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Seismic Response of Steel Bridge Piers with Aged Base-Isolated Rubber Bearing

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It is usually known that ageing causes rubber performance to drop. The purpose of this study is to evaluate the influence of aged rubber bearings on the seismic response of single steel bridge piers. Firstly, the relationship between ageing and base-isolated rubber bearings' long-term performance is studied. Then the steel bridge piers with base-isolated rubber bearings are modeled as two-degree of freedom systems, and through dynamic analysis the response and damage of the structures are calculated. Next, the rubber bearing is replaced by an aged one and the seismic performance of the pier is evaluated again. The influence of rubber bearing's ageing is compared and discussed. Finally, a procedure to determine the maintenance strategy of a base-isolated rubber bearing is proposed based on the long-term seismic performance of the steel bridge pier.

Key Words: steel bridge pier, rubber bearing, ageing, seismic response

1. Introduction

During the 1995 Hyogoken-Nanbu earthquake, severe damages were observed in steel and concrete piers, which are often utilized in Japan highway bridge systems. Consequently, a special attention has been attracted on the seismic design methodologies of such structures. Generally speaking, the intent of seismic design is to ensure serviceability during moderate earthquakes and to prevent collapse during severe earthquakes. As a result, base-isolated bearings have been widely adopted, because rubber is an ideal material to withstand a large deformation and absorb energy for its high elasticity, high damping and large elongation at failure. Thus, the laminated rubber bearings installed between the superstructure and the supporting piers can provide satisfying vertical stiffness and horizontal flexibility.

In recent years, many research projects have been carried out on the dynamic performance of rubber bearings[1] or on the seismic response of the steel piers installed with rubber bearings[2)~4)]. Also, many efforts have been made to develop more precise models reflecting the highly nonlinear characteristics of rubber materials or the hysteresis model of rubber bearings[5),6)]. Meanwhile, the ageing problems have been realized for many years[7)]. It is usually known that ageing causes rubber performance to drop, showing itself by stiffening of the rubber, causing an increase of hardness and a decrease in tensile strength as well as elongation at break. However, there are only few studies so far on the ageing of bridge rubber bearings because of their limited application history and the lack of enough fundamental experimental data. Even in the present *Design Specifications of Highway Bridges*[8)] and *Manual of Isolated Design Method for Highway Bridge*[9)] the long-term behavior of rubber bearing is not considered during the design process, although the rubber material is exposed to variant environments and attacked by different degradation factors such as oxidation, ultraviolet, ozone, temperature, acid and humidity. Without accurate predictions of the future performance of rubber bearings, the safety of a bridge against the earthquake cannot be secured. Moreover, without knowing the lifespan of the rubber bearing, economic problems may arise because of the unnecessary replacement cost. Therefore, the objective of this research is to investigate the long-term performance of aged rubber bearings and its influence on the seismic response of isolated single steel bridge piers.

From the previous research by Itoh et al. [10)~13)], it is already known that oxidation plays the most important role among all the deterioration factors affecting rubbers. And for thick rubber, the material properties do not change uniformly because of the diffusion-limit oxidation effect. The deterioration profiles inside natural rubber (NR) blocks have been studied through accelerated thermal oxidation test, and the long-term performances of bridge natural rubber bearings (NRB) were estimated using FEM analysis. Based on the above research, this study will focus on the performance of aged NRB with lead plugs and its influence on the seismic performance of single steel bridge piers. The durability of base-isolated NRB with lead plugs will be evaluated considering the performance-based design criteria for steel structures.

2. Long-term performance of base-isolated natural rubber bearing

Itoh et al. [13] found that the equivalent horizontal stiffness of NRB is related to time, temperature and bearing size. Generally, the equivalent horizontal stiffness increases over the time. Under higher temperature, the increases speed is faster. However, for the bearing with larger size this property changes more slowly. A simplified calculation method of the equivalent horizontal stiffness for aged NRB under service conditions is proposed in Ref.(13) as follows:

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

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

$$\Delta f_s = 0.066 t_{ref}^{\ 0.515} \quad (3)$$

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

where, K_h is the equivalent horizontal stiffness of NRB, G_0 is the static shear modulus of NR in the initial state. The symbols a and b are the length and width of the NRB. n is the number of rubber layers and t_R is the rubber layer thickness. d^* is the critical depth to which oxidation can progress, and it can be calculated by Eq.(2). Δf_s is the relative change of the static shear modulus on the bearing's side surface. In Eq.(2) and Eq.(3), the symbols α and β are coefficients. T is the local temperature where the NRB is mounted. t_{ref} is the ageing time in the thermal oxidation accelerated test chosen as reference. Eq.(4) is the formula of the Arrhenius methodology, in which, E_a is the activation energy (=9.94×10^{4}J/mol for NR), R is the gaseous constant (=8.314[J/mol·K]), t is the ageing time under service condition, and T_{ref} is the temperature in the reference test.

The thermal oxidation test under 60℃ is taken as reference and the coefficients in Eq.(1)～Eq.(4) are listed in Table 1.

Table 1 Coefficients in Eq.(1)～Eq.(4)

Coefficient	Value
α [10^{-4}mm]	8.00
β [10^{3}K^{-1}]	3.31
T_{ref}[K]	343

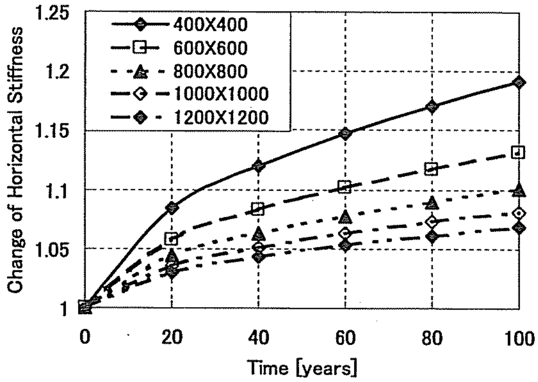


Fig.1 Long-term performance of NRB in Tokyo (15.9℃)

Using the calculation method mentioned above, the long-term performance of NRB of any size can be estimated considering the temperature on site. For example, the changes of the equivalent horizontal stiffness of different size NRB in Tokyo are illustrated in Fig.1. It is found that after 100 years, the equivalent horizontal stiffness of a 600×600 NRB increases about 13%, and nearly 20% for a 400×400 NRB. Here the average yearly temperature is assumed constant in the next 100 years.

As for base-isolated rubber bearing, it is required that the bearing should not only possess the ability of large deformation, but also absorb energy. In order to add energy dissipation to the flexibility already exists, usually lead plugs are inserted in the laminated natural rubber bearing. This kind of bearing is called lead rubber bearing (LRB). In the non-linear hysteresis model of LRB, the equivalent horizontal stiffness is calculated using the following equation[14]:

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

where, K_{Be} is the equivalent horizontal stiffness of LRB, F is the shear force corresponding to the effective shear strain γ_e. u_{Be} is the effective design displacement. G_e is the static shear modulus of NR, A_e is the area of LRB excluding the surface rubber, A_p is the area of lead plugs, and $q(\gamma)$ is the shear modulus of the lead plug. (=a_0+$a_1\gamma$+$a_2\gamma^2$+$a_3\gamma^3$ N/mm^2).

From Eq.(5) it can be seen that the equivalent horizontal stiffness of LRB K_{Be} is determined by the shear modulus of NR and the shear force of lead plug. Since NR stiffens due to the ageing, G_e increases, and K_{Be} will increase consequently. Here the property change ratio is concerned. Combined with Eq.(1), the following equation can be obtained.

$$\frac{K_{Be}}{K_{Be,0}} = \frac{K_h / K_{h,0} + C}{1+C} \quad (6)$$

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

where, $K_{Be,0}$ and K_{h0} are the equivalent horizontal stiffness of

LRB and NRB in initial state, respectively. C is the shear stiffness ratio of lead plug versus rubber.

According to the *Manual of Highway Bridge Bearing*[14], the area ratio of lead plug versus rubber layer (A_p/A_e) is usually 3～10%. The allowable shear strain γ_{ea} corresponding to Level 2 earthquake is 250% and the effective shear strain γ_e is 70% of which, namely 175%. Using the factors in Table 2, it is known that $q(\gamma)$=2.5. Thus, C=3%～24%. The relationships between the equivalent horizontal stiffness change ratio of LRB and NRB with different C are shown in Fig.2. It is clear that the change of the equivalent horizontal stiffness of LRB due to the deterioration is always less than that without lead plugs. If the area of the lead plugs is small compared with the area of rubber layer, or the shear modulus of the rubber material is large, the influence of the lead plug will be trivial. Therefore, the deterioration performance of LRB can be easily obtained from NRB of the same size. For instance, the area ratio of lead plug versus rubber layer A_p/A_e=10%, G_e=1.2 N/mm^2, then C=12%. So the long-term performances of LRBs of different sizes in Tokyo can be estimated based on the knowledge mentioned

Table 2 Factors of lead plug shear modulus (N/mm^2)

Condition	a_0	a_1	a_2	a_3
$0.50<\gamma_e\leq 2.00$	16.0	-12.6	2.8	—

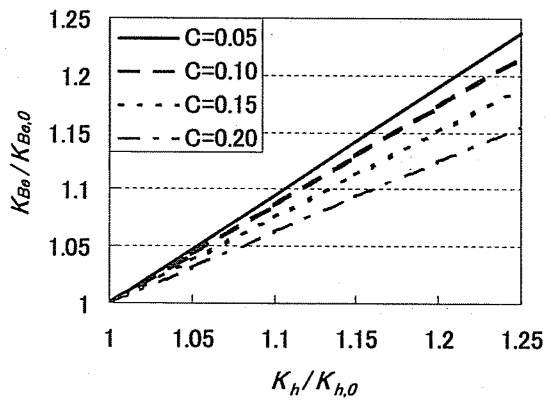


Fig.2 Relationship between equivalent horizontal stiffness change ratio of LRB and NRB

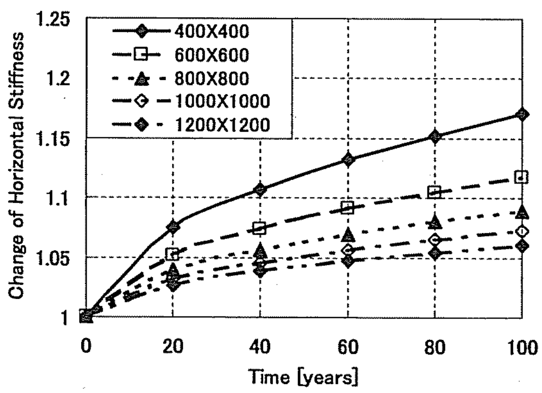


Fig.3 Long-term performance of LRB in Tokyo (15.9℃)

above, as shown in Fig.3. After 100 years, the equivalent horizontal stiffness of a 600×600 LRB increases about 12%, and about 17% for a 400×400 NRB.

3. Required performance for single steel bridge piers

The current JRA specifications[8] suggest that dynamic analysis should be carried out to check that the seismic behavior do not exceed structural capacity in order to secure that bridges can resume normal function as quickly as possible after a major earthquake. Various performance criteria have been explicitly specified in recent seismic design specifications[15]. After the 1995 Hyogoken-Nanbu earthquake, the revised JRA specifications[8] proposed a two-level seismic design method for moderate (called Level 1) and major (called Level 2) earthquake respectively. In 2004, Japanese Society of Steel Construction (JSSC)[16] proposed a modified required performance matrix with respect to the structural importance, as shown in Fig.4. As for Level 1 earthquake, it is required that no damage should occur for any structure. While for Level 2 earthquake, the following seismic performance requirements should be satisfied:

The Most Important Structure: Small damage is allowable. The damage could be repaired in very short time, and it should be passable to ordinary vehicles while being repaired.

Important Structure: Medium damage is allowable. It may take some time to repair although the basic functions remain. Only emergency vehicles are passable.

Normal Structure: The structure does not collapse, but has lost functions. It will cost a long time to repair the damage and reinforcement is needed.

Based on the prescribed performance, the serviceability after earthquake could be checked according to the residual displacement of the pier. However, it is difficult to calculate the residual displacement accurately through dynamic analysis[17].

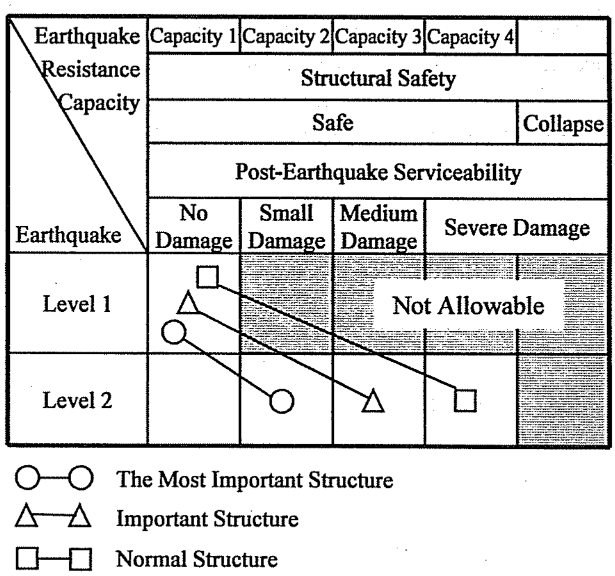


Fig.4 Required performance matrix[16]

Table 3 Allowable performances for steel Ppiers (Level 2)

Importance	Most Important	Important	Normal
Safety	$\delta_{max} \leq \delta_u$		
Serviceability	$\delta_{max} \leq 1.7\delta_y$	$\delta_{max} \leq 4.0\delta_y$	—

δ_u: Ultimate displacement of the pier;

δ_y: Yield displacement of the pier.

Since there're some relations between the residual displacement and the maximum displacement[16], it is better to utilize these relations and to convert the residual displacement to the maximum displacement. Therefore, the limitations of the maximum displacement of a concrete-non-filled steel pier under Level 2 earthquake are presented in Table 3.

According to the current JRA specifications[8], in this study, the ultimate displacement δ_u is chosen as the displacement value corresponding to the cyclic strength (under reversed static cyclic loading) dropping to 95% of the peak cyclic strength. δ_u for box-section steel bridge piers can be calculated by the following empirical equation[2]:

$$\frac{\delta_u}{\delta_y} = \frac{0.0147}{\left\{(1+P/P_y)R_f\sqrt{\bar{\lambda}}\right\}^{3.5}} + 4.20 \qquad (8)$$

4. Analysis model of steel bridge piers and earthquakes

In this study, emphases will be laid on concrete-non-filled thin-walled stiffened steel box piers. The isolated steel pier with lead rubber bearing is modeled as a two-degree-of-freedom model as shown in Fig.5(a). The mass of the superstructure is M_u and the mass on the top of the pier M_p is taken as 30% of the mass of the whole pier. The linear acceleration method is used to resolve the elasto-plastic seismic response problem. According to the JRA specifications[8], the damping ratio ξ_P of the steel pier is taken as 0.01. As for the two-degree-of-freedom system, the proportional damping matrix proposed by Wilson-Penzien is used[18]. The time interval is set as Δt=0.001s.

A two-parameter model is adopted as the hysteresis model of the steel bridge pier[19], as shown in Fig.5(b). This model has been proved to be able to reproduce the seismic response under any earthquake precisely. Especially the maximum displacement agrees well with the pseudo-dynamic test results. The parameters of the two-parameter model is defined as:

$$R_f = \frac{b}{t}\sqrt{\frac{\sigma_{y,P}}{E}\frac{12(1-\nu^2)}{4\pi^2 n^2}} \qquad (9)$$

$$\bar{\lambda} = \frac{2h}{r}\frac{1}{\pi}\sqrt{\frac{\sigma_{y,P}}{E}} \qquad (10)$$

$$\frac{H_{m,P}}{H_{y,P}} = 0.101(R_f \cdot \bar{\lambda})^{-1.0} + 0.880 \qquad (11)$$

$$\frac{\delta_{m,P}}{\delta_{y,P}} = 0.00759(R_f\sqrt{\bar{\lambda}})^{-3.5} + 2.59 \qquad (12)$$

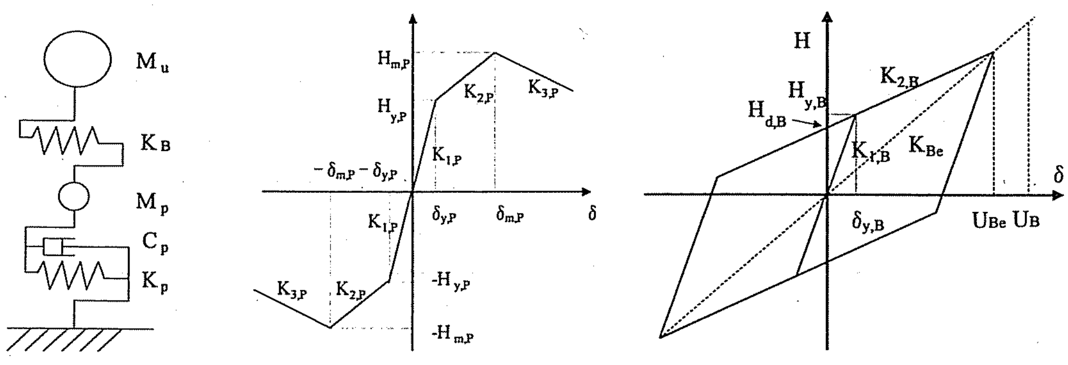
$$K_{2,P} = \frac{H_{m,P} - H_{y,P}}{\delta_{m,P} - \delta_{y,P}} \qquad (13)$$

$$K_{3,P} = -0.278(10 - \frac{\gamma}{\gamma^*})R_f^2(0.1 + \frac{P}{P_y})\bar{\lambda} \qquad (14)$$

where, R_f=width-thickness ratio parameter, $\bar{\lambda}$ =slenderness ratio parameter, E=Young's modulus, $\sigma_{y,P}$=yield stress, ν =Poisson's ratio, t=thickness, b=width of flange, n=number of sub-panels, h=pier height, r=radius of gyration of cross-section, $\delta_{y,P}$=yield displacement, $H_{y,P}$=yield horizontal load, $H_{m,P}$=the maximum horizontal load, $\delta_{m,P}$= the maximum horizontal displacement, $K_{2,P}$=hardening stiffness, $K_{3,P}$=degradation stiffness, γ/γ^*=flexural rigidity ratio of stiffener (=3.0), and P/P_y=axial load ratio. The parameters of the analyzed steel piers are presented in Table 4.

As for base-isolated rubber bearing, usually a bilinear model is used[14], as shown in Fig.5(c). The parameters of this model are defined following the next procedure[2]:

Step 1: Assume the target period T of the isolated pier and calculate the equivalent horizontal stiffness K_{Be} of the rubber bearing.



(a) Isolated steel pier model (b) 2-Parameter model of steel pier (c) Bilinear model of lead rubber bearing

Fig.5 Analysis model of base-isolated steel pier with lead rubber bearing

Table 4 Parameters of steel piers

Steel		SS400
Young's modulus	E	205GPa (21,000kgf/mm^2)
Yield stress	σ_y	235MPa (24kgf/ mm^2)
Poisson's ratio	υ	0.3
Steel plate thickness	t	20mm
Sub-plate thickness	t_s	20mm
Sub-plate width	b_s	233mm
Width-thickness ratio parameter	R_f	0.35
Slenderness ratio parameter	$\bar{\lambda}$	0.20, 0.30, 0.40, 0.50
Stiffener number on flange		3 (Sub-panel number=4)
Stiffener number on web		3 (Sub-panel number=4)
Flexural rigidity	γ	$\geq 3\gamma^*$ (γ^*=Optimum flexural rigidity)
Diaphragm space	l_d	b (b=section width)

$$K_{Be} = \frac{1}{\frac{1}{M_u}\left(\frac{T}{2\pi}\right)^2 - \frac{1}{K_{1,P}}} \tag{15}$$

Step 2: Assume the maximum response displacement of the bearing U_B. The effective displacement U_{Be} can be determined as 70% of U_B.

Step 3: Determine the optimum yield load of the rubber bearing H_{dB} to minimize the energy absorbed by the pier[2)], which is related to the natural period T and the type of ground.

Step 4: Determine the skeleton curve of the hysteresis model of rubber bearing based on the stiffness equations given by:

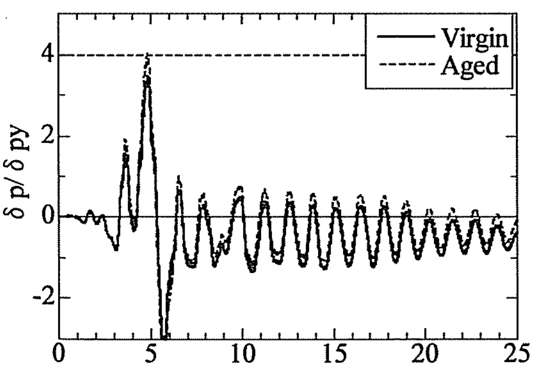
$$K_{2,B} = K_{Be} - \frac{H_{d,B}}{U_{Be}} \tag{16}$$

$$K_{1,B} = 6.5K_{2,B} \tag{17}$$

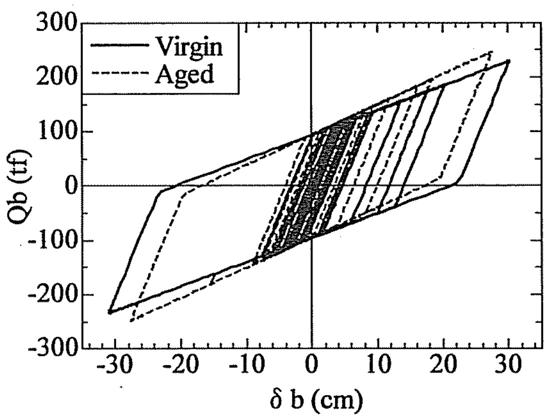
Step 5: Input the earthquake waves and calculate the maximum response displacement of the rubber bearing. If the difference between the result and the value assumed in **Step 2** is within the acceptable tolerance of 10%, then finish the design. If not, go back to **Step 2** and assume U_B again. Continue the iteration until getting the convergence.

It is already known when rubber material deteriorates, the equivalent stiffness of the base-isolated rubber bearing increases. In this study, it is assumed that the equivalent stiffness K_{Be} of rubber bearing increases while the yield load H_{dB} keeps constant, since in LRB the yield load H_{dB} is determined by lead plugs inside the bearing, which are not easily affected by environmental factors. So the primary stiffness $K_{1,B}$ and the secondary stiffness K_{2B} will change correspondingly. The seismic responses are compared with the initial state.

There are two types of earthquakes belonging to Level 2 earthquake. Type I is great plate-boundary earthquake and Type II is severe near-field earthquake. In dynamic analysis the JRA code[8)] prescribes three Level 2・Type I and three Level 2・Type II accelerograms for each ground type (Ground Type I, II and III), and suggests the average responses of three accelerograms in each ground group be taken as the final results.



(a) Displacement response of steel pier



(b) Hysteresis curve of isolation rubber bearing

Fig.6 Analysis example using Fukiai earthquake (R_f=0.35, $\bar{\lambda}$=0.40, T=1.80s)

5. Dynamic analysis results and discussions

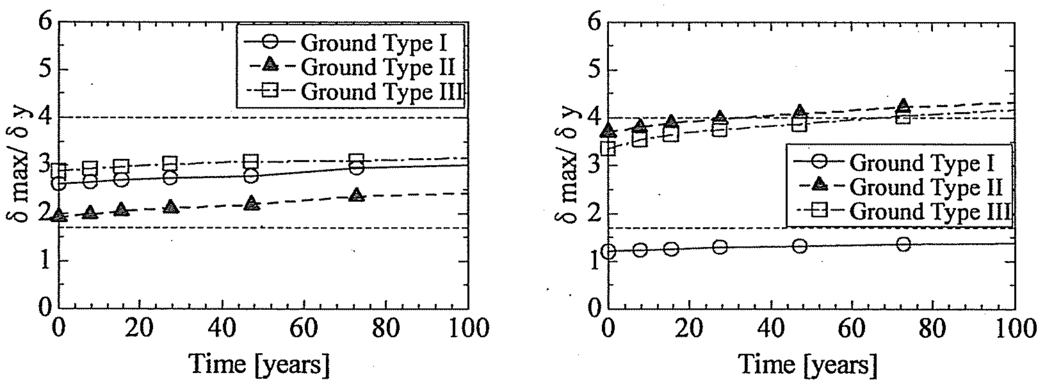
In order to explain the methodology of dynamic analysis, here an example is presented using the isolated concrete-non-filled thin-walled stiffened single steel box pier with R_f=0.35, $\bar{\lambda}$=0.40. The inputted earthquake excitation is the Fukiai accelerogram, which is recorded on Ground Type II and belongs to Level 2・Type II earthquake. The natural period of the pier is 0.88s. Usually the target period T of the isolated pier is set as double of the natural period, here T=1.80s. Then the lead rubber bearing is designed. It is assumed the size of the bearing is 600×600 and other conditions are the same as mentioned before. The bridge is located in Tokyo where average yearly temperature is 15.9℃. From Fig.3 it is known that after 100 years, the equivalent horizontal stiffness of the base-isolated LRB increases by 12%. The seismic responses of the steel pier and the LRB before and after deterioration are compared in Fig.6 as well as in Table 5.

Table 5 Results of analysis example

Item	Virgin	Aged	Change Ratio
Period T(s)	1.8	1.73	-4.2%
Pier δ_{max}/δ_y	3.47	4.02	+15.9%
LRB K_{Be}(tf/cm)	8.80	9.86	+12.0%

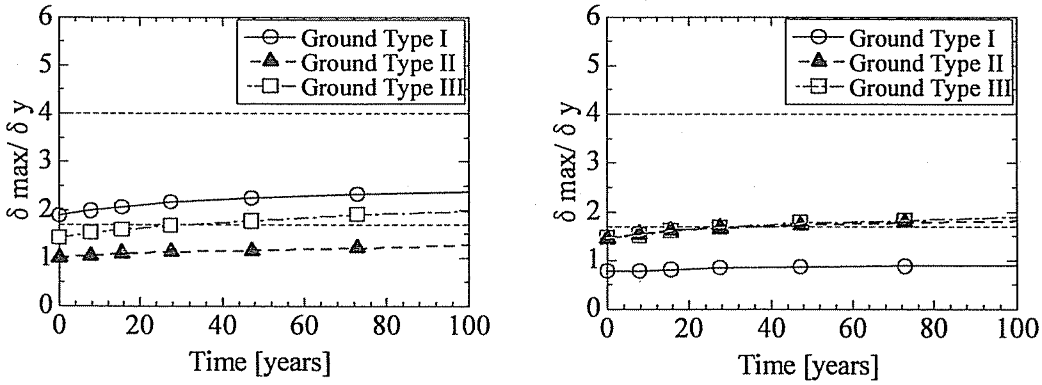
As shown in Fig.6(b), after 100 years the equivalent stiffness of aged rubber bearing increases 12%, which causes the natural period of the isolated pier to decrease from 1.8s to 1.73s. So that the acceleration response, or in other words, the earthquake force acting on the isolated pier will be larger, which consequently causes the maximum displacement of the pier to increase. From Fig.6(a) it can be seen that after deterioration, the maximum displacement of the pier increases by 15.9%. δ_{max}/δ_y changes from 3.47 to 4.02, exceeding the boundary prescribed for important structures. That is to say, after 100 years, the steel pier has already lost its ability to maintain the serviceability after a major earthquake.

Similarly, the displacement responses of the isolated steel pier (R_f=0.35, $\bar{\lambda}$=0.40) on other types of ground with target period of 1.80s are illustrated in Fig.7. In the design of the rubber bearing, the optimum yield load H_{dB} is determined by the type of ground, therefore, the initial δ_{max}/δ_y is different for each type of ground. It is already known that the variation of the pier response is related to the performance change of the LRB. Combined with long-term performance prediction model for base-isolated natural rubber shown in Fig.3, the relationship between the displacement response and the ageing time can be obtained. Here the LRB with the size of 600×600 is investigated. In Fig.7, the horizontal axis is the ageing time from 0 to 100 years, and the vertical axis is the maximum



(a) Type I earthquake (b) Type II earthquake

Fig.7 Displacement response of steel pier (R_f=0.35, $\bar{\lambda}$=0.40, T=1.80s)



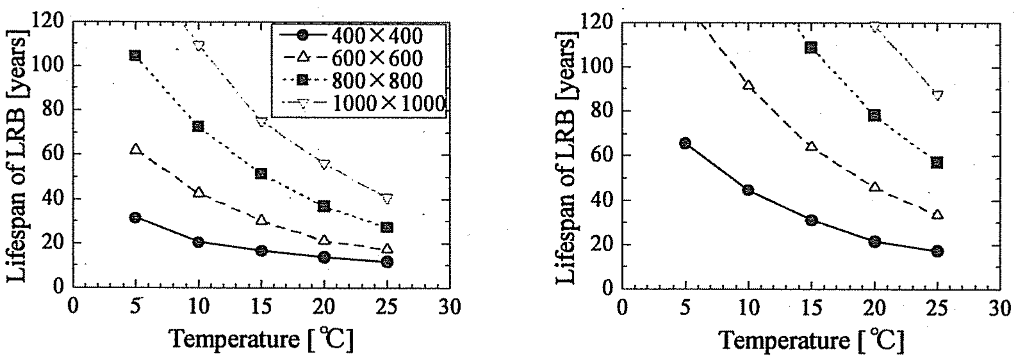
(a) Type I earthquake (b) Type II earthquake

Fig.8 Displacement response of steel pier (R_f=0.35, $\bar{\lambda}$=0.40, T=2.40s) displacement of the steel pier versus yield displacement. From Fig.7(a), it can be seen for important structure under Level 2・Type I earthquake, the LRB can fulfill its task well during 100 years. In Fig.7(b), the earthquake is Level 2・Type II. The displacement response of the steel pier is even less than the limitation for the most important structure when constructed on Ground Type I. However, when constructed on Ground Type II and III, the maximum displacement becomes larger than the allowable value for important structure at about 36 years and 64 years, respectively. It means the 600×600 LRB should be replaced before that time in order to survive after a major earthquake.

If the importance level of the structure is classified as most important, then the target period is extended to 2.40s to reduce the seismic response. The results are shown in Fig.8. If the earthquake is Level 2・Type I as shown in Fig.8(a), the LRB cannot satisfy the requirement on Ground Type I. On Ground Type II, the serviceability can be ensured even after 100 years. When constructed on Ground Type III, the maximum displacement δ_{max} will be larger than 1.7 times of the yield displacement δ_y after about 30 years. On the other hand, if the designed earthquake belongs to Level 2・Type II, it is safe for Ground Type I. However, for Ground Type II and III, the changes of the displacement response due to the ageing of LRB are almost the same, across the boundary at about 30 years. So the maintenance strategy of the 600 × 600 LRB can be determined. In Tokyo, if the ground type is II, and assume the earthquake is a severe near-field earthquake, the maintenance strategy of a 600×600 LRB is about 30 years for both the important and the most important bridges.

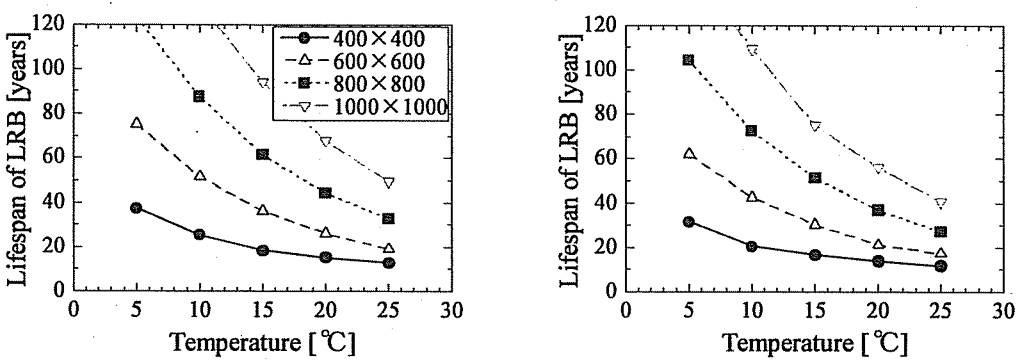
Using the Eqs.(1) ~(7), the lifespan of LRB with different sizes and under different temperature is illustrated in Fig.9 and Fig.10. If the pier with R_f=0.35, $\bar{\lambda}$=0.40 is attacked by Level II earthquake, there's no need to replace the LRB during 100 years if the pier is located on Ground Type I. It can be seen the lifespan of LRB becomes shorter under higher temperature. And for the LRB with larger size, the replacement period is longer. With the help of this method, it will be easy for engineers to decide the optimum maintenance strategy.

The seismic responses of other steel piers were investigated too, as shown in Fig.11 and Fig.12. The horizontal axis is the slenderness ratio parameter $\bar{\lambda}$ varying from 0.2 to 0.5. As mentioned above, it is assumed that the equivalent stiffness of the aged LRB increases by 12%. In Fig.11, the bridge pier is an important structure with R_f=0.35, and the natural period is 1.8s. The rubber bearing does not need to be replaced if attacked by Level 2・Type I earthquake. However, if the earthquake is Level 2・Type II, the ageing problem must be taken into consideration when the pier is located on Ground Type II and III. For Ground Type II, the displacement response of the pier with $\bar{\lambda}$=0.3~0.5 exceeds the limitation after the LRB is aged. While on Ground Type III, it will be dangerous for the pier with $\bar{\lambda}$=0.2



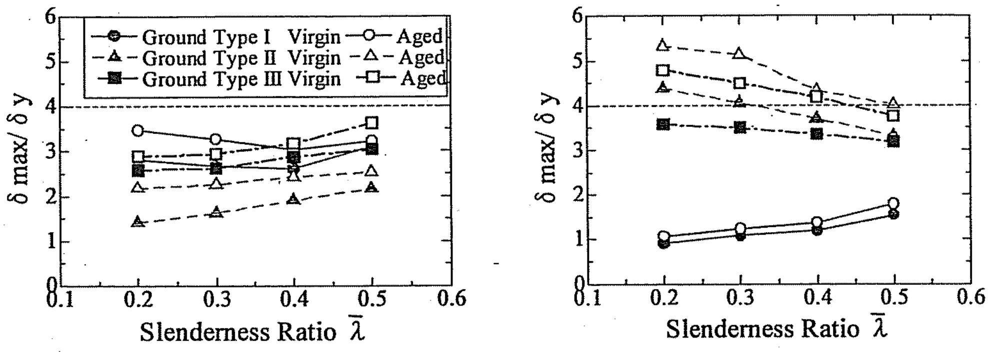
(a) Ground Type II (b) Ground Type III

Fig.9 Lifespan of LRB (Important structure, R_f=0.35, $\bar{\lambda}$=0.40, T=1.80s, Level 2・Type II earthquake)



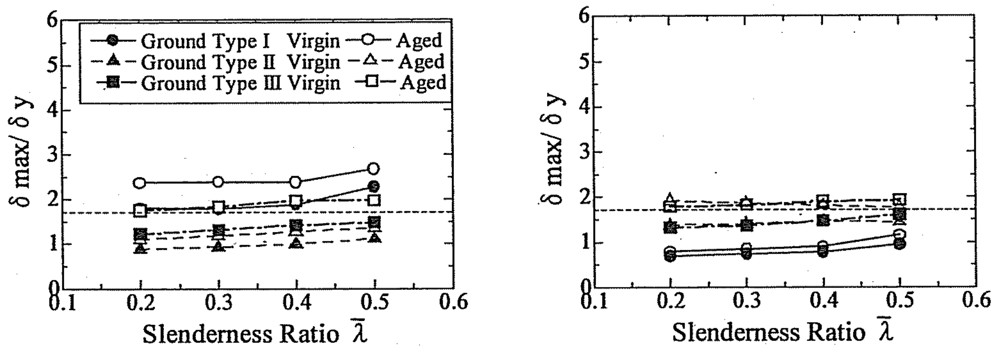
(a) Ground Type II (b) Ground Type III

Fig.10 Lifespan of LRB (Most important structure, R_f=0.35, $\bar{\lambda}$=0.40, T=1.80s, Level 2・Type II earthquake)



(a) Type I earthquake
(b) Type II earthquake

Fig.11 Seismic response of different steel piers (R_f=0.35, T=1.80s)



(a) Type I earthquake
(b) Type II earthquake

Fig.12 Seismic response of different steel piers (R_f=0.35, T=2.40s)

~0.45.

Fig.12 shows the seismic responses of the most important piers with the natural period T=2.4s. If the earthquake is Level 2·Type I as shown in Fig.12(a), the response of Ground Type I is always larger than the required performance. Ageing problem does not need to be considered for Ground Type II, but must be considered for Ground Type III. Fig.12(b) shows the response under Level 2·Type II earthquake. The responses on Ground Type II and III are almost the same, and both exceed the limitation after the LRB is aged. It is safe for any pier Ground Type I because the maximum response is very small, no matter before or after ageing. From the acceleration response spectra it is known that at certain natural period, the earthquake force acting on the pier is related to the type of ground. For example, under Level 2 · Type II earthquake, when T=2.4s, the acceleration response of Ground Type I is far less than Ground Type II and III, while the latter two are very close.

The seismic response of the steel pier decreases when extending the natural period. Therefore, it is possible to avoid replacing the LRB by choosing appropriate natural period.

6. LRB maintenance strategy

From the dynamic analysis it is known that the seismic response of the steel pier increases with the ageing of LRB. Therefore, the maximum displacement is possible to exceed the allowable value. The lifespan performance should be checked to ensure the serviceability after a major earthquake. In this paper, a procedure is proposed in Fig.13 to determine the maintenance strategy of LRB so as to keep the steel pier always satisfying the performance requirement.

Step 1: Collect the basic information such as base-isolated pier parameters, LRB size, bridge location, structural importance, and so on. The location tells the information like average yearly temperature on site, ground type, and possible earthquake, etc.

Step 2: According to the structural importance the allowable performance δ_{lim} for steel piers can be determined.

Step 3: Using the parameters of the steel pier and the LRB, an appropriate nonlinear MDOF model could be made.

Step 4: Assume a maintenance strategy t_R of the LRB, which should be no longer than the lifespan of the bridge.

Step 5: Estimate the performance at time t_R using Eqs.(1)~(7) and build the bilinear model for aged LRB.

Step 6: Perform dynamic analysis on the nonlinear MDOF model using the accelerograms of the corresponding ground type. The maximum displacement of the steel pier will be obtained.

Step 7: Compare the maximum displacement δ_{max} with the allowable performance δ_{lim}. If δ_{max} is less than δ_{lim}, there's no need to replace the LRB during assumed time t_R. Otherwise, go to **Step 4** and assume a new

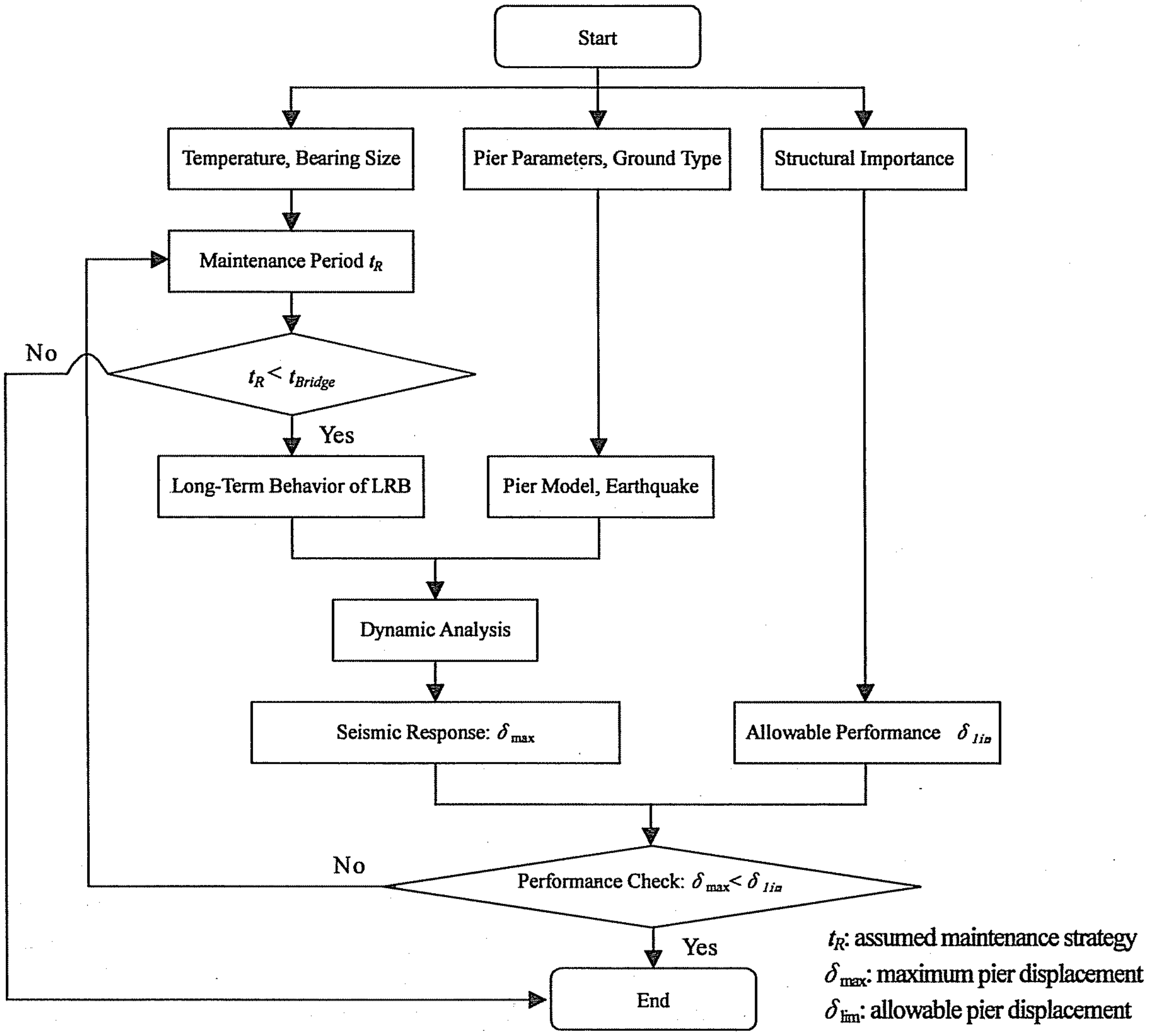


Fig.13 Determination flow chart of LRB's maintenance strategy

maintenance strategy shorter. Continue the iteration until the maximum displacement of the steel pier satisfies the performance requirement.

The assumed time t_R could be deemed as the maintenance strategy of the LRB. It is suggested the LRB be replaced before t_R to ensure the serviceability of the steel pier.

7. Summary and conclusions

In this study, the long-term performance of lead rubber bearing (LRB) is investigated and its influences on the seismic response of isolated concrete-non-filled thin-walled stiffened single steel box piers are discussed. The major findings and conclusions are summarized as follows.

1) The equivalent horizontal stiffness of LRB increases over the time, and the increase speed of the larger size is lower than the smaller one. Because of the influence of the lead plugs, the equivalent horizontal stiffness change of a LRB is always less than that of a natural rubber bearing (NRB) with the same size. They are correlated by the shear stiffness ratio of lead plug versus rubber, which is normally about 3%～24%.
2) Dynamic analysis was carried out on an example single steel bridge pier with R_f=0.35, $\bar{\lambda}$=0.40. It is found the displacement response increases with the ageing of LRB. 12% increase of the equivalent horizontal stiffness of LRB may cause the isolated pier with T=1.80s under the Fukiai accelerogram to lose its serviceability.
3) As for the base-isolated pier with R_f =0.35, $\bar{\lambda}$=0.40 located in Tokyo and to be attacked by Level 2・Type II earthquake, the maintenance strategies of a 600×600 LRB are about 36 years and 64 years for an important bridge on Ground Type II and III, respectively, and are about 30 years for the most important bridge on these two types of ground.
4) The lifespan of various LRBs installed on the pier with R_f =0.35, $\bar{\lambda}$=0.40 under various temperatures is calculated. The lifespan is shorter for small sized bearing or under higher temperature.
5) For important structure, the ageing problem should be

considered when the pier with R_f=0.35, T=1.80, which will be attacked by Level 2・Type II earthquake, is located on Ground Type II and III. As for the most important structure, if natural period T is 2.4s, attention should be paid to the cases such as the Ground Type III combined with Level 2・Type I earthquake, Ground Type II and III with Level 2・Type II earthquake.

6) A procedure to determine the maintenance strategy of LRB is proposed considering the serviceability of the steel bridge piers.

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