Performance Evaluation of a Base-Isolated Bridge with Aged Rubber Bearings

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Abstract
It is a well-known fact that aging causes shear stiffness of rubber to increase. Rubber bearings commonly used as base-isolators also tend to increase their stiffness over time. To accurately evaluate future seismic performance of a base-isolated bridge, it is, therefore, important to consider the aging effect of rubber bearings. This paper presents a case study to examine the effect of aging of rubber bearings on the seismic response of a base-isolated six-span continuous steel bridge with lead rubber bearings (LRB). The long-term performance of rubber bearings is estimated based on the size of bearing, an average temperature of the construction site, and various aging times up to 100 years. Seismic performance of the bridge is evaluated through dynamic analysis of a three-dimensional finite element bridge model. The results show that the aging of rubber bearings causes an increase in the seismic force on the bridge piers, resulting in larger damage in the piers.
Keywords: rubber bearing, aging, base isolation, seismic performance, steel bridge

1. INTRODUCTION

Significant damage that many bridges sustained during the Hyogo-ken Nanbu (Kobe) earthquake on January 17, 1995 attracted considerable attention among researchers who realized that the strength alone would not be sufficient to protect bridges from severe earthquakes. Since the beginnings of seismic engineering, it has been recognized that a structure with a fundamental natural frequency away from the predominant frequency content of the seismic input has a smaller response and less damage. Therefore, seismic isolation, artificially increasing both the natural period of vibration and the energy dissipation capacity of structure, has been regarded as an attractive way to improve seismic performance of a structure.

Rubber has a long history of applications in bridge bearings due to its special properties such as high elasticity and large elongation at failure. For example, natural rubber (NR) bearings were first used in a bridge in 1889 [1]. Nowadays, laminated rubber bearings have been widely adopted as isolation bearings in bridges. A lead rubber bearing (LRB) where lead plugs are inserted to a laminated natural rubber bearing is currently one of the most popular isolation bearings because it can provide both horizontal flexibility and large energy dissipation capability. Lead has low yield strength, sufficient high initial shear stiffness, essentially elastic-plastic behavior and good fatigue properties.

Rubbers can be damaged by oxygen, ozone, heat, light, dynamic strain, oil, and other liquids. Physical properties of rubbers may change as a result of the degradation process over time, called aging. Aging causes rubber to stiffen and its tensile strength and elongation at break to decrease. For example, Itoh et al. [2] reported the aging characteristics of rubber materials due to various deterioration factors based on the experimental study using accelerated exposure tests.

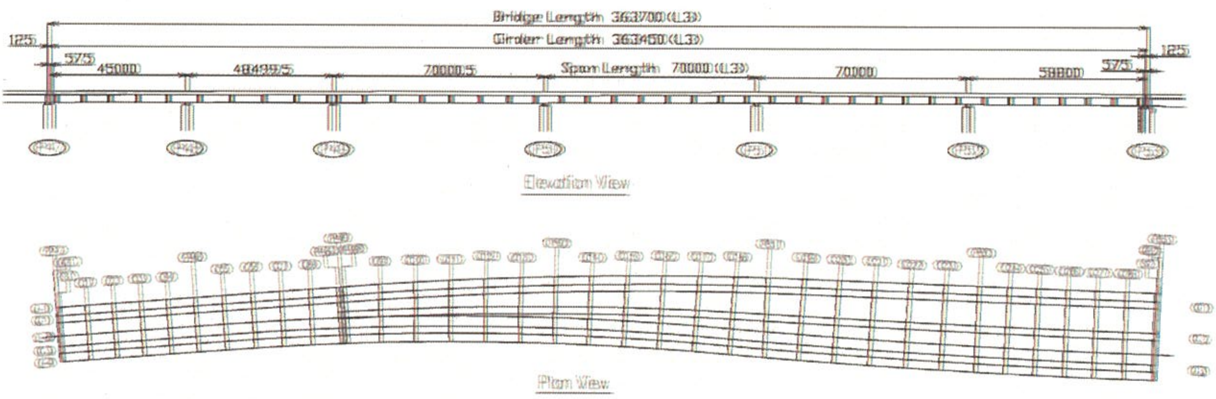


Fig. 1 General drawing of the bridge

As aging causes shear stiffness of rubber material to increase, rubber bearings also tend to increase their stiffness over time. In the current design specifications, the stiffness increase of rubber bearings due to aging is not considered. To examine the effect of aging of rubber bearings on the seismic response of a base-isolated bridge, a case study was conducted. A base-isolated six-span continuous steel box girder bridge (Fig. 1) with LRBs owned by Nagoya Expressway Public Corporation was examined through dynamic analysis. The long-term performance of LRBs was first evaluated for this particular bridge, and LRB stiffness was calculated for 20, 40, 60, 80, and 100-year aging times. To examine the aging effect of rubber on the seismic performance, the peak responses from the dynamic analysis were compared with those for the initial state.

The finite element (FE) model of the bridge is shown in Fig. 2. The bridge has T-type steel piers, two main girders between Pier 47 and Pier 49, and three main girders between Pier 49 and Pier 53. The pier heights range from 15.4 to 15.8 m. Piers have stiffened steel box sections, and are filled with concrete at the bottom. Dimensions of the bearings and piers are shown in Table 1.

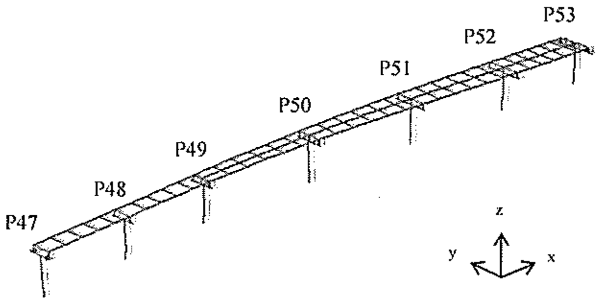


Fig. 2 FE model of the bridge

Table 1 Dimensions of bearings and piers

Pier Number	Bearing				Pier				
	Length (m)	Width (m)	Total Rubber Thickness (m)	Static Shear Modulus of Rubber (kN/m^2)	Height (m)	Width of cross section (m)	Height of cross section (m)	Web thickness (m)	Flange thickness (m)
P47	0.66	0.66	0.165	800	15.5	2.8	2.5	0.027	0.030
P48	0.90	0.90	0.198	1,200	15.7	2.5	2.5	0.031	0.031
P49	0.90	0.90	0.189	1,000	15.7	2.5	2.5	0.041	0.041
P50	1.00	1.00	0.117	1,200	15.8	2.6	3.0	0.027	0.030
P51	1.05	1.05	0.144	1,200	15.8	2.6	3.0	0.042	0.042
P52	1.10	1.10	0.144	1,200	15.4	2.6	3.0	0.043	0.043
P53	0.92	0.92	0.187	1,200	15.5	2.8	3.0	0.047	0.041

2. LONG-TERM PERFORMANCE OF LRB

In the previous studies, Itoh [3, 4] examined the time-dependent and temperature-dependent aging characteristics of NR bearings, and developed simple formulas to calculate a change in the equivalent shear stiffness of the NR bearing. Generally, the shear stiffness increases with aging time and temperature. Based on the bearing size, aging time, and average yearly temperature at the construction site, the horizontal shear stiffness of an NR bearing can be calculated by Eq. (1), where G_0 is the static shear modulus of NR before aging, t_R is thickness of rubber layer, n is the number of rubber layers, and variables, B, H, and d^* are shown in Fig. 3. d^* is the critical depth, which can be estimated from Eq. (2), where the symbols α and β are constants determined by the thermal oxidation test. From the accelerated thermal oxidation tests, it was found that material properties would change at the most at the surface, while those in the interior region would remain virtually unchanged. The critical depth is a depth beyond which oxidation cannot reach. Δf_s is the amount of change in the static shear modulus of rubber due to aging. Itoh et al. [3] identified that α and β as 0.0008 mm and $3.31\text{x}10^3$ K, respectively, and proposed Eq. (3) for estimating Δf_s. The temperature used in an accelerated thermal oxidation test is typically much higher than that in the real environment. This is because the high temperature can accelerate the deterioration. The Arrhenius methodology is commonly used to correlate the accelerated aging results with the aging under service conditions formulas shown in Eq. (4). This formula converts an aging time t at any temperature T to an equivalent aging time t_{ref} at a reference temperature T_{ref}. The parameters used in this study to estimate the variation of the equivalent shear stiffness are listed in Table 2.

$$K_h = G_0\left[(B-2d^*)(H-2d^*)+2d^*(B+H-4d^*)(1+\Delta f_s/3)+4d^{*2}(1+\Delta f_s/2)\right]/nt_R \qquad (1)$$

$$d^* = \alpha exp\left(\frac{\beta}{T}\right) \qquad (2)$$

$$\Delta f_s = 0.053\, t_{ref}^{\ 0.534} \qquad (3)$$

$$ln\left(\frac{t_{ref}}{t}\right) = \frac{E_a}{R}\left(\frac{1}{T_{ref}} - \frac{1}{T}\right) \qquad (4)$$

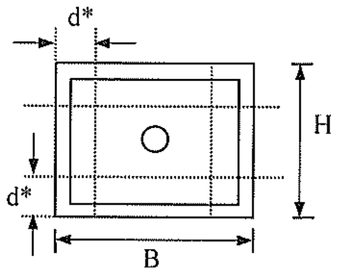


Fig. 3 Plan view of rubber bearing

Table 2 Parameters to estimate the variation of bearing equivalent shear stiffness

Temperature at the site	T	288.4 K (15.4°C, average temperature in Nagoya)
Activation energy of NR rubber	E_a	9.49×10^4 J/mol
Gaseous constant	R	8.314 J/mol·K
Reference temperature	T_{ref}	333 K (60°C)
Actual aging time	t	20, 40, 60, 80, 100 years

The equivalent shear stiffness of an LRB is calculated by the following equation:

$$K_{Be} = \frac{F}{u_{Be}} = \frac{G_e A_e \gamma_e + A_p q(\gamma_e)}{u_{Be}} \qquad (5)$$

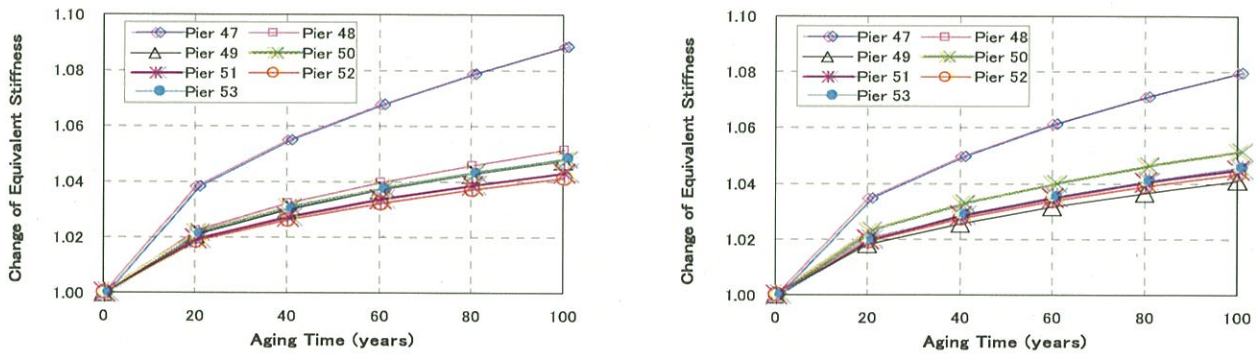
where K_{Be} is the equivalent shear stiffness of the LRB, and F is the shear force corresponding to the effective shear strain γ_e, which is 70% of the maximum expected shear strain. u_{Be} is the effective shear displacement, G_e is the static shear modulus of NR, A_e is the area of LRB excluding the surface rubber and the area of the lead plugs, A_p is the area of lead plugs, and $q(\gamma_e)$ is the shear stress of the lead plug at γ_e. Since G_e of NR increases due to aging, K_{Be} will increase consequently. A relative change of the equivalent shear stiffness to the initial stiffness can be calculated by Eq. (6).

$$\frac{K_{Be}}{K_{Be0}} = \frac{K_h / K_{h0} + C}{1+C} \qquad (6)$$

where

$$C = \frac{A_p q(\gamma_e)}{G_e A_e \gamma_e} \qquad (7)$$

By using Eq. (6), the equivalent shear stiffness of LRBs shown in Table 1 was calculated for an average temperature of Nagoya City (15.4°C) and various aging times up to 100 years, and variations are shown in Fig. 4 (a) and (b) for the longitudinal and transverse directions, respectively. In the figure, a ratio of the stiffness at a certain aging time to the initial stiffness, K_{Be}/K_{Be0}, is plotted for each pier. As can be seen from Figs. 4(a) and 4(b), the maximum stiffness change is about 9% and 8% for the longitudinal and transverse directions, respectively. Bearings on Pier 47 have the largest change because the bearings on Pier 47 have the smallest size. Itoh et al. [3] showed that the smaller the bearing size, the higher the ratio, K_{Be}/K_{Be0}, is. As for a NR bearing, material properties of rubber in the interior region beyond the critical depth are not influenced by aging. Since the critical depth depends on the environmental temperature as shown in Eq. (2), the critical depth will be the same regardless of the bearing size. Therefore, the smaller the bearing size, the larger a ratio of the region affected by aging to the entire bearing is. The increase in the bearing equivalent shear stiffness results in a decrease in the global natural period of the bridge. In this case, the decrease in the global natural period turned out to be about 4% after 100 years.



(a) Longitudinal direction

(b) Transverse direction

Fig. 4 Change of the bearing equivalent shear stiffness due to aging

3. SEISMIC RESPONSE OF SIX-SPAN BASE-ISOLATED CONTINUOUS STEEL BRIDGE DUE TO AGING OF RUBBER BEARINGS

Dynamic analysis of the bridge was performed by using the general purpose finite element analysis program, ABAQUS. LRBs were modeled as truss elements with a bilinear force-displacement relationship as shown in Fig. 5, where K_{Be}=equivalent stiffness, $H_{d,B}$=yield shear force, $K_{1,B}$=primary stiffness, $K_{2,B}$=secondary stiffness, U_B= design displacement, and U_{Be}=effective design displacement of LRB bearings. Mass elements were used to account for mass of girders, transverse beams, footings, and the influence of dead loads from the adjacent bridges on Pier 47 and Pier 53. However, seismic force from the adjacent bridges was not considered. Girders, transverse beams, and piers were modeled with beam elements. Three earthquake records specified in Design Specifications of Highway Bridges [5] for Level 2 Type II and soil type III were used in the dynamic analysis, and they are shown in Fig. 6. Figs. 7(a) and 7(b) present the stress-strain curves of steel and concrete, respectively. Material properties of steel and concrete used in the analysis are shown in Table 3.

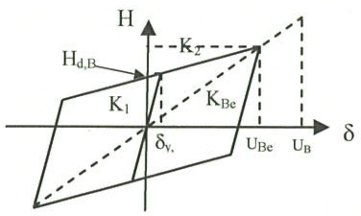
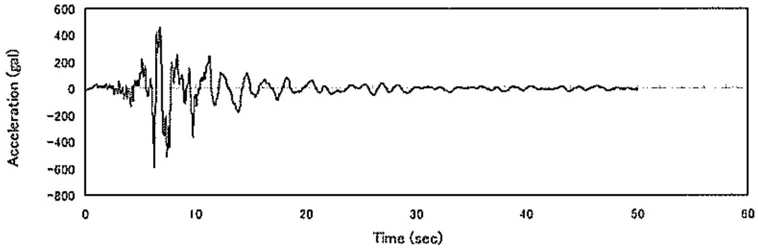
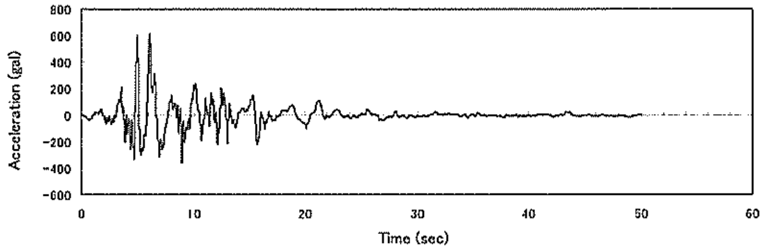


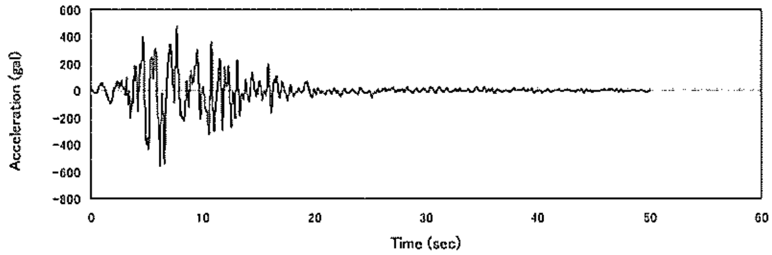
Fig. 5 Bilinear force-displacement relationship of LRB



(a) East Kobe Bridge N12W

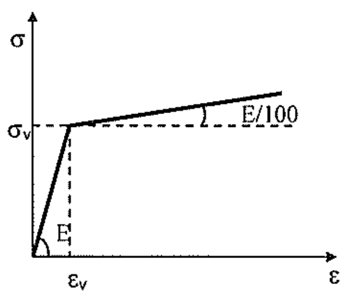


(b) Port Island EW

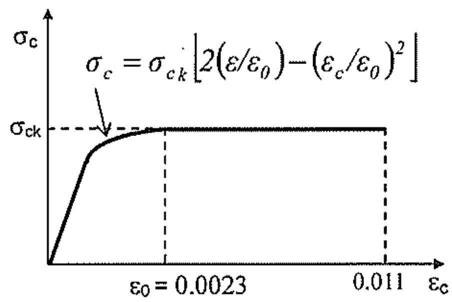


(c) Port Island NS

Fig. 6 Level 2 Type II Earthquake records used in the analysis



(a) Steel



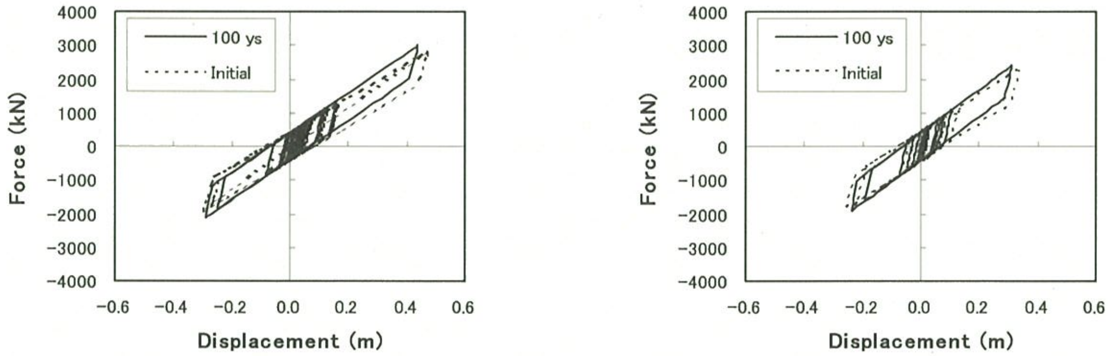
(b) Concrete in compression

Fig. 7 Stress-strain curves for steel and concrete

Table 3 Material properties of concrete and steel

Concrete	Design Strength	16 MPa
	Young's Modulus	22 GPa
	Shear's Modulus	9.6 GPa
Steel	Yield Stress	235 MPa (SM400) 335 or 355 MPa (SM490Y)
	Young's Modulus	200 GPa
	Shear's Modulus	77 GPa

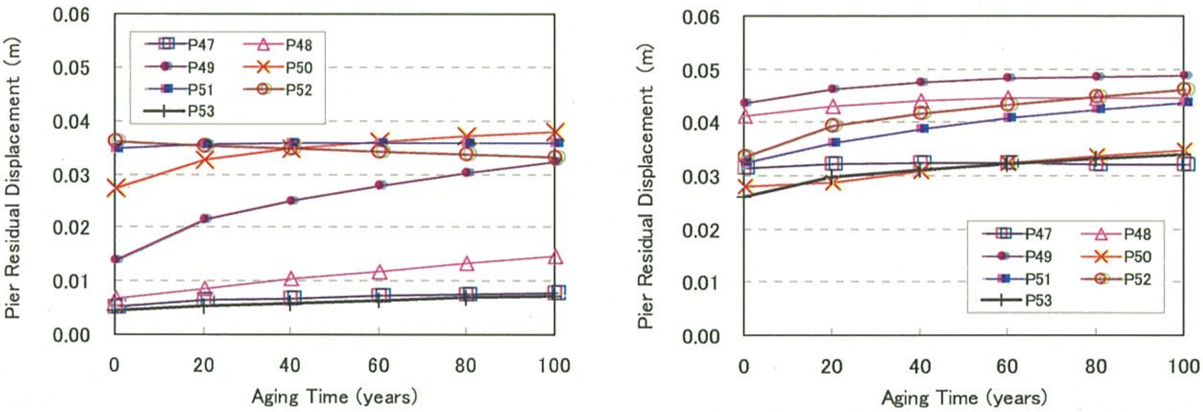
Results from the dynamic analysis of the bridge are shown in Fig. 8 through Fig. 13. Since the analyses were performed for the three earthquakes shown in Fig. 6, the responses presented in this paper are the average of the three analyses. Fig. 8 shows hysteretic curves obtained at the bearings on Pier 48 for the initial state (the condition of new bridge) and the state after 100-year aging time. It can be seen from the figure that the aging of a rubber bearing results in an increase in the bearing shear stiffness and in the horizontal force. Since the increase of the bearing equivalent shear stiffness causes the global natural period to decrease, acceleration responses of the base-isolated bridge increase, and, consequently, seismic forces on the piers increase. At Pier 48, for the longitudinal direction, the maximum force increased by 6% after 100 years while the maximum displacement decreased by 7%. For the transverse direction, the maximum force increased by 5%, and the maximum displacement decreased by 7%.



(a) Longitudinal direction (b) Transverse direction

Fig. 8 Hysteretic curve of bearing on Pier 48

In this paper, the seismic performance of concrete-filled steel piers is discussed in terms of residual displacements and damage indices. Fig. 9 shows residual displacements of each pier for different aging times. The residual displacements in the figure were obtained at the elevation where the seismic force acted on. Therefore, they were obtained at the top of pier and the girder elevation for the longitudinal and transverse directions, respectively. As can be seen in Fig. 9, the pier residual displacements tend to increase with the aging time. After 100 years, the pier residual displacements increased by 150% at Pier 49 for the longitudinal direction and by 40% at Pier 52 for the transverse direction. An allowable residual displacement specified in the current specifications [5] is $h/100$, where h=distance from the bottom of the pier to the location where the seismic force acts on. At the aging time of 100 years, the maximum residual displacement for the longitudinal direction is 0.038 m at Pier 50, which is 24% of the allowable value, while it is 0.049 m at Pier 49, which is 28% of the allowable value for the transverse direction. All piers were found to satisfy the performance requirement for the residual displacement.



(a) Longitudinal direction (b) Transverse direction

Fig. 9 Residual displacement of piers

Local buckling examinations are also required in the seismic performance evaluation of steel piers to ensure the pier shows ductile behavior during a severe earthquake. To determine capacity of a concrete-filled steel column, a failure criterion proposed by Usami [6] and Zheng [7] was used in this study. A damage index for a concrete-filled steel column is defined as a ratio of the average strain over an effective failure length to the failure strain as in Eq. (8).

$$D_s = \frac{\varepsilon_a}{\varepsilon_u} \tag{8}$$

When D_s reaches 1.0, the ultimate state is considered to be attained. Here, ε_a represents the average strain of the compressive flange (for the box section) over an effective failure length that will be defined below. The value ε_u denotes the failure strain and, to define it, the empirical formulas are employed.

For the damage index for a steel section, $\varepsilon_{a,s}$ is a compressive strain on the steel flange over effective failure length, where the effective failure length is defined as the smaller value of $0.7b$ and a. a is diaphragm interval, while b is flange width. The critical strain for the compressive flange, $\varepsilon_{u,s}$ is defined in Eq. (9).

$$\frac{\varepsilon_{u,s}}{\varepsilon_y} = \frac{0.8(1-N/N_y)^{0.94}}{\left(R_R \overline{\lambda_s}^{-0.18} - 0.168\right)^{1.25}} + 2.78\left(1-\frac{N}{N_y}\right)^{0.68} \leq 20 \tag{9}$$

where

$$\overline{\lambda_s} = \frac{1}{\sqrt{Q}}\frac{a}{r_s}\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}} \tag{10}$$

$$Q = \frac{1}{2R_R}\left(\beta - \sqrt{\beta^2 - 4R_R}\right) \leq 1.0 \tag{11}$$

$$\beta = 1.33R_R + 0.868 \tag{12}$$

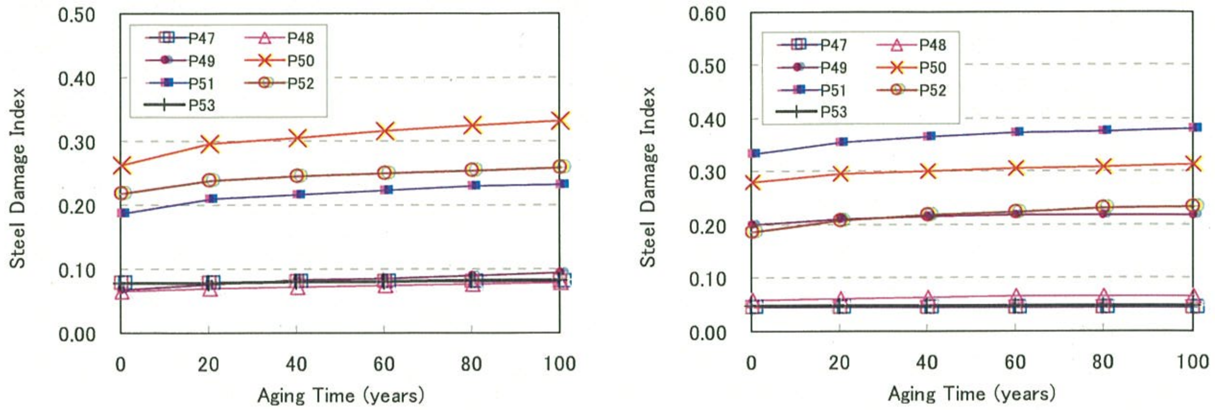
$$R_R = \frac{b}{t}\sqrt{\frac{\sigma_y}{E}\frac{12(1-\upsilon^2)}{\pi^2 4n^2}} \tag{13}$$

where r_s= radius gyration of T section comprised of one stiffener and flange panel (width = b/n) about the axis parallel to the flange plate, n= number of sub-panels, R_R= width-to-thickness parameter of stiffened flange, σ_y= yield stress of flange, ε_y= yield stress of flange, and N/N_y = ratio of axial load to yield strength.

Fig. 10 and Fig. 11 show the value of the damage index for the stiffened steel box sections of each pier and its change due to aging, respectively. The damage index of the pier steel section tends to increase over time because the seismic force increases due to aging. The maximum change of the damage index is about 40% at Pier 49 for the longitudinal direction, and 27% at Pier 52 for the transverse direction. However, all piers still have the damage index smaller than 1.0 after 100 years. The maximum steel damage index is 0.33 at Pier 50 for the longitudinal direction, and 0.38 at Pier 51 for the transverse direction, which implies that the piers will have much redundancy against Level 2 earthquakes even when the aging effect of 100-year old rubber bearings is considered.

For the damage index of a concrete-filled section, concrete strain is used to evaluate the section capacity. $\varepsilon_{a,c}$ is the average compressive strain at the extreme fiber of concrete section over the effective failure length. The effective failure length is the smaller of $0.7b(d/b)^{2/3}$ and h_c, where b and d are width and length of concrete section, respectively, and h_c= height of concrete filled section. The failure strain of concrete, $\varepsilon_{u,c}$ is 0.011.

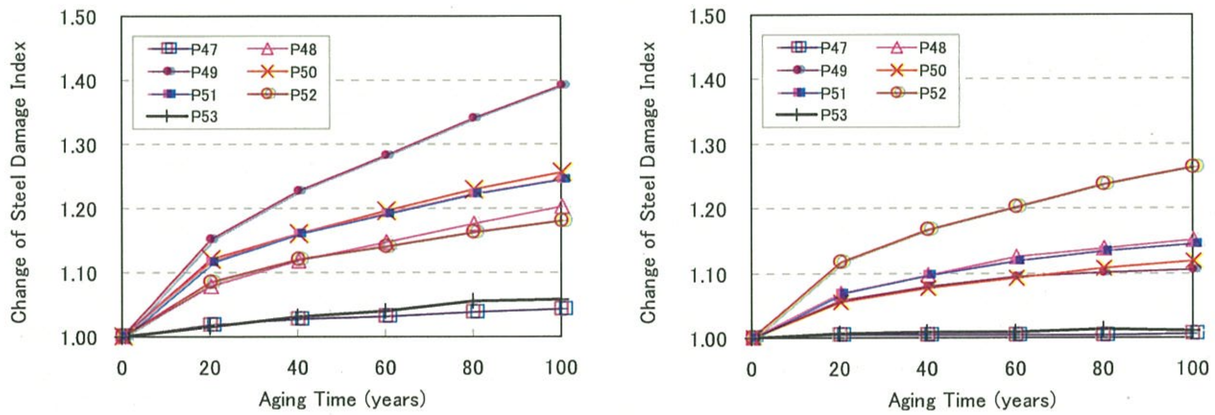
Fig. 12 and Fig. 13 show the damage index of the pier for the concrete-filled section and its change due to aging, respectively, As the damage index for the steel section, the damage index of the concrete-filled section also tends to increase over time because the seismic force increases due to aging. The maximum change is about 45% at Pier 49 for the longitudinal direction, and 18% at Pier 52 for the transverse direction. It was found that after 100 years the damage index of the concrete-filled section is still less than 1.0. The maximum concrete damage index is 0.024 at Pier 52 for the longitudinal direction, and 0.026 at Pier 50 for the transverse direction. These results also show that the piers will have much redundancy against Level 2 earthquakes even after 100 years.



(a) Longitudinal direction

(b) Transverse direction

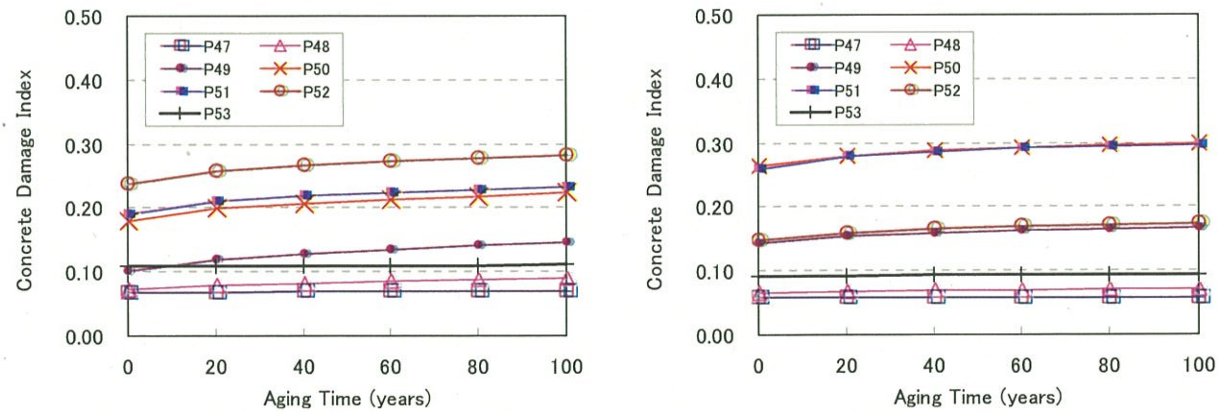
Fig. 10 Pier steel damage index



(a) Longitudinal direction

(b) Transverse direction

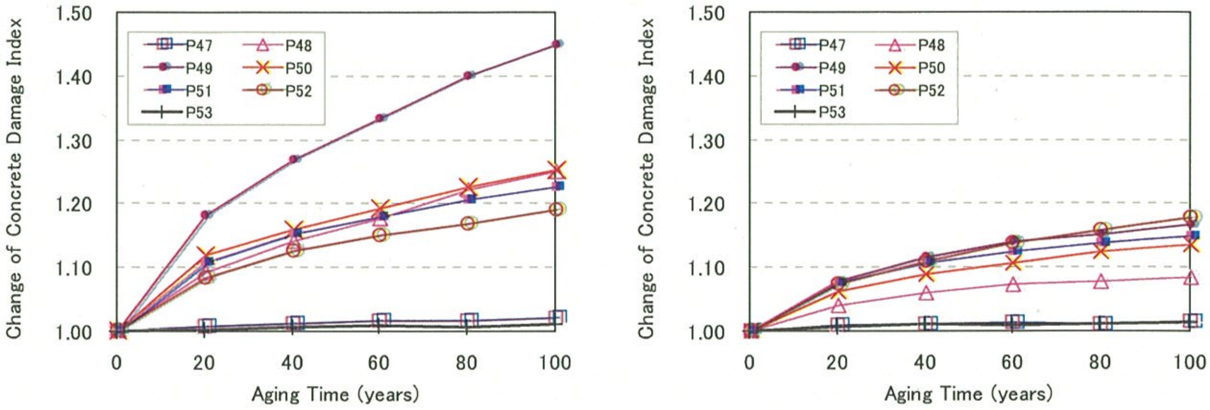
Fig. 11 Change of pier steel damage index



(a) Longitudinal direction

(b) Transverse direction

Fig. 12 Pier concrete damage index



(a) Longitudinal direction (b) Transverse direction

Fig. 13 Change of pier concrete damage index

4. CONCLUSIONS

Through dynamic analysis, the long-term seismic performance of a base-isolated six-span continuous steel bridge was evaluated by considering aging of rubber bearings. The long-term performance of rubber bearings was first examined for the conditions particular for this bridge. The aging effect of rubber bearings on the seismic performance was discussed in terms of the pier damage levels, and residual displacements. The following conclusions can be drawn from the study:

1) The equivalent shear stiffness of LRB increases due to aging of natural rubber, which causes the seismic force on the bridge to increase.
2) The base-isolated bridge analyzed in this study will satisfy seismic performance requirements in 100 years. It means that rubber bearings do not need to be replaced for 100 years in this particular case.

Although the case study presented in this paper shows that the aging effect of rubber bearings on the seismic performance is not significant, the effect may be significant in an isolated bridge that does not have much redundancy in the seismic design. It is, therefore, important to evaluate future seismic performance of a base-isolated bridge including the aging effect of rubber bearings so that an appropriate maintenance strategy for a life span of the bridge can be determined at the initial planning phase.

5. REFERENCES

[1] The Society of Rubber Industry, Japan, *Handbook of Isolated Rubber-Metal Laminated Bearing for Designers*, Science and Technology Book, 2000.

[2] Itoh, Y., Gu, H.S., Satoh, K., and Kutsuna, Y., "Experimental investigation on aging behaviors of rubber used for bridge bearings," *Journal of Structural Mechanics and Earthquake Engineering,* JSCE, No.808/I-74, pp. 17-31, 2006.

[3] Itoh, Y., Satoh, K., Gu, H.S., and Yamamoto, Y., "Study on the deterioration characteristics of natural rubber bearings," *Doboku Gakkai Ronbunsyuu*, Vol. 62, No. 2, pp. 255-266, 2006 (in Japanese).

[4] Itoh, Y. and Gu, H.S., "Seismic response of steel bridge piers with aged-based isolated rubber bearing," *Journal of Structural Engineering,* JSCE, Vol. 52A, pp. 537-546, March 2006.

[5] Japan Road Association, *Design Specifications of Highway Bridges*, V, 2002 (in Japanese).

[6] Usami, T., Suzuki, M., Mamaghani, I.H.P., and Ge, H.B., "A proposal for check of ultimate earthquake resistance of partially concrete-filled steel bridge piers," *Structural Mechanics/Earthquake Engineering*, JSCE, Vol. 525 (I-33), pp. 69-82, 1995.

[7] Zheng, Y., Usami, T., and Ge, H. B., "Ductility evaluation procedure for thin-walled steel structures," *Journal of Structural Engineering,* ASCE, Vol. 126, No.11, pp. 1312-1319, 2000.

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It is a well-known fact that aging causes shear stiffness of rubber to increase. Rubber bearings commonly used as base-isolators also tend to increase their stiffness over time. To accurately evaluate future seismic performance of a base-isolated bridge, it is, therefore, important to consider the aging effect of rubber bearings. This paper presents a case study to examine the effect of aging of rubber bearings on the seismic response of a base-isolated six-span continuous steel bridge with lead rubber bearings (LRB). The long-term performance of rubber bearings is estimated based on the size of bearing, an average temperature of the construction site, and various aging times up to 100 years. Seismic performance of the bridge is evaluated through dynamic analysis of a three-dimensional finite element bridge model. The results show that the aging of rubber bearings causes an increase in the seismic force on the bridge piers, resulting in larger damage in the piers.
Keywords: rubber bearing, aging, base isolation, seismic performance, steel bridge

1. INTRODUCTION

Aging causes rubber to stiffen and its tensile strength and elongation at break to decrease. As aging causes shear stiffness of rubber material to increase, rubber bearings also tend to increase their stiffness over time. In the current design specifications, the stiffness increase of rubber bearings due to aging is not considered. To examine the effect of aging of rubber bearings on the seismic response of a base-isolated bridge, a case study was conducted. A base-isolated six-span continuous steel box girder bridge with lead rubber bearings (LRB) owned by Nagoya Expressway Public Corporation was examined through dynamic analysis. The long-term performance of LRBs was first evaluated for this particular bridge, and LRB stiffness was calculated for 20, 40, 60, 80, and 100-year aging times. In the dynamic analysis, the aging effect was included in the LRB stiffness for the corresponding aging time. To examine the aging effect of rubber on the seismic performance, the peak responses from the dynamic analysis of the bridge with aged bearings were compared with those for the initial state (the bridge with new bearings). The finite element (FE) model of the bridge is shown in Fig. 1. The bridge has T-type steel piers, two main girders between Pier 47 and Pier 49, and three main girders between Pier 49 and Pier 53. The pier heights range from 15.4 to 15.8 m. Piers have stiffened steel box sections, and are filled with concrete at the bottom.

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Itoh [1, 2, 3] examined the time-dependent and temperature-dependent aging characteristics of natural rubber (NR) bearings, and developed simple formulas to calculate a change in the equivalent shear stiffness of the NR bearing. Generally, the shear stiffness increases with aging time and temperature. Based on the bearing size, aging time, and average yearly temperature at the construction site, the horizontal shear stiffness of an NR bearing can be calculated. The equivalent shear stiffness of LRBs was calculated for an average temperature of Nagoya City (15.4°C) and various aging times up to 100 years. The maximum stiffness change after 100 years is about 9% and 8% for the longitudinal and transverse directions, respectively. Due to the increase in the bearing equivalent shear stiffness, the global natural period of the bridge decreased by about 4% after 100 years.

3. SEISMIC RESPONSE OF SIX-SPAN BASE-ISOLATED CONTINUOUS STEEL BRIDGE DUE TO AGING OF RUBBER BEARINGS

Dynamic analysis of the bridge was performed by using the general purpose finite element analysis program, ABAQUS. LRBs were modeled as truss elements with a bilinear force-displacement relationship. Mass elements were used to account for mass of girders, transverse beams, footings, and the influence of dead loads from the adjacent bridges on Pier 47 and Pier 53. However, seismic force from the adjacent bridges was not considered. Girders, transverse beams, and piers were modeled with beam elements. Three earthquake records specified in Design Specifications of Highway Bridges [4] for Level 2 Type II and soil type III were used in this study. Fig. 2 shows hysteretic curves obtained at the bearings on Pier 48 for the initial state and the state after 100-year aging time. It can be seen from the figure that the aging of a rubber bearing results in an increase in the bearing shear stiffness and in the horizontal force. Since the increase in the bearing stiffness causes the global natural period to decrease, acceleration responses of the base-isolated bridge increase, and, consequently, seismic forces on the piers increase. At Pier 48, for the longitudinal direction, the maximum bearing shear force increased by 6% after 100 years while the maximum bearing shear displacement decreased by 7%.

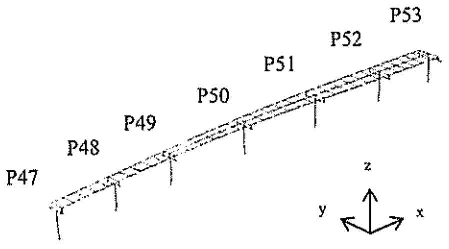

Fig. 1 FE model of the bridge

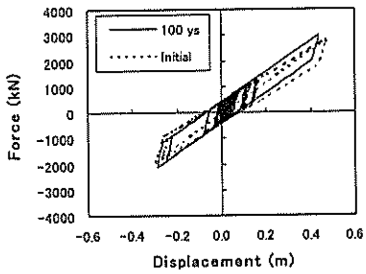

Fig. 2 Hysteretic curve of bearing on Pier 48 for the longitudinal direction

In this paper, the seismic performance of concrete-filled steel piers is discussed in terms of residual displacements and damage indices. The dynamic analysis showed that pier residual displacements tend to increase with the aging time. After 100 years, the residual displacement increased by 150% at Pier 49 for the longitudinal direction and by 40% at Pier 52 for the transverse direction. At the aging time of 100 years, the maximum residual displacement is only 24% and 28% of the allowable value for the longitudinal and transverse directions, respectively. All piers were, therefore, found to satisfy the performance requirement for the residual displacement.

Local buckling examinations are also required in the seismic performance evaluation of steel piers to ensure the pier shows ductile behavior during a severe earthquake. Two damage indices were used to evaluate the state of a concrete-filled steel pier: one for a steel section; the other for a concrete-filled steel section. A damage index is defined as a ratio of the average strain over an effective failure length to the failure strain. To determine capacity of a concrete-filled steel column, a failure criterion proposed by Usami [5] was used in this study. Results from the dynamic analysis showed that both damage indices increase with aging time because the seismic force increases due to aging. The maximum increase of the damage index for the steel section is about 40% at Pier 49 for the longitudinal direction, and 27% at Pier 52 for the transverse direction. The maximum increase of the damage index for the concrete-filled steel section is about 45% at Pier 49 for the longitudinal direction, and 18% at Pier 52 for the transverse direction. However, both damage indices are much smaller than 1.0 after 100 years, implying that the piers will have much redundancy against Level 2 earthquakes even when the aging effect of 100-year old rubber bearings is considered.

4. CONCLUSIONS

Through dynamic analysis, the long-term seismic performance of a base-isolated six-span continuous steel bridge was evaluated by considering aging of rubber bearings. The long-term performance of lead rubber bearings (LRB) was first examined for the conditions particular for this bridge. The aging effect of rubber bearings on the seismic performance was discussed in terms of the pier damage levels and residual displacements. Based on the results from this study, the following conclusions can be drawn.

(1) The equivalent shear stiffness of LRB increases due to aging of natural rubber, which causes the seismic force on the bridge to increase.

(2) The base-isolated bridge analyzed in this study will satisfy seismic performance requirements in 100 years. Therefore, rubber bearings do not need to be replaced in this particular case.

Although the case study presented in this paper shows that the aging effect of rubber bearings on the seismic performance is not significant, the effect may be significant in an isolated bridge that does not have much redundancy in the seismic design. It is, therefore, important to evaluate future seismic performance of a base-isolated bridge including the aging effect of rubber bearings so that an appropriate maintenance strategy for a life span of the bridge can be determined at the initial planning phase.

5. REFERENCES

[1] Itoh, Y., Gu, H.S., Satoh, K., and Kutsuna, Y., "Experimental investigation on aging behaviors of rubber used for bridge bearings," *Journal of Structural Mechanics and Earthquake Engineering,* JSCE, No.808/I-74, pp. 17-31, 2006.

[2] Itoh, Y., Satoh, K., Gu, H.S., and Yamamoto, Y., "Study on the deterioration characteristics of natural rubber bearings," *Doboku Gakkai Ronbunsyuu*, Vol. 62, No. 2, pp. 255-266, 2006 (in Japanese).

[3] Itoh, Y. and Gu, H.S., "Seismic response of steel bridge piers with aged-based isolated rubber bearing," *Journal of Structural Engineering,* JSCE, Vol. 52A, pp. 537-546, March 2006.

[4] Japan Road Association, *Design Specifications of Highway Bridges*, V, 2002 (in Japanese).

[5] Usami, T., Suzuki, M., Mamaghani, I.H.P., and Ge, H.B., "A proposal for check of ultimate earthquake resistance of partially concrete-filled steel bridge piers," *Structural Mechanics/Earthquake Engineering,* JSCE, Vol. 525 (I-33), pp. 69-82, 1995.