaged state to a damaged state, the instantaneous damage rate of the member is used as the transition probability of the Markov process. Also, where a member changes from the undamaged state or from a certain damaged state to the failure state, the instantaneous failure rate of the bridge system is used as the transition probability of the Markov process.

This allows for a quantitative calculation of the probability that a bridge system is in the undamaged state (the "safety probability"), the damage probability, and the failure probability.

b) Conformity between Successive Failure Probability Calculation and Successive Damage Probability Calculation

The failure probability of a bridge system as calculated using the successive failure probability method must be identical to the probability of failure as calculated by the successive damage method. The use of the transition equation of the Markov process to ensure identical values from the two methods is explained below.

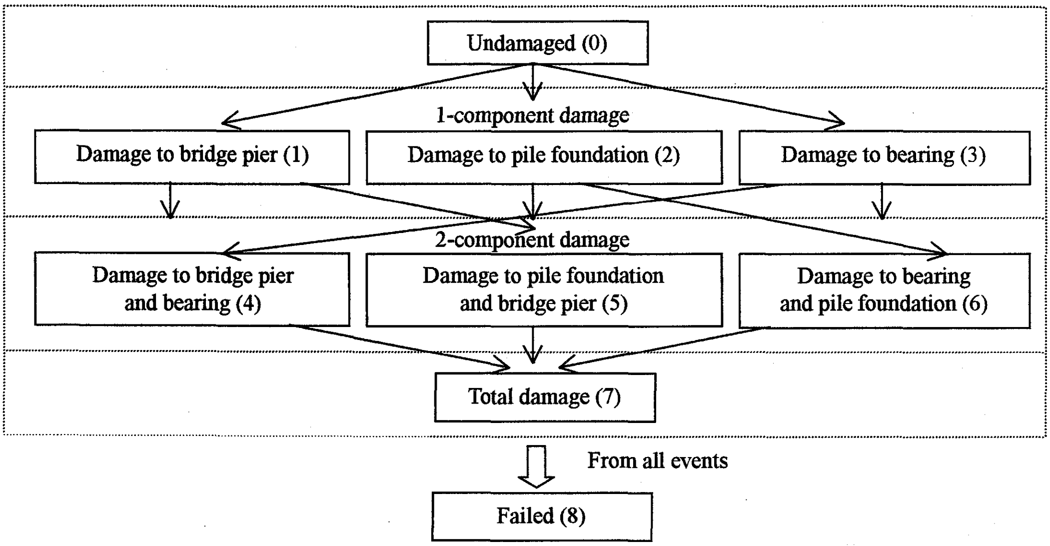

Figure 3. Damage Change Model of Bridge Analyzed

The equation for the transition from each phenomenon to failure is Equation (4).

$$P_8(t+dt) = P_{fsys}dt\sum_{i=0}^{7} P_i(t) + P_8(t) \tag{4}$$

Where, P_0: probability of being in undamaged state, $P_i(i=1,\cdots,7)$: probability of being in a damaged state, P_8: probability of being in failure state. The value of i in $P_i(i=1,\cdots,7)$ corresponds to the value in ( ) in Figure 3.

If it approximates $P_8(t+dt) \approx P_8(t) + \dot{P}_8(t)dt$, or it is substituted in $\sum_{i=0}^{8} P_i = 1$, the probability of the bridge system being in the failure state is represented by Equation (5).

$$P_8(t) + \dot{P}_8(t) = P_{fsys} \cdot dt(1 - P_8(t)) + P_8(t)$$

$$\frac{\dot{P}_8(t)}{1 - P_8(t)} = P_{fsys}$$

$$P_8(t) = 1 - \exp\left[-\int_0^t P_{fsys}(t)dt\right] \tag{5}$$

Here, the failure probability equation is the same as Equation (3) [9], [10].

2.4 Bridge System Safety Evaluation Procedure

A safety evaluation on a bridge is carried out using the above failure probability calculation method and the successive damage probability calculation method. The evaluation method proposed in this study is described below.

1) The secondary moment method is used to calculate the instantaneous failure rate for the ultimate limit state.
2) The structural system reliability evaluation method is used to calculate the instantaneous failure rate of each member.
3) Equation (1) is used to calculate the instantaneous failure rate of the bridge system.
4) The secondary moment method is used to calculate the instantaneous damage rate for the damage limit state.
5) The probabilities of each of the states defined in the model are calculated from the instantaneous failure rate and the instantaneous damage rate of the bridge system in accordance with the Markov process.

Steps 1) to 3) above represent the failure probability calculation procedure for a bridge system, and steps 4) and 5) are the failure probability and damage probability calculation procedure using the Markov process.

3. BRIDGE SYSTEM SEISMIC SAFETY ANALYSIS METHOD

3.1 Outline

A safety evaluation using reliability theory is a means of evaluating the safety margin of an analyzed structure at the defined limit state (in this study, yield strength — external force). It is, therefore, necessary to appropriately establish an ultimate limit state and damage limit state based on records of earthquake damage. Further, it is also necessary to appropriately set a mean value and scatter (coefficient of variance) for the stochastic variable used by the established limit state equation.

The following is an explanation of this limit state equation and of the mean value and uncertainty in the stochastic variable that it uses.

3.2 Setting the Limit States

a) Setting the Damage Limit State
The ductility design method stipulated in Design Specifications for Highway Bridges [1] allows for plasticization of the bridge pier foundation and non-linear response of seismic isolation bearings when strong seismic forces act on the structure. However, considering the difficulty entailed in surveying and repairing damage to pile foundations, these should in fact be provided with seismic performance one rank higher than that required for bridge piers and bearings [11]. For this reason, taking Seismic Performance 1 and 2 as the boundary, the damage limit state of a bridge pier is the point at which plasticization of the bridge pier base begins. For a bearing, it is the point when non-linear response begins, and for a pile foundation it is the point at which bending cracks occur at the pile head. In brief, the damage limit state of a bridge pier, pile foundation, and bearing are defined as described below.

Damage Limit State of Bridge Pier
Taking the damage limit state of a bridge pier to be the point at which the bending moment acting on the pier base reaches the bending yield strength, the limit state equation is defined as given by Equation (6).

$$g_{D1} = \alpha_{D1} M_{yD} - M_{S1} \tag{6}$$

Where: α_{D1} : coefficient accounting for uncertainty in the equation, M_{yD} : bending yield strength, and M_{S1} : bending moment acting on the bridge pier base.

Damage Limit State of Pile Foundation
Taking the damage limit state of a pile to be the point at which the bending moment acting on a pile head in the center row (of the three rows in the bridge pier analyzed in Figure 4) reaches the load that causes bending cracks, the limit state equation is defined as in Equation (7).

$$g_{D2} = \alpha_{D2} M_c - M_{S2} \tag{7}$$

Where, α_{D2} : coefficient accounting for uncertainty in the equation, M_c : Moment that causes bending cracks, and M_{S2} : bending moment acting on pile head in the center of the three rows.

Damage Limit State of Bearing
Taking the damage limit state of a rubber bearing to be the point at which non-linear deformation occurs in the rubber, the limit state equation is defined as given in Equation (8).

$$g_{D3} = \alpha_{D3} \delta_R - \delta_{Shoe} \tag{8}$$

Where, α_{D3}: coefficient accounting for uncertainty in the equation, δ_R : displacement of the point of change of the stiffness, and δ_{Shoe} : relative displacement of the superstructure with respect to the top of the substructure.

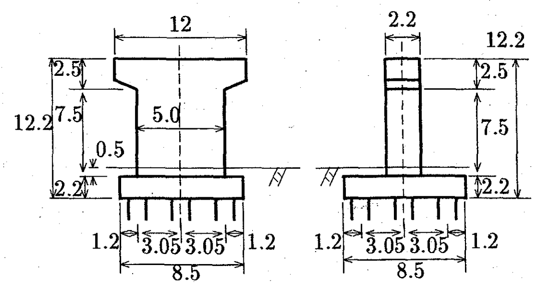

Figure 4. Bridge Pier Analyzed

b) Setting of Ultimate Limit State
Next, the ultimate limit states of the bridge pier, pile foundation, and bearing are set as indicated below with

Seismic Performance 2 and 3 as the boundary.

Ultimate Limit State of Bridge Pier

The ultimate limit state of a bridge pier is determined based on the bending yield strength, shear yield strength, and deformation performance. These various limit state equations are written as shown in Equations (9) to (11), respectively.

$$g_1 = \alpha_1 M_u - M_{S1} \tag{9}$$

$$g_2 = \alpha_2 (V_{c1} + V_{s1}) - \frac{M_{S1}}{a} \tag{10}$$

$$g_3 = \alpha_3 \left[\frac{N}{N_B} + \left(1 - \frac{N}{N_B}\right)\left\{12\left(\frac{0.5V_{c1} + V_{s1}}{M_u / a}\right) - 3\right\}\right] - \frac{\delta}{\delta_y} \tag{11}$$

Where, M_u: bending yield strength of bridge pier, V_{c1}: shear yield strength of bridge pier attributable to concrete, V_{s1}: shear yield strength of bridge pier attributable to steel reinforcement, N: axial compressive strength, N_B: axial compressive strength at time of balance failure, δ_y: yield displacement, a: shear span, M_{S1}, δ: bending moment and response displacement obtained by dynamic analysis, α_1, α_2: coefficients accounting for the scatter in the yield strength calculation equation, and α_3: coefficient accounting for the scatter in the ductility calculation equation.

Ultimate Limit State of Pile Foundation

The ultimate limit state of a pile foundation is assumed to be yielding and shear failure of the pile. The yield point, in accordance with the Design Guidelines for Highway Bridges [12], is defined as either 1) the time of bending failure of all piles or 2) the time at which the pile head reaction force in one row reaches the upper limit of bearing strength. The ultimate limit state of a pile foundation is verified based on bending yield strength and shear yield strength of the piles and on the bearing strength in the axial direction. These limit state equations are as shown in Equations (12) to (14), respectively.

$$g_4 = \alpha_4 M_y - M_{S2} \tag{12}$$

$$g_5 = \alpha_5 (V_{c2} + V_{s2}) - P \tag{13}$$

$$g_6 = \alpha_6 \left(q_d A + U \sum L_i f_i\right) - P_H \tag{14}$$

Where, M_y: bending yield strength of the pile, V_{c2}: shear strength of the pile attributable to the concrete, V_{s2}: shear strength of the pile attributable to the steel reinforcement, q_d: ultimate bearing strength per unit surface area of the pile end, A: pile end sectional area, U: circumference of the pile, L_i: thickness of the layer accounting for the skin friction force, f_i: maximum skin friction force of layer accounting for the skin friction force, M_{S2}: bending moment acting on pile head in the center of the three rows, P: lateral force acting on the pile, P_H: axial compression force acting on the pile body, α_4, α_5: coefficients accounting for uncertainty in the yield strength equation, and α_6: coefficient accounting for uncertainty in the bearing strength equation.

Ultimate Limit State of Bearing

The ultimate limit state of a bearing is set according to the damage caused by the Hyogo-ken Nanbu Earthquake [13], because the connections between superstructure and substructure consist of a variety of components including bearings and unseating-prevention structures.

Taking the ultimate limit state of a bearing to be either the time at which the bolts holding the bearing attachments break and the shear strain in the rubber bearings reaches the failure strain, or as the time at which the relative displacement of the superstructure and the top of the substructure exceeds the seat length, the respective limit state equations are as shown in Equations (15) to (17).

$$g_7 = 1.2 - \alpha_7 \left\{ \left(\frac{\sigma_s}{\sigma_a}\right)^2 + \left(\frac{\tau}{\tau_a}\right)^2 \right\} \tag{15}$$

$$g_8 = 4.0 \alpha_8 \sum_{i=1}^{n} t_i - \delta_{Shoe} \tag{16}$$

$$g_8 = \alpha_9 S_E - \delta_{Shoe} \tag{17}$$

Where, σ_a: allowable tensile stress of the bolts, τ_a: allowed shear stress of the bolts, $\sum_{i=1}^{n} t_i$: height of the bearing rubber, S_E: seat length or length from center of bearing to top end of substructure (the seat length is calculated as $S_E = 70 + 0.5\, l$ *($l \leq 100$ m)*, $S_E = 80 + 0.4 l$*($l \geq 100$ m)*) [14], l: span, σ_s: tension stress acting on the bolts τ : shear stress acting on the bolts, δ_{Shoe} : relative displacement between the superstructure and the top of the substructure, and $\alpha_7, \alpha_8, \alpha_9$: coefficients that account for uncertainty in the equations. The failure strain of the seismic isolation bearings is hypothesized to be 400% in this study based on the Design Specifications for Highway Bridges [1], which stipulate the failure strains of natural rubber and chloroprene rubber as 500% and 400%, respectively (both being reference values).

Table 2. Mean Value and Coefficient of Variation of Stochastic Variable of the Damage Limit State

Component	Stochastic variable	Mean value	Coefficient of variation
Bridge pier	M_{yD}	Calculated value	10%
	M_{S1}	Result of Response	30%
	α_{D1}	1	10%
Pile foundation	M_c	Calculated value	10%
	M_{S2}	Result of Response	30%
	α_{D2}	1	20%
Bearing	δ_R	Calculated value	20%
	δ_{Shoe}	Result of Response	30%
	α_{D3}	1	20%

c) Stochastic Variables of the Limit State Equations

The mean values of the stochastic variables of the defined limit state equations, their coefficients of variation, their probability distributions, and the correlations between them are hypothesized and explained below.

All stochastic variables are assumed to fit the normal distribution, and correlations between them are not taken into account. The parameters were set as shown in Tables 2 and 3. The calculated values in these tables are the mean values of the yield strength term as calculated according to the Design Guidelines for Highway Bridges [1], [12], and the response values are the mean values of the external force term obtained by seismic response analysis. The shear span, *a,* is constant.

The coefficient of variation of pile bearing strength was taken from the results of experiments by Okahara et al. [15] on the bearing strength of a pile in various types of ground. The bending yield strength, shear yield strength, and deformation performance of reinforced concrete were based on results obtained previously by the authors [6]. The coefficient of variation of the bearings was set in consideration of the materials and structure of the bearings. Correction factor α was selected to take into account the precision of the dynamic analysis.

Table 3. Mean Value and Coefficient of Variation of Stochastic Variable of the Ultimate Limit State

Component	Stochastic variable	Mean value	Coefficient of variation
Bridge pier	M_u, V_{s1}	Calculated value	8%
	V_{c1}, δ_y	Calculated value	10%
	N, N_B	Calculated value	5%
	M_{S1}, δ	Result of Response	30%
	$\alpha_1, \alpha_2, \alpha_3$	1	10%
Pile foundation	M_y, V_{s2}	Calculated value	8%
	V_{c2}	Calculated value	10%
	$q_d A$	Calculated value	58%
	$U\sum L_i f_i$	Calculated value	41%
	M_{S2}, P, P_H	Result of Response	30%
	α_4	1	10%
	α_5, α_6	1	20%
Bearing	$\sigma_a, \tau_a, S_E, \sum_{i=1}^{n} t_i$	Design value	10%
	$\sigma_s, \tau, \delta_{Shoe}$	Result of Response	30%
	$\alpha_7, \alpha_8, \alpha_9$	1	10%

3.3 Bridge System Earthquake Safety Evaluation Procedure

The procedure for evaluating the earthquake safety of a bridge system using the proposed method is as follows.

1) Select the bridge and ground to be analyzed
2) Establish the limit state equations for the damage limit state and the ultimate limit state
3) Calculate the yield strength term

to be used by the limit state equations

4) Calculate the external force term used by the limit state equations using earthquake response analysis.
5) Calculate the failure probability and damage probability by following the methods proposed in section 2.
6) Repeat steps 3) to 5) above for each time increment in the analysis time period.

Table 4. Specifications of Bridge Analyzed

Span		40m×5 span
Bearing	height × layer	1.4cm×11 layer
	number	5
	bolt	M42 - 8 bolts
Bridge pier	Section	5.0m× 2.2m
	Axial reinforcing bars	D32×182
	Hoop ties	D16 ctc 150
Pile foundation	Diameter and length of pile	$\phi = 1.2m, 16.0m$
	Axial reinforcing bars	D22×20
	Hoop ties	D22 ctc 125

3.4 Choice of Bridge and Seismic Waveform for Analysis

Table 5. Materials Used

Component	Concrete	Reinforcing Bar	Bolt
Bearing			Strength classification :4.6
Bridge pier	σ_{ck} = 20.58 MPa	SD295	
Pile foundation	σ_{ck} = 23.52 MPa	SD295	

The pier selected for analysis in this study is an intermediate RC bridge pier and pile foundation in a 5-span continuous steel I-girder bridge stipulated in Document Concerning Seismic Design of Road Bridges [16]. Details of this bridge are given in Figure 4 and Table 4, and the materials used are listed in Table 5. The pier studied is RC pier with a horizontal capacity of 4.50 MN and a shear yield strength of 7.08 MN. The pile foundation is a cast-in-place pile foundation, and the seismic coefficient corresponding to the foundation's yield point described above (and abbreviated here to "yield seismic coefficient") is 0.50. The bearings are hypothesized as isolation bearings. The results of eigenvalue analysis yielded a primary natural period of 0.94 seconds.

The seismic waveforms used for the analysis consisted of one representing an earthquake directly below the bridge and a plate-boundary earthquake. The waveform for the earthquake directly below the bridge was the acceleration record obtained during the Hyogo-ken Nanbu Earthquake at a point 32 m below Port Island. This is viewed as a bedrock wave form. The waveform for the plate-boundary earthquake was obtained by applying the general purpose program SHAKE to an acceleration waveform obtained during the Miyagi-ken Oki Earthquake. A surface observation taken at the Kaihoku Bridge was used to estimate the bedrock surface acceleration waveform, and this was adopted as the bedrock waveform for analysis. To compare the two types of earthquake, the two acceleration waveforms were adjusted such that the maximum acceleration input into the bedrock was 500 gal.

The ground was hypothesized to be ground type II; this is reasonable, given the choice of pile foundations and isolation bearings for the bridge. The ground was assumed to have a natural period (T_G) of 0.43 seconds, as shown in Figure 5.

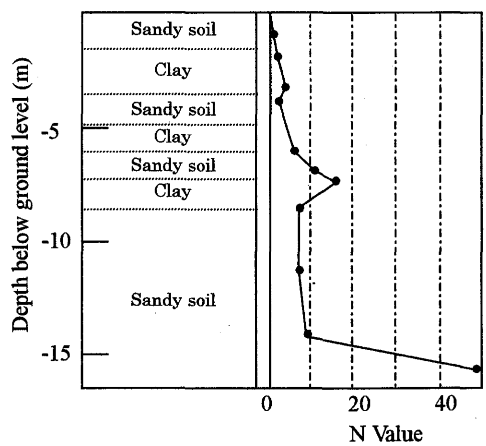

Figure 5. N Value of Ground Analyzed

3.5 Earthquake Response Analysis

This bridge was modeled based on Penzien's model [17], [18], as shown in Figure 6. The input motion was the response displacement of the natural ground, as calculated using overlapping reflection theory, with an interaction spring inserted between the pile foundation and the ground. The structural members of the bridge pier were modeled as explained below.

The isolation bearing was modeled as a non-linear spring

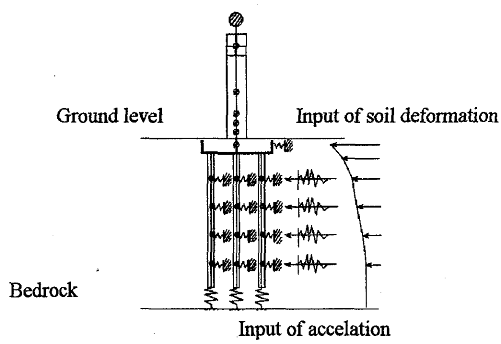

Figure 6. Analytical Model

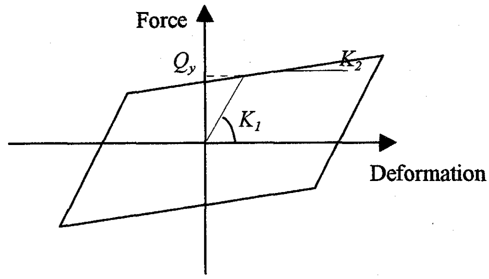

Figure 7. Bilinear Model of the Seismic Isolation Bearings

with bilinear restoring force characteristics, as shown in Figure 7. The break points were set as shown in Table 6 in accordance with the Road Bridge Seismic Isolation Design Manual (Draft) [19]. The bridge pier and pile foundation were modeled using non-linear bridge elements, with the Takeda model providing the restoring force characteristics. Static elasto-plastic analysis was used to calculate their break points. As noted above, a non-linear spring was inserted between the pile foundation and the ground, and complete elasto-plastic restoring force characteristics were set in compliance with the Design Guidelines for Highway Bridges [12].

The damping factor was 0% for the isolation bearing, 2% for the bridge pier and pile foundation, and 20% between the foundation and ground (based on a prediction of radiation damping). It was set at 0% for the footing because it is a rigid-body and Rayleigh damping was specified.

Table 6. Stiffness and Yield Load of Seismic Isolation Bearings

Elastic stiffness K_1	Post-yield stiffness K_2	Yield Strength Q_y
47.54 MN/m	9.07 MN/m	0.77 MN

The earthquake response calculations were performed using an incremental method based on Newmark's β method ($\beta = 1/4$). This earthquake response analysis of the bridge as modeled by a spring–mass point system yielded the external force term in the limit state equations. Seismic motion was input in the direction of the bridge axis.

4. EVALUATION OF BRIDGE SYSTEM EARTHQUAKE SAFETY

4.1 Outline

The method outlined in section 3 was used to evaluate the earthquake safety of the bridge system under study. First, the proposed evaluation method was used to calculate the failure probability and the damage probability of the bridge system, its members, and the limit states. This indicates the safety of the bridge system under the input seismic motion. Next, with the focus now on the pile foundation, a similar safety evaluation was performed on the bridge system with a greatly strengthened pile foundation in order to study the safety of the bridge system under the input seismic motion.

4.2 Earthquake Safety Evaluation of Bridge System with Miyagi-ken Oki Earthquake Motion

The safety of the bridge system described in Table 4 was verified for a case where the Miyagi-ken Oki Earthquake seismic waveform was input. First, the failure probability of the entire bridge system by the end of the earthquake, the probability of each damage state (g_{D1}, etc.) (damage probability), the probability of damage to all components (total damage probability), and the probability of each of the hypothetical damage states and of the failure state not occurring (safety probability) were calculated. The results are shown in Table 7. In this table, the 1-component damage probability represents the probability of a state in which one component is damaged, and the 2-component damage probability represents the probability that two components are damaged.

An examination of the safety probability and failure probability in a case where the yield seismic coefficient of the pile is 0.5 reveals that the safety probability of the system is higher than the failure probability; that is, the probability of no damage occurring is higher than the probability of failure. A comparison of the various damage probabilities reveals that the bearing has the greatest probability of being damaged; that is, most damage occurs in the bearing. However, because the damage probabilities of both the bearing and the pile foundation exceed their safety probabilities, damage to the bridge system occurs primarily in the bearing, and there is likelihood of secondary damage in the pile foundation.

Table 7. Safety Probability, Damage Probability and Failure Probability of the Bridge System

Type of Seismic wave		Plate-boundary	Plate-boundary	Inland
Yield seismic coefficient of bridge pier		0.68		
Yield seismic coefficient of pile foundation		0.5	1.1	0.5
Safety probability		0.147	0.256	0.036
1-component damage probability	g_{D1}	0.000	0.001	0.007
	g_{D2}	0.095	0.089	0.072
	g_{D2}	0.427	0.452	0.189
2-component damage probability	g_{D1},g_{D2}	0.000	0.000	0.013
	g_{D2},g_{D3}	0.277	0.156	0.385
	g_{D3},g_{D1}	0.000	0.001	0.036
Total damage probability		0.000	0.000	0.072
Failure probability		0.054	0.045	0.190

To provide a comparison with a case where secondary damage is prevented, a similar safety evaluation was carried out but with yield seismic coefficient of the pile foundation raised to 1.1.

Table 7 shows the calculated stochastic probability, damage probability, and safety probability for the bridge system with this higher yield strength of the pile foundation. A comparison of this with the a case where the yield seismic coefficient of the pile foundation was 0.5 (plate-boundary type earthquake wave) shows that the safety probability at a pile yield seismic coefficient of 1.1 is greater than that at a yield seismic coefficient of 0.5. A comparison of the damage probabilities reveals that the higher yield seismic coefficient reduces the 2-component damage probability but increases the 1-compoenent damage probability, while the safety probability rises. A comparison of the two failure probabilities shows that, proportionally, the decline in failure probability resulting from the increased yield seismic coefficient is less than the increase in the safety probability. That is, raising the yield seismic coefficient of the pile reduces damage to the pile but slightly increases damage to the bearings, leading to a slight increase in the safety of the overall system.

4.3 Earthquake Safety Evaluation of Bridge System with Hyogo-ken Nanbu Earthquake Motion

The safety of the same bridge system was verified for a case where the Hyogo-ken Nanbu Earthquake seismic waveform was input. The failure probability of the entire bridge system, the probability of each damage state (g_{D1}, etc.), the total damage probability, and the safety probability were calculated using the proposed method. The results are shown in Table 7.

Judging from the results, the combined damage probability and failure probability of the bridge system exceeds the safety probability, and there is a high risk of the bridge system reaching a damaged state or failure. A comparison of these results with those for the plate-boundary seismic motion (at a pile yield seismic coefficient of 0.5) reveals higher failure probability in the Hyogo-ken Nanbu Earthquake scenario, and the governing state of the bridge system includes more damaged members. That is, the bridge system analyzed here would suffer greater damage if exposed to the Hyogo-ken Nanbu Earthquake seismic waveform than the Miyagi-ken Oki Earthquake seismic waveform.

The changes over time of the failure probability of the bridge system and its parts was studied with the focus on failure state. Figure 8 shows how the failure probability of the bridge system and its parts changes over a period of 15 seconds beginning with the onset of the earthquake. This demonstrates that the failure probability of the bridge system is influenced by the failure probability of the pier and the pile foundation. It also shows that at the time of maximum response (4 to 5 seconds after onset) the pier failure probability exceeds the failure probability of the pile foundation. However, at 10 seconds the pile foundation shows a greater failure probability than the pier. The

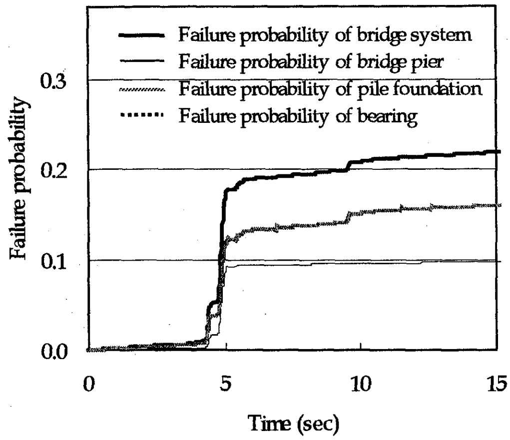

Figure 8. Failure Probability of Bridge System and its Components at Pile Yield Seismic Coefficient of 0.50

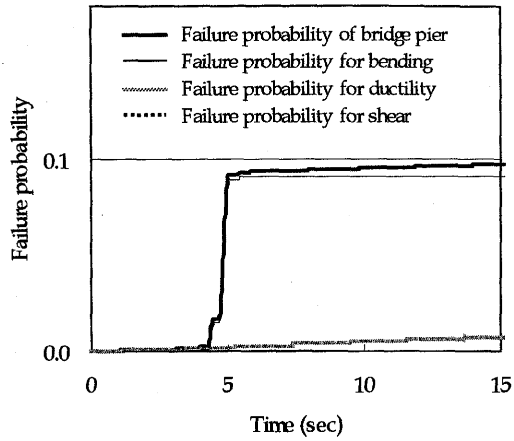

Figure 9. Failure Probability at Ultimate Limit State of Bridge Pier at Pile Yield Seismic Coefficient of 0.50

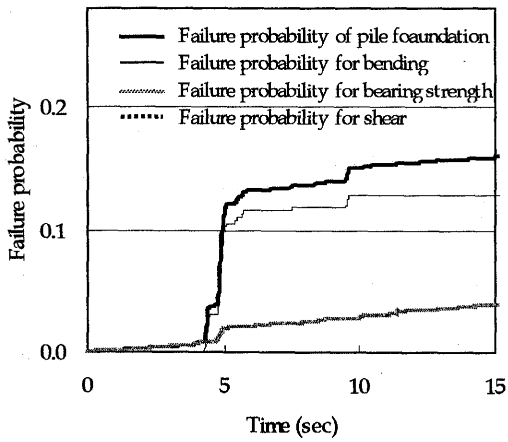

Figure 10. Failure Probability at Ultimate Limit State of Pile Foundation at Pile Yield Seismic Coefficient of 0.50

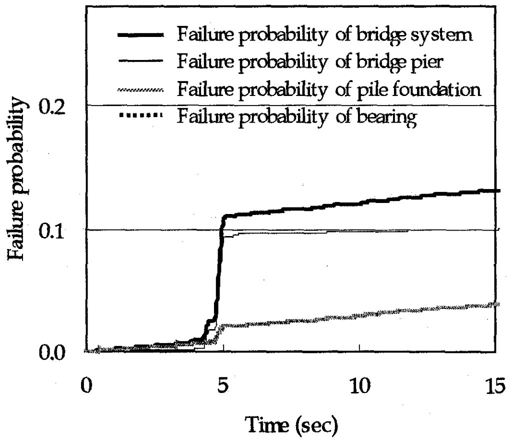

Figure 11. Failure Probability of Bridge System and its Components at a Pile Yield Seismic Coefficient of 0.95

failure probability of the bearing is almost zero. Given this result, the limit states governing each part were studied with the focus on the pier and the pile foundation.

Figure 9 shows changes over time in failure probability of the limit states of the bridge pier. This reveals that the bridge pier is governed by the bending limit state. The shear failure probability is almost zero. Next, Figure 10 shows changes over time in the failure probability of the limit states of the pile foundation. Here it is clear that the pile foundation is influenced by the limit states for bending and for bearing strength, but it is primarily governed by the bending limit state. The shear failure probability is almost zero.

Focusing on the bending limit state of the pile, the safety of the bridge system when the yield seismic coefficient of the pile foundation was set at 0.95 was evaluated. Figure 11 shows changes over time in the failure probability of the bridge system and all its parts from earthquake onset until 15 seconds later. This shows that the failure probability of the bridge system as a whole is governed by the failure probability of the pier. This is assumed to be a consequence of raising the bending yield strength of the pile foundation, which reduces the failure probability of the pile foundation and that the safety of the bridge system is governed by the safety of the bridge pier. In this analysis, also, the probability of bearing failure is almost zero.

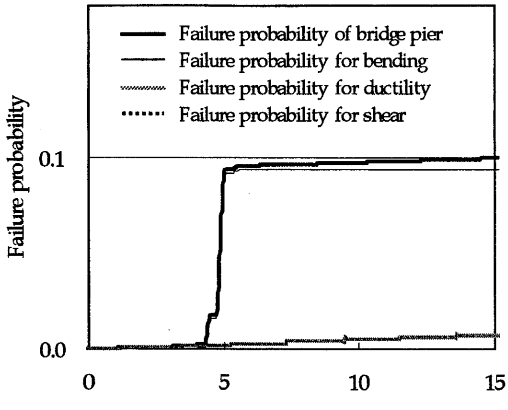

Figure 12. Failure Probability at Ultimate Limit State of Bridge Pier at Pile Yield Seismic Coefficient of 0.95

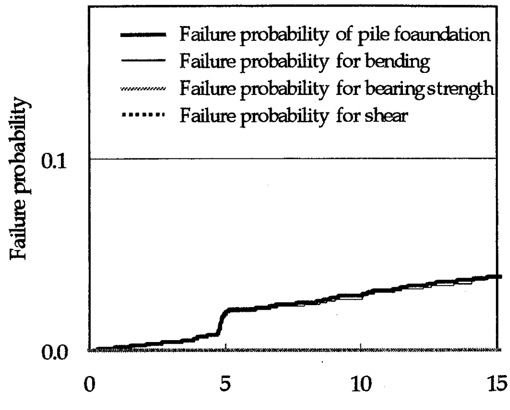

Figure 13. Failure Probability at Ultimate Limit State of Pile Foundation at Pile Yield Seismic Coefficient of 0.95

Next, the reason for pier failure dominating the failure probability of the bridge system as a whole is investigated by focusing on the bridge pier and the pile foundation. First, Figure 12 shows the changes over time in failure probability of the limit states of the bridge pier. This shows that the safety of the pier is determined by its safety under bending, as in the case of a pile foundation yield seismic coefficient of 0.50. Then Figure 13 shows the changes over time in the failure probability of the limit states for the pile foundation. This shows that increasing the yield strength of the pile foundation increases its safety under bending, and under these conditions the safety of the pile foundation is determined by its bearing strength. That is, when the bending yield strength of the pile foundation is increased, the limit state that determines the safety of the bridge system as a whole becomes the pier's bending limit state.

5. EFFECTS ON SAFETY OF BRIDGE SYSTEM OF PILE FOUNDATION YIELD STRENGTH

In section 4, it was demonstrated that the safety of the pile foundation under bending has a significant effect on the safety of the overall bridge system. Here, the effects of differences in the yield strength of the pile foundation on the safety of the bridge system are clarified.

The first step was to vary the bending yield strength of the pile foundation by changing the quantity of longitudinal steel reinforcing bars used; in other words, by performing trial design of the pile foundations with varying yield seismic coefficients. Then the effects on the safety of the bridge system of these variations in the pile foundation were studied on the basis of failure probability at the end of the earthquake event. Figures 14 and 15 show the results of safety evaluations of these trial-designed bridges when the Hyogo-ken Nanbu Earthquake seismic waveform was adjusted to 500 gal and input to the analytical ground.

First, Figure 14 shows the relationships between yield seismic coefficient of the pile and the bridge system, bridge pier, and pile foundation failure probabilities. This reveals that raising the yield strength of the pile foundation reduces the failure probability of the pile foundation and that of the entire bridge system, and that the safety of the bridge system is then governed by the safety of the bridge pier.

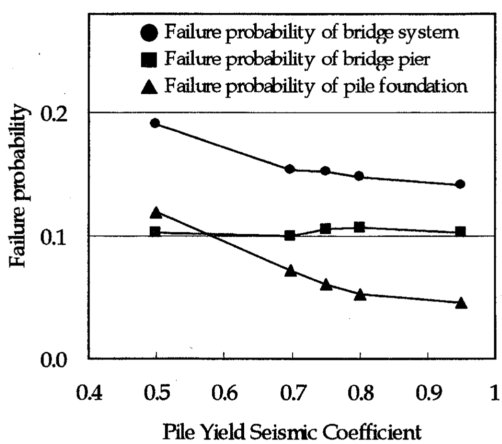

Figure 14. Relationship of Pile's Yield Seismic Coefficient to Failure Probabilities of Bridge System and with its Components

It also shows that increasing the yield seismic coefficient of the pile above 0.8 does not lead to any significant improvement in the safety of the pile foundation.

Next, Figure 15 shows the relationship between yield seismic coefficient of the pile and the failure probability for the ultimate limit state of the pile foundation. According to this figure, the limit state governing the pile foundation is bending of the pile body when the yield seismic coefficient of the pile is less than 0.6. On the other hand, if the yield seismic coefficient is greater than 0.6, the governing limit state becomes the bearing strength limit state. Then, if the yield seismic coefficient of the pile rises above 0.8, the bending failure probability of the pile falls to almost zero. Therefore, it can be concluded that in the analytical ground, providing the pile foundation with a yield seismic coefficient above 0.8 gives it greater yield strength than needed and does not necessarily result in an increase in safety of the pile foundation.

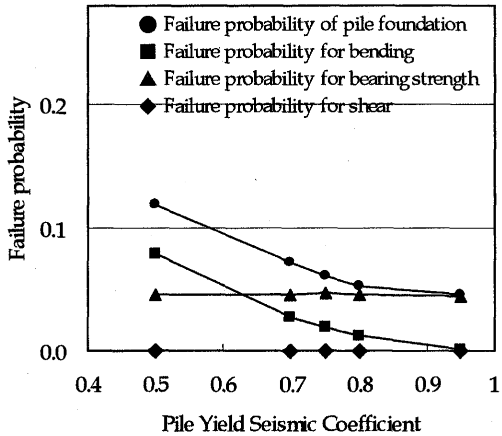

Figure 15. Relationship of Pile's Yield Seismic Coefficient to Failure Probability in Ultimate Limit State of Pile Foundation

6. EFFECTS ON THE SAFETY OF BRIDGE SYSTEMS OF SEISMIC ISOLATION SYSTEMS

6.1 Outline

At the bridge design stage, engineers generally consider the following three methods of lowering the lateral force that acts on the substructure during an earthquake [1].

1) Distributing the lateral force resulting from an earthquake among multiple bridge piers
2) Increasing the damping performance
3) Setting a suitably long period

Generally, seismic isolation systems make use of all three approaches, while non-isolation systems, such as elastomeric bearings, rely on 1) and 3).

Bridges with and without seismic isolation systems were trial designed for compliance with the Design Guidelines for Highway Bridges [1]. The effects on bridge system safety of the damping action of seismic isolation systems were studied by carrying out earthquake safety evaluations by the proposed methods.

Tables 8 and 9, respectively, give the specifications of the trial-designed bridges with seismic isolation bearings and with non-isolation bearings. The bearing materials are included in Table 5. The lead rubber bearings were modeled bilinearly, and their

Table 8. Specifications of Analyzed Bridge with Seismic Isolation System

Bearing	Type	Lead rubber bearing
	height × layer	1.4cm × 11 layer
	number	5
	bolt	M42 - 8 bolts
Bridge pier	Section	$5.0\text{m} \times 2.2m$
	Axial reinforcing bars	D29 × 144
	Hoop ties	D16 ctc 150
Pile foundation	Diameter and length of Pile	$\phi = 1.2m, 16.0m$
	Axial reinforcing bars	D22 × 20
	Hoop ties	D22 ctc 125

Table 9. Specifications of Analyzed Bridge without Seismic Isolation System

Bearing	Type	Elastomeric bearing
	height × layer	2.4cm × 4 layer
	number	5
	bolt	M42 - 8 bolts
Bridge pier	Section	$5.0\text{m} \times 2.2m$
	Axial reinforcing bars	D32 × 182
	Hoop ties	D16 ctc 150
Pile foundation	Diameter and length of Pile	$\phi = 1.2m, 16.0m$
	Axial reinforcing bars	D25 × 20
	Hoop ties	D22 ctc 125

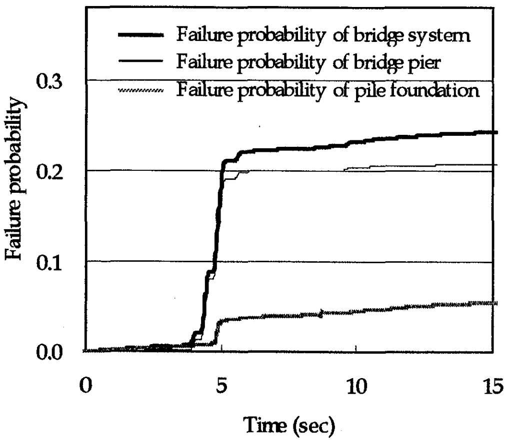

Figure 16. Failure Probability of Bridge System and its Components When Equipped with Seismic Isolation Bearings

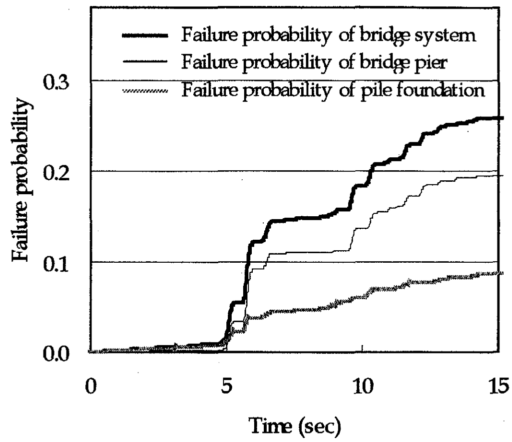

Figure 17. Failure Probability of Bridge System and its Components when Equipped with elastomeric Bearings

stiffness was set as shown in Table 6. The non-isolation bearings used were elastomeric bearings. These elastomeric bearings were modeled by a linear spring, and their stiffness was set at 23.60 MN/m. The lead rubber bearings reduce the inertial force of the superstructure through hysteresis damping more than elastomeric bearings.

6.2 Effects on the Safety of Bridge System of Seismic Isolation System

The maximum acceleration of the Hyogo-ken Nanbu Earthquake seismic waveform was adjusted to 500 gal and input into the analytical ground to evaluate the safety of the bridge systems with seismic isolation systems and non-isolation systems. First, Figure 16 shows the changes over time in the failure probability of a bridge system and its components when equipped with seismic isolation. In this case, the safety of the bridge system is governed by the safety of the pier, and the failure probability rises rapidly around the time of maximum response. Thereafter, it does not rise very much.

Next, Figure 17 shows changes over time of the failure probability of a bridge system and its components when equipped with a non-isolation system (or elastomeric bearing). In this case also, the safety of the bridge system is governed by the safety of the bridge pier, as in the seismic isolation system case. However, the failure probability continues to rise after the time of maximum response (which occurs about 5.5 seconds after the earthquake onset). Ultimately, the failure probability is equal to that of the seismic isolation case. Looking at the pile foundation, the failure probability in the non-isolation system case is greater that that in the seismic isolation case.

The results in Figure 17 are compared with Figure 8. The difference between the bridge systems in the two figures are the seismic isolation system assumed in obtaining Figure 8 and the smaller quantity of longitudinal steel reinforcement. In Figure 17, the failure probability of the pile foundation is lower while the failure probability of the bridge pier is higher. This is a consequence of the elastomeric bearing system, which reduces damage to the pile foundation and but leads to increased pier damage because of the greater quantity of longitudinal steel reinforcement in the pile, giving it higher bending strength. In Figure 17, seismic energy is not absorbed by the bearings and damage to the bridge pier and pile foundation gradually rises as time passes as a result of using an elastomeric bearing system. As a result, the failure probability of the bridge system gradually rises until it surpasses that of the bridge system equipped with a seismic isolation system. It can be concluded that, in order to increase the safety of a bridge system in this way, it is necessary to also consider the way its components interact.

7. APPLICATION TO SEISMIC DESIGN

The present Concrete Standard Specifications [20] require that structures be designed to meet certain seismic resistance requirements under a hypothetical external force. However, seismic resistance is not stipulated quantitatively, and it is difficult at the design stage to predict the level of damage a structure will suffer. So, focusing on meeting the requirements of Seismic Performance 1, the proposed method was used to quantitatively clarify the limit point at which a structure satisfies a certain seismic performance stipulation.

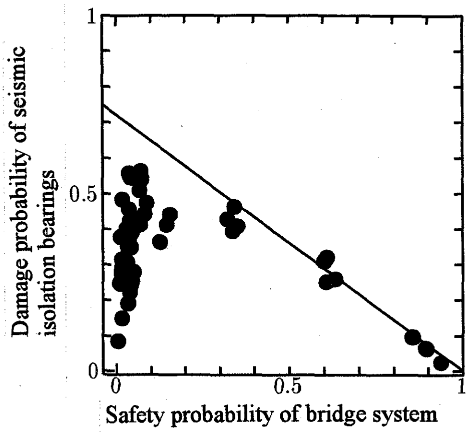

Figure 18. Bridge System Safety Probability versus Seismic Isolation Bearing Damage Probability Relationship

For a bridge system, when Seismic Performance 1 is no longer satisfied, the system is considered to have entered the regime where a component that absorbs the hypothetical earthquake energy begins to suffer damage. For example, in the case of a bridge with a seismic isolation system, this is when the seismic isolation system begins to exhibit non-linear properties. If a steel bearing is used, non-linear behavior of the bridge piers begins at this point.

This study has already shown (in section 4) that damage occurs primarily in the bearings of a bridge system equipped with a seismic isolation system, and that secondary damage occurs to the pile foundation. Noting that bearing damage may be translated into damage or failure of other parts in this way, we define the boundary of Seismic Performance 1 as the point at which non-linear behavior of the seismic isolation bearing begins. The proposed method is thus able to quantitatively verify required performance by indicating where the boundary between Seismic Performance 1 and 2 occurs.

The proposed method was used to calculate the safety probability and the probability of seismic isolation system damage after an earthquake for combinations of bridge systems with varying pile diameters and quantities of longitudinal steel reinforcement and for four variations of Type II ground (with natural periods T_G of 0.33, 0.34, 0.43, and 0.51 seconds). The earthquake inputs were the Miyagi-ken Oki Earthquake motion and the Hyogo-ken Nanbu Earthquake motion (both adjusted for maximum accelerations ranging from 100 gal to 800 gal). The results are shown in Figure 18.

First, within the range of good safety probability, there is a linear relationship between the probability of damage to the seismic isolation bearing and the safety probability of the entire bridge system. This relationship is a consequence of seismic isolation system behavior; when the external force is small, its non-linear response reduces the potential for damage to other components. The loss of this linear relationship as the safety probability of the bridge system declines is a result of damage to other components beginning to occur as the external force increases. Thus, for bridge system safety probabilities of 0.2 and lower, the probability of damage to the seismic isolation bearing is scattered between 0 and 0.6. This is because damage to other components members is higher and the probability of 2-component damage or failure is higher than the probability of damage to the seismic isolation system alone (1-component damage).

Looking at linear relationship between safety probability of the bridge system and the probability of damage to the seismic isolation system, if the regression line has a gradient of 0.5 or more, the following equation can be written:

$$\text{(probability of damage to seismic isolation bearing)} = 0.72 - 0.72\times \text{ (safety probability)} \qquad (18)$$

This tells us that the point at which the probability of damage to the seismic isolation bearing exceeds the safety probability of the bridge corresponds to a bridge safety probability of 0.42. It also makes it possible to categorize a bridge system into one of three states by using the proposed method to calculate the reliability of the bridge system under hypothetical external forces and comparing the results with safety probabilities.

Thus, the proposed method can be used to quantify the boundaries between Seismic Performance levels by accounting simultaneously for the ultimate limit state and the damage limit state. During seismic design, it makes it possible to predict the seismic resistance of a bridge system by comparing the calculated boundary values with the safety probability of the bridge system.

8. CONCLUSIONS

The following conclusions have been reached in this study:

1) A method of incrementally calculating the failure probability of bridge systems and their constituent parts and of incrementally calculating the damage probability of constituent parts has been proposed. This method can be used to quantitatively evaluate changes over time in the safety and damage state of a bridge system.

2) The proposed method was used to evaluate changes in the safety of a bridge system during an earthquake. It was found to be possible to detail the changes in the bridge components and in the individual limit states that affect the safety of the bridge system as a whole.

3) The safety and damage state of a bridge system in the case earthquake motion directly below the bridge and in the case of plate-boundary seismic motion were evaluated. This led to clarification of the difference in damage to a bridge system according to the type of seismic motion.

4) The bending strength of the pile foundation for a pier was varied to evaluate the safety and state of damage to bridge systems with pile foundations having various yield seismic coefficients.

5) The safety of bridge systems with and without seismic isolation systems was evaluated. The revealed that secondary damage to a bridge system fitted with a seismic isolation system is primarily plasticization of the base of the piers.

6) The boundary between levels of Seismic Performance 1 and 2 was quantitatively identified by looking at the safety probability of the bridge system.

Since the Hyogo-ken Nanbu Earthquake, it has been necessary to guarantee not only the seismic resistance of each component and structural member, but also the ability of entire bridge systems to withstand earthquakes. The method proposed in this study is applicable to design methods structured to achieve this goal. It is able to quantitatively clarify both the safety level of all components and structural members of a bridge system, while also indicating the safety of the entire bridge system as these change over time. It can also account for the design (stochastic) variables that affect the seismic resistance of the system. However, although the importance of applying reliability theory to seismic design has been well noted, in a practical sense it has not reached the level of development that allows it to be reflected in design standards. This is mainly because of insufficient statistical data concerning various uncertainties interposed within structures. Databases of various stochastic variables will need to be completed in order to resolve this problem. Still, we are now at the stage where it will be possible to perform adequate reliability analysis once parameters for stochastic variables are established based on existing measurement data such as that obtained by this study [21]. This will clarify the uncertainties taken into account here, to a greater degree than was possible with past design methods based on safety factors and similar that were set according to vague understanding. We look forward to using a common criterion that we have called "failure probability" to construct a design system able to stipulate the seismic resistance a structure will provide following an earthquake.

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