Seismic Performance Evaluations of Bridge-Pier System under Aging Structural Properties

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SUMMARY
Deterioration of Japan's social infrastructure due to aging has progressed with time and is now a challenging issue. Establishment of a method for evaluating seismic performance accompanying damage and aging during the service life of structures is extremely important for proper maintenance. In the present research, the seismic safety of the structure with 3 degrees of freedom was carried out under aging situations in structural properties using the cumulative probability distribution of the damage index and the reliability index for each year obtained by Monte Carlo Simulation (MCS), treating the maximum acceleration of the input seismic motion as an uncertain parameter. It is shown that damage index and reliability index can play considerably useful indexes for seismic evaluations considering aging effects in structural properties.

Keywords: Seismic Performance Evaluations, Deterioration, Damage Index, Reliability index

1. GENERAL INSTRUCTIONS

Deterioration of Japan's social infrastructure facilities due to aging has progressed with time and is now a challenging issue. Establishment of evaluation methods for seismic performance accompanying the deterioration and damage occurring over the lifetime of structures is extremely essential for appropriate maintenance control of those structures. In particular, when structural design is performed on performance based design, it is necessary to clarify nonlinear response characteristics when the structure concerned is subjected to strong seismic motions exceeding the yield level of that structure (Iemura et al(1998) , Kimura at al(2007)).

In the present study, a nonlinear response analysis under aging situations in the properties of structural materials was, therefore, carried out using an idealized vibration model of a nonlinear bridge pier system with 3 degrees of freedom, and a damage evaluation of the structure was performed using the damage index (Park and Ang(1985)). The seismic safety of the structure under aging in structural properties was also studied using the cumulative probability distribution of the damage index and reliability index for each elapsed year obtained by Monte Carlo Simulation (MCS), treating the maximum acceleration of input earthquake motion as an uncertain parameter (Guan et al(2000),(Kawano et al(2007))). It is suggested that damage index and reliability index can provide considerably useful indexes for seismic evaluations regarding aging effects in structural properties.

2. NONLINEAR SEISMIC RESPONSE ANALYSIS

2.1. Analysis conditions

(1) Analysis model

The analytical model of the structure of interest in this research is shown in Figure 1. The analytical model is an idealized model of a bridge, which is considered to comprise 3 elements (1, 2, 3) from the bottom, i.e., the bridge pier (1 and 2) and bridge girder (3), and mass and damping constant of each of the elements are 102 *KN/m/s2* and *2%*, respectively. Three variations of this structure were examined by changing the stiffness from the element 1 to element 3 so that the primary natural period T_1 of the structure was *0.5 sec, 0.8 sec, and 1.2 sec*.

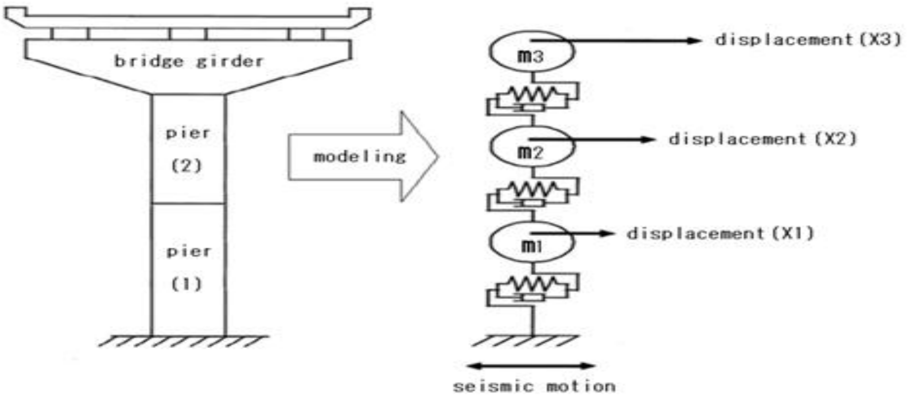


Figure 1. An idealized vibration model of bridge pier system

(2) Evaluation of nonlinearity of structure

The nonlinearity of the structure was evaluated using the bi-linear hysteresis model as shown in Figure 2. The model is set up so that all elements enter the nonlinear region at a maximum input acceleration of *200 gal* or more, and seismic response analyses are performed by assumption with the ratio of stiffness under linear and nonlinear conditions being *10 : 1*.

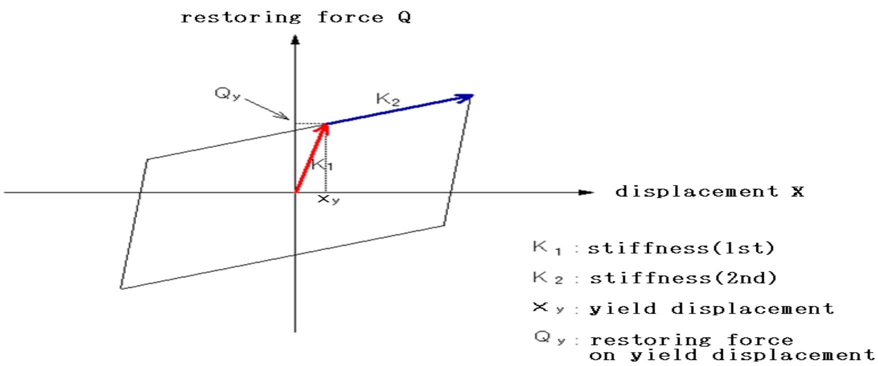


Figure 2. Bi-Linear hysteresis model

(3) Input seismic wave motion

Three components which were observed during the 1995 Kobe Earthquake were used in the present study, namely, Kobe-NS, observed on the ground of the Kobe Marine Observatory, Taka-NS, observed on the ground at JR West Japan Takatori Station, and Port-NS, observed on the ground at Port Island. Because these cause the different maximum acceleration due to the ground conditions, the analysis was performed with standardizing the maximum input acceleration of each motion in order to examine the dynamic characteristics of the seismic motions.

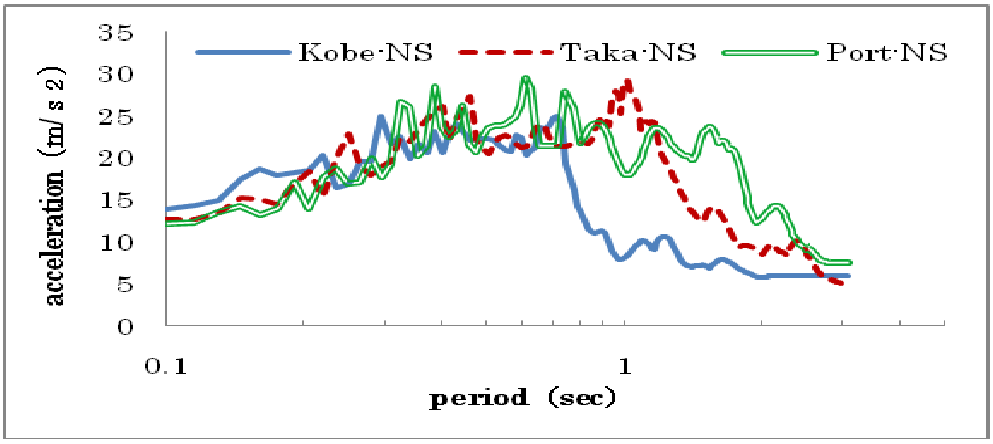


Figure 3. Acceleration response spectra of seismic motions

(4) Acceleration response spectra of seismic motions

Figure 3 shows the acceleration response spectra of the seismic motions. It is understood that response spectra have tendency rapidly decrease on the long period side from a threshold around *0.7 sec* with the Kobe-NS, 1.2 sec with the Taka-NS, and *1.8 sec* with the Port-NS component.

2.2. Results and discussions

(1) Time history response analysis

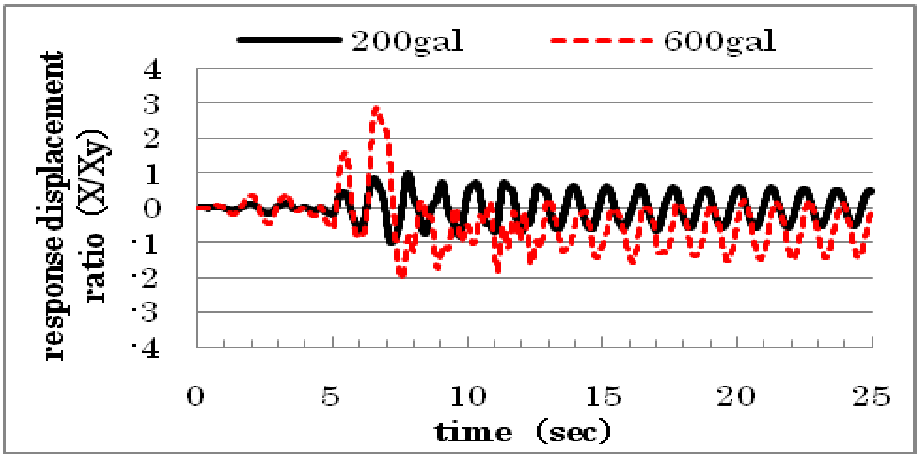
Figure 4.-(a) shows an example of the time history response displacement (X/X_y) of the bridge girder section with a primary natural period T_1 = *1.2 sec*, for the Port-NS component of maximum acceleration of *200 gal* and *600 gal*. For the case of maximum acceleration of *600 gal*, Figure 4.-(b) also shows the time history response displacement regarding the primary natural period of the structure, T_1= *0.5 sec* and *1.2 sec*.

The ratio X/X_y denotes a ductility factor which can be determined with dividing displacement X by yield displacement X_y. The following points can be understood from Figure 4. i) As the maximum input acceleration increases from *200 gal* to *600 gal*, the structure causes the nonlinear response , and as a result the ductility factor (X/X_y) increases and also it is figured out being nonlinear response characteristics, for example, the position of the zero point of the vibration component deviates. ii) The time history response component displays response which substantially coincides with the primary natural period of the respective structures. iii) The maximum value of displacement response and the attenuation are deeply depending on how the primary natural period of the structure is set. This is attributed to the relationship between the predominant period of the input seismic motion and the equivalent natural period of the structure due to nonlinearity.

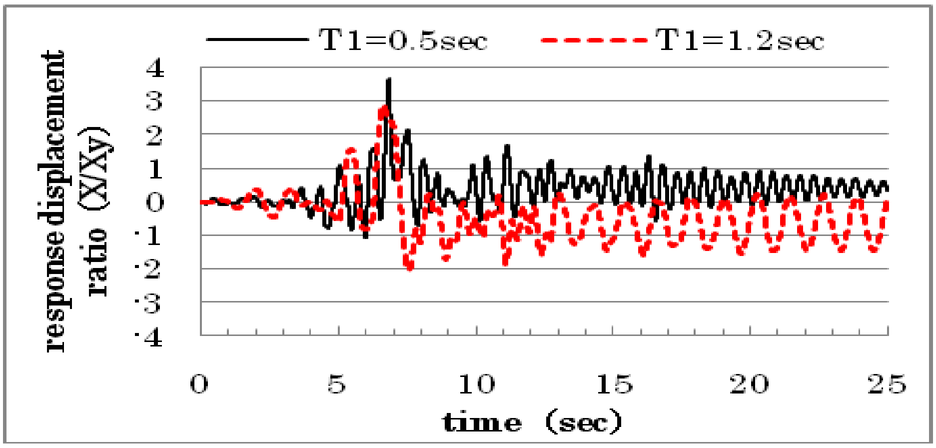
3. DAMAGE EVALUATION UNDER AGING IN MATERIAL PROPERTIES

3.1. Setting of aging of structural properties

In the present study, a nonlinear seismic response analysis is carried out in a condition which the duration of survice is assumed to be *50* years, and after *50* years, yield strength decreased *40%* in accordance with an exponential function.



(a) Influence of input acceleration
(T_1=1.2sec、maximum input acceleration 200gal and 600gal)



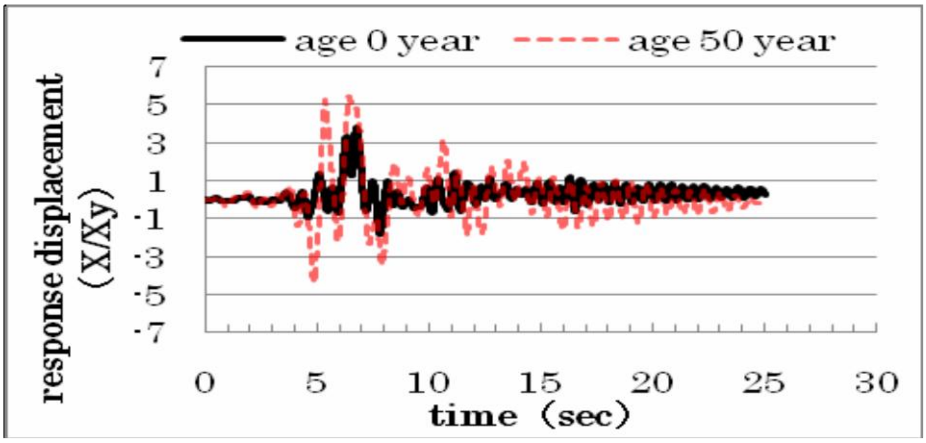
(b) Influence of primary natural period of the structure

(T_1=0.5sec、1.2sec、maximum input acceleration 600gal))

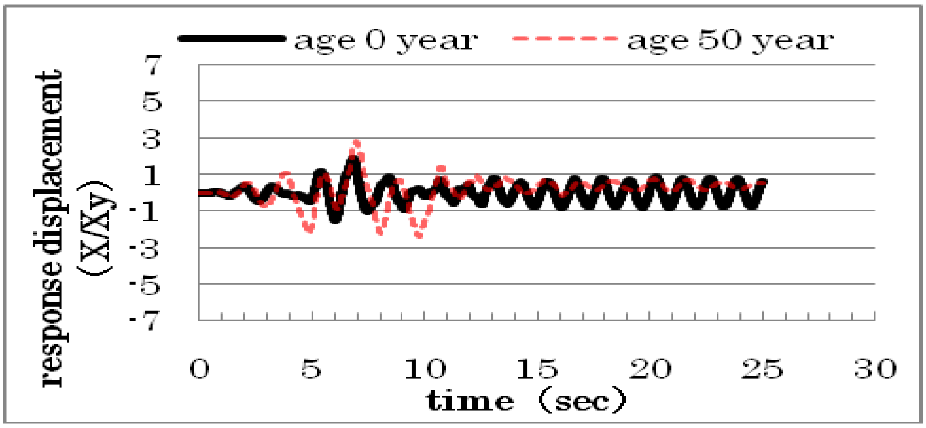
Figure 4. Time histories of displacement on girder (Port-NS)

3.2. Time history displacement response under aging in structural properties

It is examined about the time history of displacement response for the situation of aging effects due to seismic motions. For the response of the pier (element1) in case of a Port-NS component with a maximum input acceleration of *600 gal*, Figure 5.-(a) and Figure 5.-(b) show the time history responses of structures having primary natural periods of T_1 = 0.5 sec and T_1 = 1.2 sec, respectively. The following can be understood from Figure 5. , i) Because the apparent stiffness of the structure decreases over time, the response magnification factor of displacement, i.e., the ductility factor (X/X_y) increases, and the equivalent natural period of the structure becomes longer. ii) The change in the primary natural period of structures has considerably effect on the response displacement of the structure because the relationship with seismic motion characteristics such as dominant frequency and dynamic intensity. iii) The displacement response ratios of the structural elements are closely depended on the relationship between natural period of the structure and seismic wave characteristics such as the dominant frequency and dynamic intensity.



(a) T_1=0.5sec



(b) T_1=1.2sec

Figure 5. Time history response displacement under aging

(Port-NS, maximum input acceleration 600gal, pier(element1))

3.3. Park and Ang damage index *D*

Park and Ang proposed a method in which a damage index *D* is used as a reference standard forcollapse of RC structures affected by repeated loading, based on a statistical analysis of a large set of experimental results. In the present study, a nonlinear property is represented with the bi-linear hysteresis model as shown in Figure 2. Because the damage index *D* is defined as the linear combination on the ratio of maximum displacement and allowable displacement, and the hysteretic absorbed energy, *D* is given by the following equation (Park.and Ang(1985)):

$$D = \frac{x_{\max}}{x_u} + \frac{\beta_e}{Q_y x_u} \int dE \qquad (3.1)$$

where, $x_{\max}$ and x_u are maximum displacement and ultimate displacement, respectively, ∫dE is hysteretic absorbed energy, and Q_y is yield strength. β_e is a positive coefficient which depends on the cross-sectional properties of a member. In this research, a value of β_e = 0.15 is used, referring to previous research. Table 1 shows the relationship between Park's damage index and the degree of damage.

The seismic evaluations of structures are carried out with the target performance for seismic design of existing roadway bridges shown in Table 1. Seismic performance 1 is applicable to seismic motions corresponding to level 1, and seismic performance 2 and 3 are applicable to seismic motions level 2. Accordingly, in the present study, the range of $0.2<D<0.4$ corresponds to seismic performance 2, and $0.4<D<0.6$ corresponds to seismic performance 3.

Table 1 Relationship between Park's damage index and degree of damage

Usability	Degree of damage	Condition of damage	Park's damage index D
Use possible		No damage	0-0.1
	Light	Use possible	0.1-0.2
Use temporarily not possible	Light Medium Severe	Repair possible	0.2-0.4
		Repair impossible	0.4-1.0
Use impossible	Collapse	Collapse	>1.0

3.4. Damage evaluation under aging in structural properties

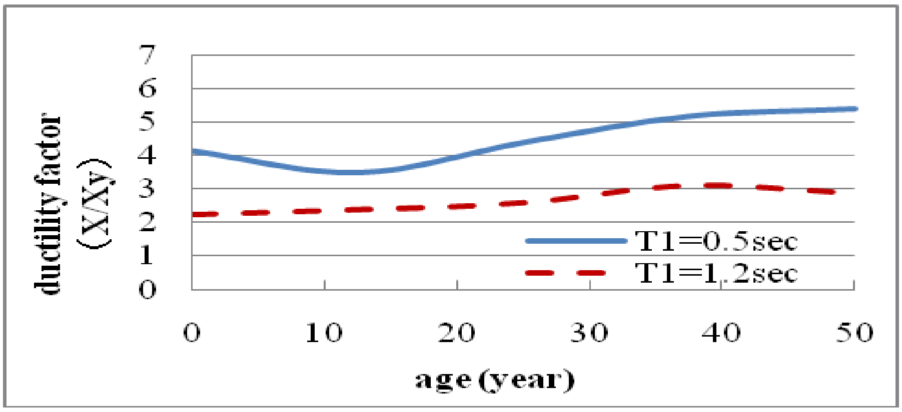
Damage evaluation under aging in structural properties is carried out using Park and Ang's damage index *D*, based on the results of a nonlinear seismic response analysis of a vibration system with 3 degrees of freedom. In this analysis, the damage index proposed by Park and Ang is obtained by calculating the maximum displacement response and hysteretic absorbed energy, assuming the ultimate displacement X_u is 7 times the yield displacement X_y of the elements of the structure. The damage evaluation at 0 years, 25 years, and 50 years of elapsed time is also examined by treating the maximum input acceleration of the input seismic motion as an uncertain quantity. Giving uncertain factors stochastically by a lognormal random number and applying MCS, it can evaluate effectively the cumulative probability distribution of the damage index *D*.

(1) Aging effects in ductility factor (X/X_y) and damage index D

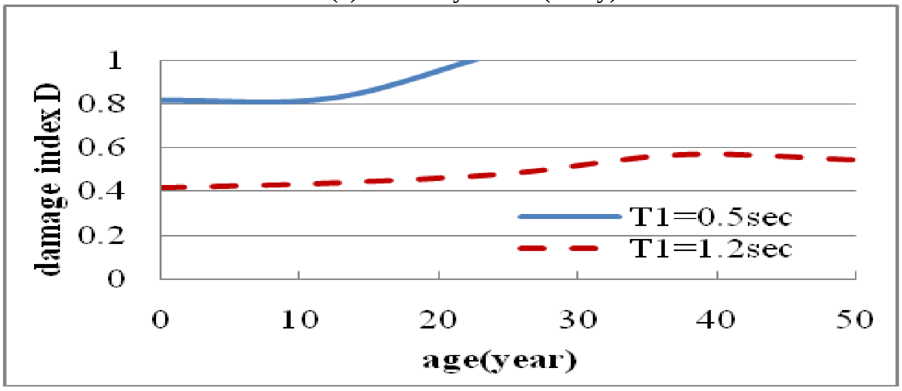
In the response of the bridge pier part (element 1) in case of a Port-NS component with a maximum input acceleration of *600 gal*, Figure 6.-(a) shows fluctuation in the ductility factor and Figure 6.-(b) shows the change in the damage index *D* due to elapsed time for structures having primary natural periods T_1 = *0.5 sec* and T_1= *1.2 sec*. The following can be understood from Figure 6. i) The ductility factor (X/X_y) of the elements tends to increase with increasing time. ii) As the ductility factor (X/X_y) increases, the damage index *D* also increases. iii) In case of T_1 = *0.5 sec*, it can be understood that the structure is in a situation of collapse or near-collapse if the position of the damage index *D* is *0.8* or more. iv) In case of T_1 = *1.2 sec*, the damage index *D* shows approximately between *0.4 to 0.6* and this condition falls under seismic performance 3.

(2) Cumulative probability distribution of damage index D

In the response of the bridge pier part (element 1) for Port-NS component with a maximum input acceleration of 600 gal, Figure 7.-(a) and Figure 7.-(b) show the cumulative probability distributions of the damage index *D* for structures with primary natural periods of T_1 = 0.5 sec and T_1 = 1.2sec,



(a) ductility factor (X/Xy)



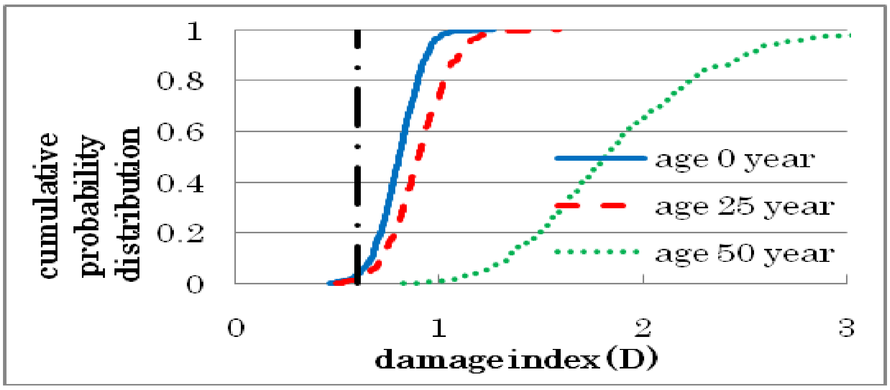
(b) damage index D

Figure 6. Aging effects in ductility factor *(X/*X_y*)* and damage index *D*

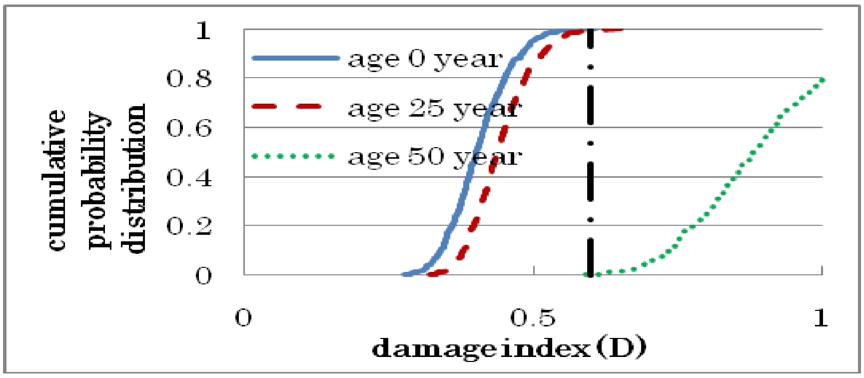
(Port-NS, maximum input acceleration *600gal*, pier(element1))

respectively. The following can be understood from Figure 7. i) In case of T_1 = 0.5 sec, the damage index *D* is already in a situation which cannot exceed seismic performance 1 to the first year. Evaluated from the viewpoint of collapse, the probability that the damage index *D* will exceed 1 in the first year is substantially 0%, and the probability that *D* will exceed 1 after 25 years is approximately 20%. After 50 years, the probability that *D* will be greater than 1 is approximately 100%. After 50 years, the structure is positioned in a collapse situation. ii) In case of T_1 = 1.2 sec, *D* is distributed in the range from 0.3 to 0.6 in the first year and after 25 years. The probability of exceeding 0.4 in the first year is approximately 50%, and the probability of exceeding 0.4 rises to approximately 80% after 25 years. Both of these conditions fail to satisfy seismic performance 2 and fall under seismic performance 3. Because *D* exceeds 0.6 after 50 years, the structure fails to satisfy seismic performance 3, and large damage classified as severe (Repair impossible) or worse can be predicted. iii) From the analysis results, it can be understood that the degree of damage increases with increasing time (years of service). iv) For a structure with a primary natural period of T_1 = 1.2 sec, it can be noted that the increase in the damage index *D* due to time is slight in comparison with the case of the T_1 = 0.5 sec.

This is attributed to the fact that, from Figure 3, dominant frequency of seismic motions exists around 0.5 sec rather than around 1.2 sec.



(a)T_1=0.5sec



(b) T_1=1.2sec

Figure 7. Cumulative probability distribution of damage index *D*
(Port-NS, maximum input acceleration 600gal, pier(element1))

4. RELIABILITY EVALUATIONS UNDER AGING OF MATERIAL PROPERTIES

4.1 Reliability index β

Dynamic response characteristics of structures and its strength properties subjected to random forces are generally expressed as stochastic quantities. The reliability index β is used as an indicator for safety evaluation corresponding to failure probability, and is given by Eq.(4.1) using MCS(Kawano et al(2007)).

$$\beta = \frac{\overline{M}}{\sigma_M} = \frac{\overline{R} - \overline{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{4.1}$$

Here, in case of evaluation by strength and load, the performance function *M*= *R*-*S* is expressed, in which strength and load in the failure mode are represented by *R* and *S*, respectively. *M* is the performance function of the failure mode, and $\overline{M}$ and σ_M are the mean and standard deviation, respectively. In general, the strength of materials is suggested with a variation on approximately order of 10%. Therefore, the standard deviations σ_R of $\overline{R}$ for strength and displacement is assumed to be 10%, and calculations are performed assuming $\sigma_R = 0.1\overline{R}$. For seismic response analysis, MCS is carried out with an random variable on maximum seismic accelerations with a coefficient of variation of 10%.

4.2. Reliability evaluation under aging in structural properties

Two cases were examined in connection with reliability evaluation. (1) Case 1 (reliability evaluation with respect to restoring force), in which 1.5 times the yield strength Q_y of a member is used in strength *R* and restoring force *Q* is used in load *S*, and (2) Case 2 (reliability evaluation with respect to displacement), in which 5 times the yield displacement X_y of the member is used in the design limit displacement $\overline{R}$, and the response displacement *S* of the structure is used.

(1) Case 1(reliability evaluation with respect to restoring force)

Figure 8. shows the reliability index for time-restoring force for structures having primary natural periods of $T_1 = 0.5\ sec$ and $T_1 = 1.2\ sec$ in the response of the pier part (element 1) in the case of a Port-NS component with a maximum input acceleration of *600 gal*. The following can be understood from Figure 8. i) In case of $T_1 = 0.5\ sec$, β becomes less than 1.0 to the 25 year and tends to decrease until the 50 year. As the failure probability at this time is $P_f = 1.0 \times 10^{-1}$ *(= 1/10)*, a seismic safety can be suggested extremely low. ii) In case of $T_1 = 1.2\ sec$, the reliability index β is around *2.5* to the first year and decreases with time; however, even at the 50 year, β keeps larger than *2.0*. Since the failure probability is 2.27×10^{-2} *(= 1/44*), this can be considered to be a situation that maintains seismic performance to a certain extent. iii) The above-mentioned results virtually correspond to the tendency of the damage index *D* in Figure 6.

(2) Case 2(reliability evaluation with respect to displacement)

For the reliability index for time-displacement for structures, the following results are obtained; that is, it can be understood that the reliability index for the performance function of displacement provides the different results from the reliability evaluation for restoring force, and this evaluation gives a safer result than the reliability index for restoring force. Since this type of reliability index shows different tendencies depending on the performance function, it is very essential to clarify the available performance function on the reliability evaluation in comparison with damage to actual structures.

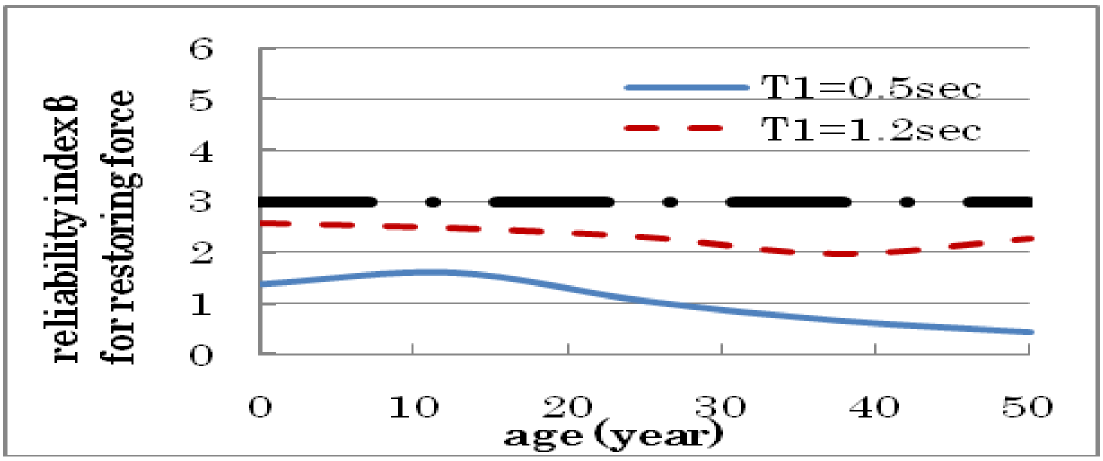


Figure 8. Reliability evaluation with respect to restoring force

(Port-NS, maximum input acceleration *600gal*、pier(element1))

5.CONCLUSIONS

The main results in the present study are summarized as follows.

(1) In the severe seismic situation, the apparent stiffness of the structure decreased due to duration and seismic input acceleration such as the time history response component of the structure displayed a longer period. This means that the equivalent natural period of the structure became longer, which is brought about considerable effects on structural response with related to seismic wave characteristics.

(2) As the maximum acceleration of the input seismic motion becomes larger, the damage index D increases, together with the ductility factor, and the extent of the increase is greatly affected by the natural period of the structure, dynamic characteristics of the elements, and dynamic characteristics of the input seismic motion.

(3) In evaluations by the damage index D, an understanding of uncertain parameters is important roles in damage evaluation, which can be estimated with the cumulative probability distribution of D using MCS, which is closely related with structure element characteristics on nonlinear responses.

(4) In seismic evaluations of structures using the damage index D, it plays important roles on seismic performance evaluation for random properties of input seismic motions and structural properties.

(5) The allowable situation of damage in structures where nonlinear response is permissible is a substantial issue. Taking into account the uncertainty of the maximum acceleration of the seismic motion and the aging in the structural properties, it is possible to evaluate the performance by the reliability index β with respect to the design values of the structure.

(6) For performing seismic evaluations of structures which are subject to nonlinear response and under aging, reliability index β and damage index D play important roles on the structure performance evaluation. However, since their correspondence is not necessarily clarified in all cases, elaborated seismic performance evaluation could be provided by using both approaches in the future.

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