SEISMIC RESPONSE VARIATION OF STEEL BRIDGE PIERS WITH DETERIORATED RUBBER BEARING

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It is usually known that ageing causes rubber performance to drop. The purpose of this research is to study the influence of the deteriorated rubber bearing on the seismic response of the steel bridge piers. Through thermal oxidation tests on natural rubber blocks, the deterioration characteristics are revealed and a deterioration prediction model is proposed. The long-term performance of bridge natural rubber bearings is estimated using FEM analysis. Then numerical analysis is carried out to investigate the relationship between the horizontal stiffness increase and the seismic response of base-isolated steel piers. It is found that the deterioration of the rubber bearing has large effects on the seismic performance of base-isolated steel bridge piers, and in most cases the damage degree will upgrade to a higher level if the horizontal stiffness of the rubber bearing increases by 20%.

INSTRUCTION

Natural rubbers are often used in horizontal force distributing bridge rubber bearings and base-isolated bridge rubber bearings, because rubber is an ideal material to withstand large deformation and absorb energy because of its high elasticity, high damping and large elongation at failure. However, it is usually known that ageing causes rubber performance to drop, showing itself by stiffening of the rubber, causing a decrease in tensile strength as well as elongation at break and an increase of hardness. 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)]. Also, many efforts have been made to develop more precise models reflecting the highly nonlinear characteristics of rubber materials[3)]. Although the ageing problems of rubber materials have also been studied for many years[4)], the effects of rubber bearing ageing on the seismic response of base-isolated steel piers are seldom studied, because there are still many unknown issues relating to the long-term behaviors of rubber bearings. Even in the present *Design Specifications of Highway Bridges* by Japan Road Association (JRA), the long-term behavior of the rubber bearing is not considered during the design stage. Without accurate predictions of the future performance of rubber bearings, the safety of the bridge against the earthquake cannot be secured. The objective of this research is to investigate the mechanical performance of aged natural rubber (NR) bearing and its influence on the seismic response of base-isolated bridges.

From the previous research[5), 6)], 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. Therefore, in this research, firstly thermal oxidation test are performed on natural rubber blocks, and the property variation inside the block is investigated. Based on the test results, a deterioration prediction model is developed, and the Arrhenius methodology is used to correlate the accelerated ageing tests with ageing under service conditions. Then, the natural rubber bearing is modeled using the constitutive law and the finite element method (FEM). Through FEM analysis the performance of deteriorated bridge rubber bearing is studied. Next, the bridge rubber bearing is represented by a macro-model, and dynamic analyses are carried out to examine the seismic response variation of the steel pier due to the performance change of rubber bearing.

ACCELERATED THERMAL OXIDATION TEST ON RUBBER BLOCKS

It is obvious that for thick rubber, the surface is more easily affected by deterioration factors than the interior. In order to understand the variation of the material property inside the NR bearing, accelerated tests were performed on rubber blocks using the most significant factor – thermal oxidation[5]. It is already known that thermal oxidation results in the increase of NR's stiffness as well as the decrease of the elongation at break and tensile strength. The test method and results are described as follows.

Accelerated thermal oxidation test method

15 NR blocks were used as test specimens, as shown in **Fig.1**. The dimension is 220×150×50mm (length×width×thickness). The specimens were accelerated aged in a Thermal Ageing Gear Oven. The test conditions are listed in **Table 1**. Three elevated temperatures, 60℃, 70℃, and 80℃ were applied in the oven. For the test under each temperature, the experiment duration were set as 5 stages, with the maximum lasting of 300 days.

Fig.2 shows the accelerated thermal oxidation test flow. When the rubber block specimen was taken out from the oven, it was sliced into pieces with a thickness of 2mm. From each slice, 4 No.3 dumbbell specimens were cut out. Then through the tensile tests on these dumbbell

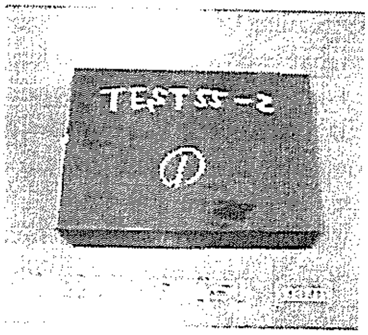
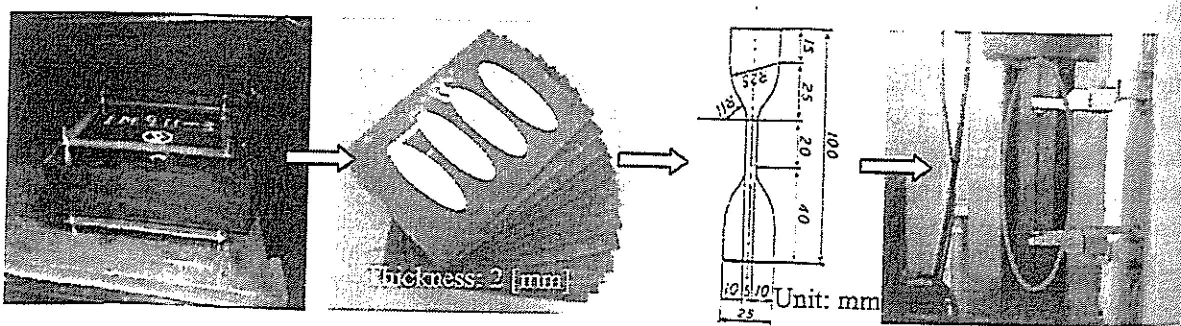


Fig. 1 NR block specimen

Table 1 Accelerated thermal oxidation test condition

Material	Temperature [℃]	Test Duration [days]
NR	60	31, 60, 100, 200, 300
	70	12, 22, 38, 75, 113
	80	4, 8, 14, 28, 42



(a) Accelerated test (b) Rubber slices (c) No.3 dumbbell specimen (d) Tensile test

Fig.2 Accelerated thermal oxidation test flow specimens, the stress-strain curves were obtained, which represented the rubber properties at the corresponding position.

In the following paper, the stress corresponding to the strain of 100% is called M100, and the distribution of mechanical properties like stiffness, elongation at break and tensile strength is called property profile.

Test results and examinations

The profiles of M100 and elongation at break in 70℃ are illustrated in **Fig.3**. The horizontal axis is the relative position with regard to the thickness of NR block. The values 0 and 1 mean the surface of the block. In **Fig.3** the vertical axis is the normalized change of property with the original value equaling one.

From **Fig.3**, it can be found that material properties on the surface change the greatest and kept changing over all the time. From the surface to the interior, due to the diffusion-limit oxidation effect, the property change decreased gradually. Beyond a certain depth, the properties do not change any more. That depth is called critical depth, which is correlated with temperature, as expressed by the following equation:

$$d^* = \alpha \exp\left(\frac{\beta}{T}\right) \quad (1)$$

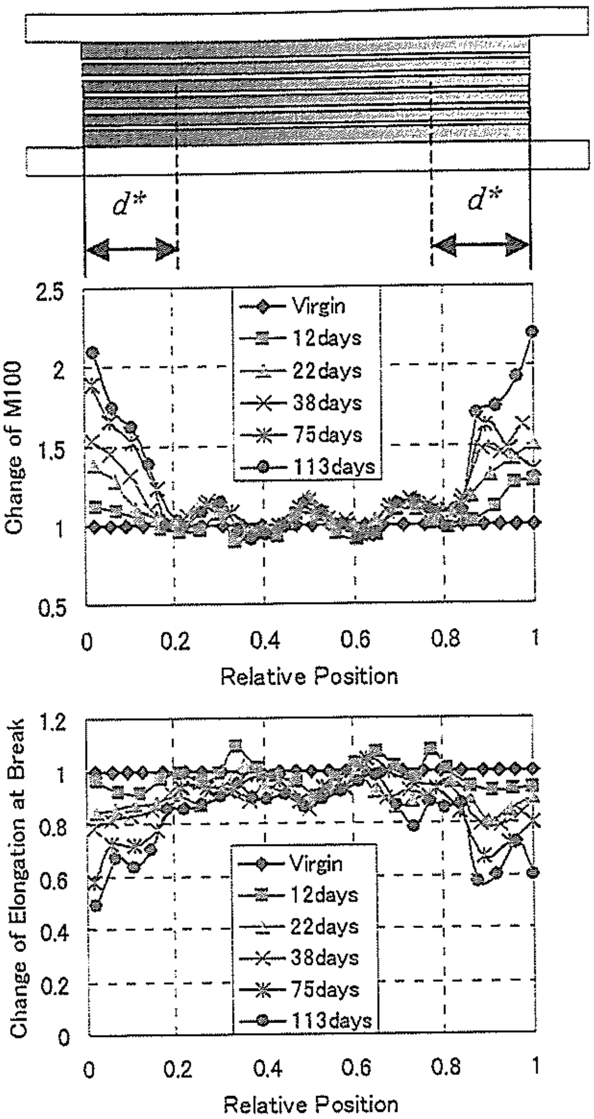


Fig.3 Property profile of NR (70℃)

where, d^* is the critical depth, T is the absolute temperature, and symbols α and β are coefficients determined by the ageing test. Here, α=8.0×10^{-4}mm, β=3.3×10^{3}K.

The property change on the surface of the rubber bearing is related to the time, and it is discovered the property change is proportional to the square root of the time, and can be formulated as:

$$\Delta U_s = k_s \cdot \sqrt{t} \quad (2)$$

where, ΔU_s is the relative property change on the bearing surface, t is time, and k_s is a factor determined by the tests.

For the region from block surface to the critical depth, the property change is assumed to be a square relation to the relative position from the surface to the critical depth, and a property profile model for NR is proposed:

$$\frac{U(t)}{U_0} = 1 + w\Delta U_s \quad (3)$$

where, U(t) and U_0 are the NR material property at time t and original state, respectively. w is the position coefficient correlated with the position x, the critical depth d^* and the thickness l of the

NR block, and can be expressed by the following equation:

$$w = \begin{cases} \left(\dfrac{x-d^*}{d^*}\right)^2 & \left(0 \le x \le d^*\right) \\ 0 & \left(d^* \le x \le l-d^*\right) \\ \left(\dfrac{x-\left(l-d^*\right)}{d^*}\right)^2 & \left(l-d^* \le x \le l\right) \end{cases} \tag{4}$$

In the thermal oxidation test the temperature is much higher than the temperature in the real environment. It is because high temperature can accelerate the deterioration[7]. The Arrhenius methodology is commonly used to correlate accelerated ageing results with ageing uder service conditions. Gerenally the thermal oxidation is assumed as the 1st chemical reaction for rubber materials[8]. Then the deterioration time in the accelerated exposure tests can be converted into the service conditions through the following formula:

$$\ln(\frac{t_r}{t}) = \frac{E_a}{R}(\frac{1}{T_r} - \frac{1}{T}) \tag{5}$$

where, 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_r indicates the absolute temperature in the service condition, and T is the absolute temperature in the thermal oxidation test. The symbols t_r and t are the actual time and test time, respectively.

PERFORMANCE OF AGED NATURAL RUBBER BEARING

FEM modeling and analytical conditions

Yoshida et al.[3] have proposed a constitutive model for rubber materials, which is composed of two parts. The first part is an elastoplastic body with a strain-dependent isotropic hardening law, which represents the energy dissipation of the material, and the second part consists of a hyperelastic body and a damage model, which expresses the evolutional direction of the stress

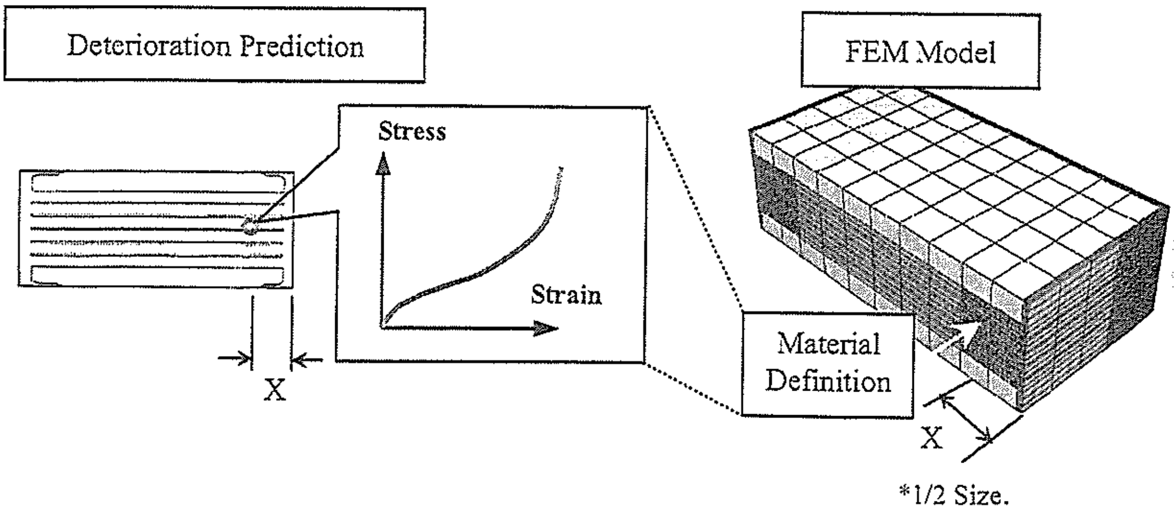


Fig. 4 Construction of FEM model of HDR bridge bearing

Table 2 Parameters of FEM model

Bearing size [mm]	600×600
Thickness of rubber layer [mm]	19
Thickness of steel plate [mm]	4.5
Thickness of coating rubber [mm]	10.0
Number of rubber layers	5
Number of steel plates	4
The 1st shape factor	7.89
Number of Nodes	10,650
Number of Elements	4,080

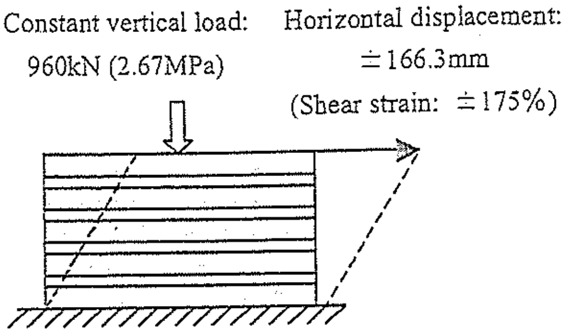
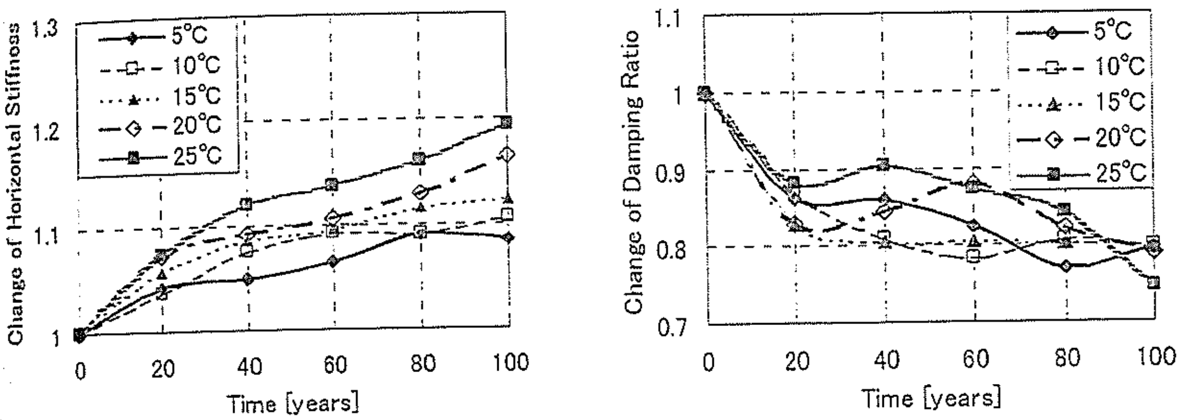


Fig. 5 Loading condition of FEM analysis



(a) Eqv. horizontal stiffness (b) Eqv. damping ratio

Fig. 6 Time-dependency of NR bridge bearing' performance (600×600)

tensor. Using the Yoshida model, a FEM model of NR bridge bearing is constructed. The modeling procedure is shown in **Fig.4**. Firstly, using the deterioration prediction model, the property profile of aged NR bridge bearing can be obtained. Then, the stress-strain relationship at any position will be known. Combined with Yoshida model, the material properties are defined to the elements at the corresponding position in the FEM model. Because the material properties only vary from the surface to the critical depth, this region is meshed finely. In Japan, the yearly average temperature varies from 5.4℃ to 24℃. According to **Eq.(1)**, it is known the critical depth will be larger in lower temperatures. Thus the maximum critical depth in Japan is nearly 120mm. So the periphery with a thickness of 120mm is averagely divided into 10 layers.

The parameters of the FEM model are listed in **Table 2**. In order to obtain the equivalent horizontal stiffness of the NR bridge bearing, the FEM model is loaded in simulation following the conditions shown in **Fig.5**. The loading conditions conform to the *Manual of Highway Bridge Bearing* (JRA). A constant vertical force is loaded to the upper plane of the bearing while keeping the upper and lower planes horizontal. The nodal displacements are constrained to ± 166.3mm, which corresponds to ±175% of shear strain.

FEM analysis results

From the hysteretic loop produced by FEM analysis, the equivalent horizontal stiffness and the equivalent damping ratio can be obtained, as shown in **Fig.6**. This figure illustrates the time-dependency of the equivalent horizontal stiffness and the equivalent damping ratio of a 600 ×600 NR bridge bearing. The values are normalized by taking the properties of the NR bearing in virgin state as one. From **Figs.6(a)**, it can be seen that, the equivalent horizontal stiffness increases over the time. After 100 years, the equivalent horizontal stiffness increases by about 8% under 5℃, and by about 20% under 25℃. From **Figs.6(b)**, it is found after 100years, the equivalent damping ratio decreases by about 20%.

SEISMIC ANALYSIS ON BASE-ISOLATED STEEL PIERS

Required performance

The current *Design Specification of Highway Bridges* of JRA suggests that dynamic analysis should be carried out to check that maximum displacement and residual displacement do not exceed structural capacity in order to secure that highway bridges can resume normal function as quickly as possible after a major earthquake. Usually the performance of bridge rubber bearing is thought stable and the effect of its long-term performance on the seismic response of the steel pier is not considered. However, according **Eq.(5)**, it is known that 113 days in the thermal oxidation of 70℃ test corresponds to 60 years under the temperature of 25℃. And from Fig.6, we can find that under 25℃ the equivalent stiffness of a 600×600 NR bridge bearing will increase by about 15% after 60 years. It is needed to check whether the structural capacity can still satisfy the requirement when the stiffness of rubber bearing increases.

As required by the *Design Specification of Highway Bridges* of JRA, not only the maximum displacement response but also the residual displacement response should not exceed the allowable value. In this study, allowable maximum displacement is chosen as δ_{95}, which is the displacement value corresponding to the cyclic strength dropping to 95% of the peak cyclic strength, and it can be calculated using **Eq.(6)**. As for the allowable residual displacement, the Committee of New Technology for Steel Structure, JSCE (1996), proposed the classification of damage degree, as shown in **Table 3**. The damage degree is divided into five ranks with respect to the residual displacement δ_R and the corresponding time needed for repair. The residual displacement can be achieved from the maximum displacement δ_{max} through **Eq.(7)**.

$$\frac{\delta_{95}}{\delta_y}=\frac{0.0147}{\left\{(1+P/P_y)R_f\sqrt{\bar{\lambda}}\right\}^{3.5}}+4.20 \tag{6}$$

$$\frac{\delta_R}{h}=\frac{1}{200}\left(\frac{\delta_{max}}{\delta_y}\right)^{0.75}-\frac{3}{400} \tag{7}$$

where, δ_y=yield displacement, P/P_y=axial load ratio, R_f=width-thickness ratio parameter, $\bar{\lambda}$ =slenderness ratio parameter, h=pier height.

Table 3 Ranking of damage degree

Rank	Residual Displacement	Damage Degree
A_s (collapse)	$h/100\leq\delta_R$	Collapsed
A (Large Damage)	$h/150\leq\delta_R<h/100$	Not collapsed, but have lost function. More than two months are required for restoring.
B (Medium Damage)	$h/300\leq\delta_R<h/150$	Only emergency vehicles can run. Two weeks~ two months are required for restoring.
C (Small Damage)	$h/1000\leq\delta_R<h/300$	Several days are required for restoring, or ordinary vehicles can pass while being repaired.
D (No Damage)	$\delta_R<h/1000$	Almost no damage.

Note: δ_R — residual displacement; h — pier height.

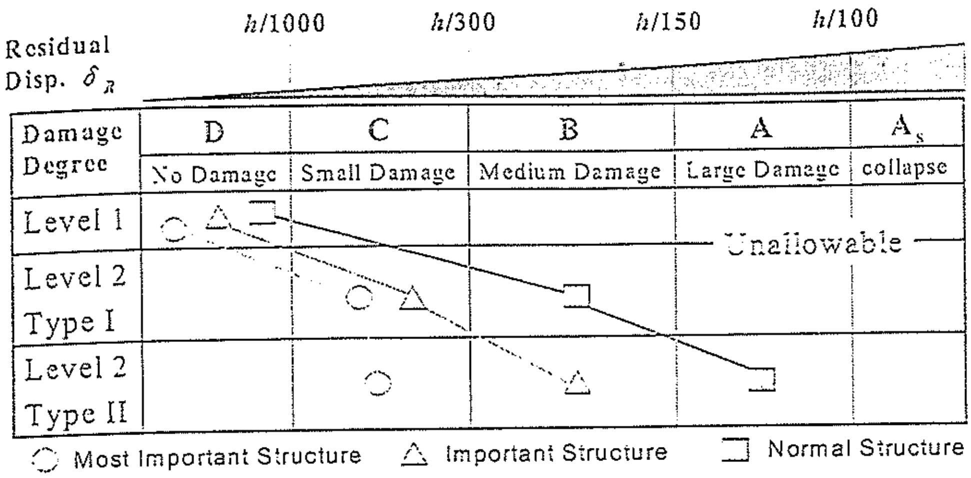


Fig.7 Required performance matrix for steel structures

According to the ranking method of damage degree shown in **Table 3**, and considering the importance of the structure, earthquake level and type, the required performance can be illustrated as **Fig.7**.

Analysis model and earthquakes

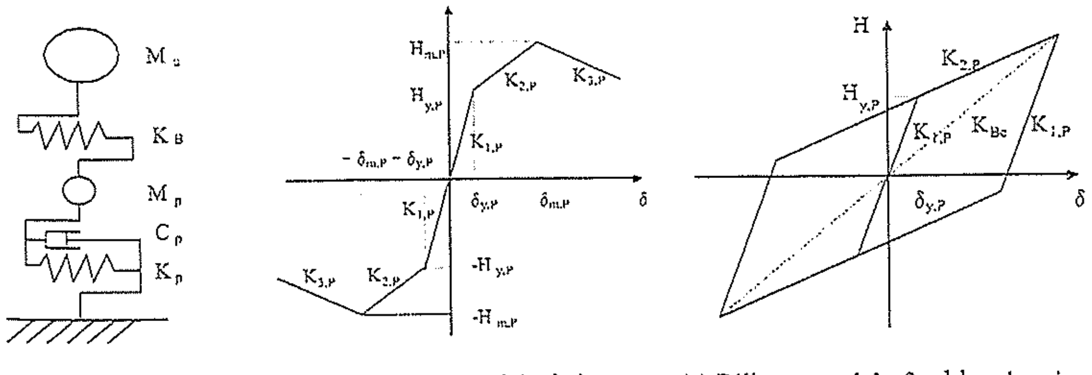
In this research, isolated thin-walled stiffened box piers are studied and a two-degree-of-freedom model is adopted as shown in **Figs.8(a)**. The mass of the superstructure is M_u and the mass on the top of the pier M_p is taken as 30% of the mass of the whole pier. **Figs.8(b)** and **Figs.8(c)** illustrate the resilience models of the steel pier and the rubber bearing, respectively. The linear acceleration method is used to resolve the elastoplastic seismic response problem. According to the *Design Specification of Highway Bridges* of JRA, the damping factor of the steel pier is ξ_P=0.01 and the damping factor of the rubber bearing is ignored. The time interval is set as Δt=0.001s.

As for the steel pier, SM490 steel is used. The structural parameter of the steel pier R_f is the width-thickness ratio parameter, and $\bar{\lambda}$ is the slenderness ratio parameter, which are defined by **Eq.(7)**.

$$R_f = \frac{b}{t}\sqrt{\frac{\sigma_y}{E}\frac{12(1-\nu^2)}{4\pi^2 n^2}}, \quad \bar{\lambda} = \frac{2h}{r}\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}} \tag{7}$$

where, δ_y=yield displacement, E=Young's modulus, υ =Poisson's ratio, σ_y=yield stress, t=thickness, b=width of flange, n=number of sub-panels, r=radius of gyration of cross-section.

In dynamic analysis the JRA code prescribes three Type I acceleraograms and three Type



(a) Base-isolated pier model (b) 2-Parameter model of pier (c) Bilinear model of rubber bearing

Fig.8 Analysis model of base-isolated steel pier with natural rubber bearing

Table 4 Parameters of steel pier

Steel type		SM490
Young's modulus	E	205Gpa (21,000kgf/mm^2)
Yield stress	σ_y	315MPa (24kgf/ mm^2)
Poisson's ratio	ν	0.3
Steel plate thickness	t	20mm
Sub-plate thickness	t_s	20mm
Sub-plate width	b_s	178mm (R_f=0.35), 189mm (R_f=0.45)
Width-thickness ratio parameter	R_f	0.35, 0.45
Slenderness ratio parameter	$\bar{\lambda}$	0.2, 0.3, 0.4, 0.5, 0.6, 0.7
Stiffener number on flange		2 (Sub-panel number=3)
Stiffener number on web		2 (Sub-panel number=3)
Flexural rigidity	γ	≥3 γ* (γ *=Optimum flexural rigidity)
Diaphragm space	l_d	b (b=section width)

II accelerograms for each ground type (I , II and III), and suggests the average responses of the three accelerograms in each group be taken as the final analysis results. Steel piers with R_f=0.35and 0.45, $\bar{\lambda}$=0.20~0.70 are analyzed. The parameters of the analyzed steel piers are presented in **Table 4**.

Analysis procedure

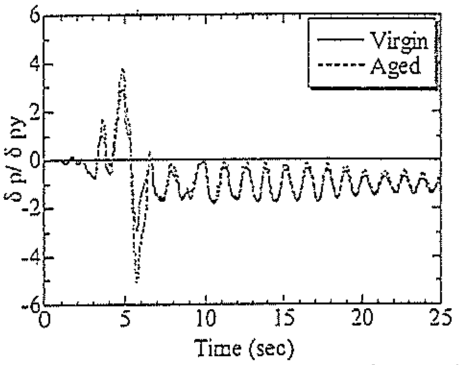
For each pier, rubber bearing is designed with different target natural periods and the response of the steel pier is investigated. Then take the cases satisfying the performance requirement into consideration. It is assumed that when aged the equivalent stiffness K_{Be} of rubber bearing increases while the most agreeable yielding load $H_{d,B}$ invariable. So the primary stiffness $K_{1,B}$ and the secondary stiffness $K_{2,B}$ will change correspondingly. The seismic responses are compared with the cases when rubber bearings are in the virgin state.

Seismic analysis results

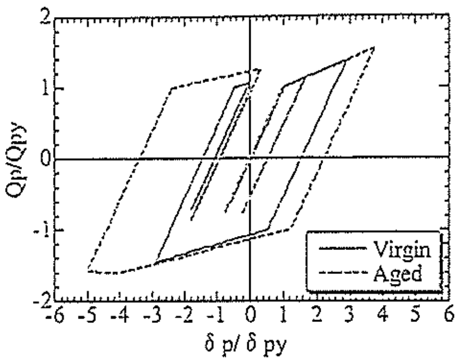
Here an example is presented to explain the analysis results. The isolated steel pier with $\bar{\lambda}$ =0.35, R_f=0.40 is analyzed using the Fukiai-M earthquake, which belongs to Level 2, Type II, and the ground type is II. The target natural period T of the isolated pier is set as 2.0 second. It is assumed after deterioration, the equivalent horizontal stiffness of the natural rubber bearing increases by 20%. The seismic responses before and after deterioration are compared in **Fig.9** as well as **Table 5**.

From **Figs.9(a)** and **Figs.9(b)** it can be seen that after deterioration, the maximum displacement of the pier will increase 72.5%. From the maximum displacement, the residual displacement can be calculated through **Eq.(7)**, as shown in **Table 5**. It should be noticed that the residual displacement increases by 153.3%. **Figs.9(c)** shows that the maximum displacement of superstructure does not change much after deterioration, only increases about 2.3%. As shown in **Figs.9(d)**, after deterioration the equivalent stiffness of the rubber bearing increases 20%, which cause the natural period of the isolated pier to decrease from 2.0s to 1.85s, so that the energy absorbed by the isolated pier will increase. On the other hand, because the energy absorbed by the rubber bearing does not change much, the energy absorbed by the pier will increase, which consequently causes the maximum displacement of the pier to increase.

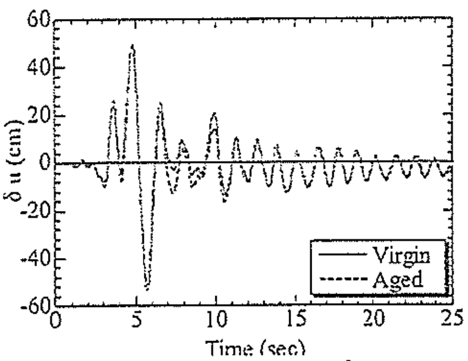
Similarly, the behaviors of other piers are illustrated in **Fig.10** and **Fig.11**. The natural period of the isolated steel pier is extended to 2s and the equivalent horizontal stiffness of aged natural rubber bearing is assumed to increase by 20%. **Fig.10** shows the seismic response of the isolated steel piers with earthquakes belonging to Level 2, Type I inputted. The vertical axis is the residual displacement versus the pier height in 100. The horizontal axis is the slenderness



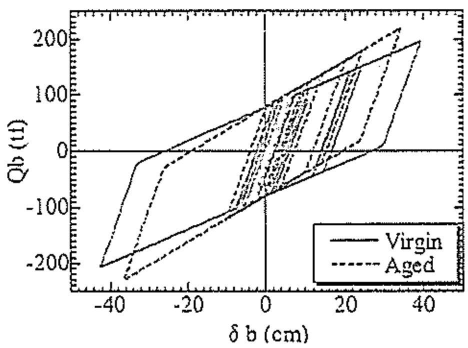
(a) Displacement response of steel pier



(b) Hysteresis curve of pier



(c) Displacement response of superstructure

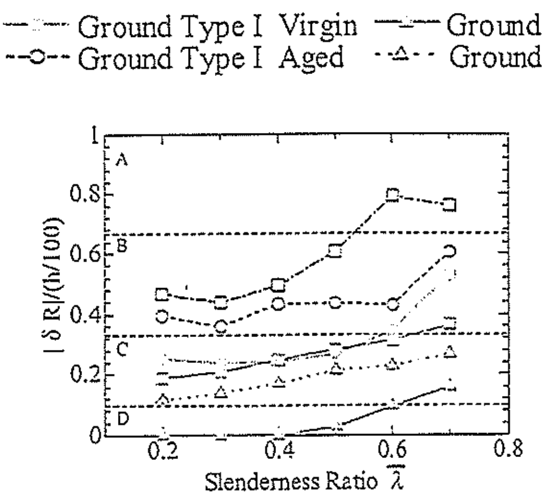


(d) Hysteresis curve of bridge bearing

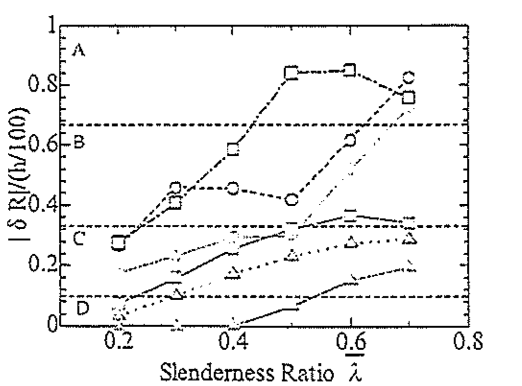
Fig.9 Analysis example using Fukiai-M (R_f=0.35, $\overline{\lambda}$=0.40, T=2.0s)

Table 5 Results of analysis example

Pier Response	Before deterioration	After deterioration	Change
Maximum Disp. [cm]	10.2	17.6	+72.5%
Residual Disp. [cm]	2.2	5.7	+155.3%

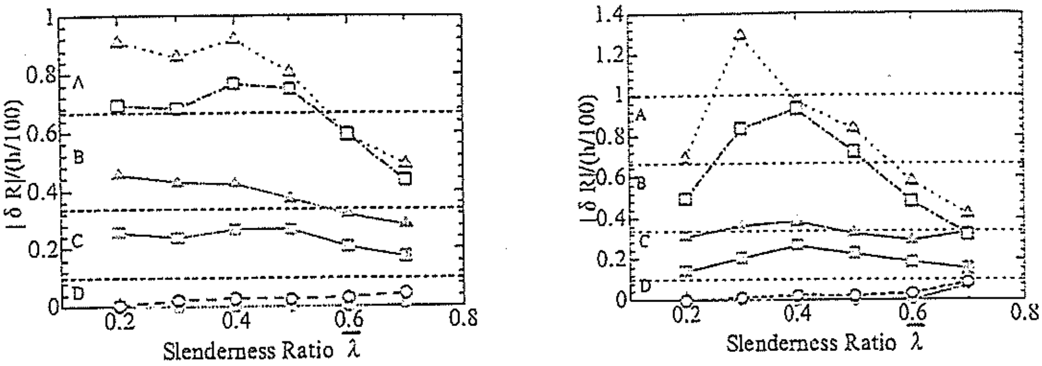


(a) R_f=0.35



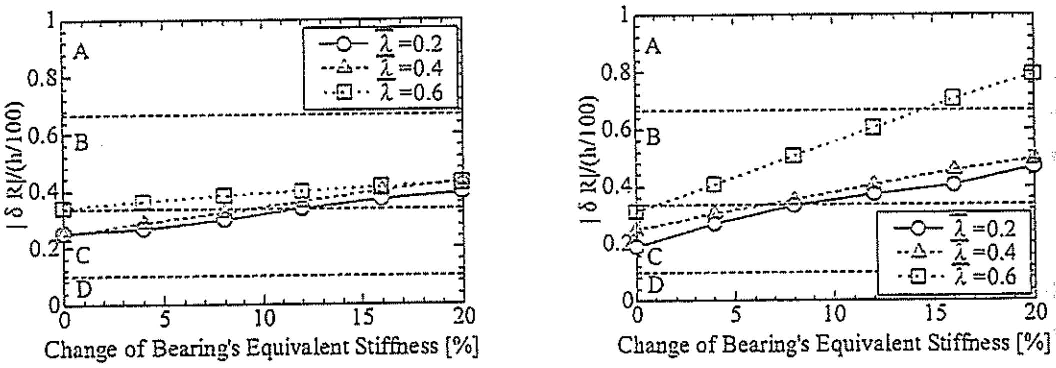
(b) R_f=0.45

Fig.10 Seismic Response of Base-Isolated Single Steel Piers (Level 2, Type I)

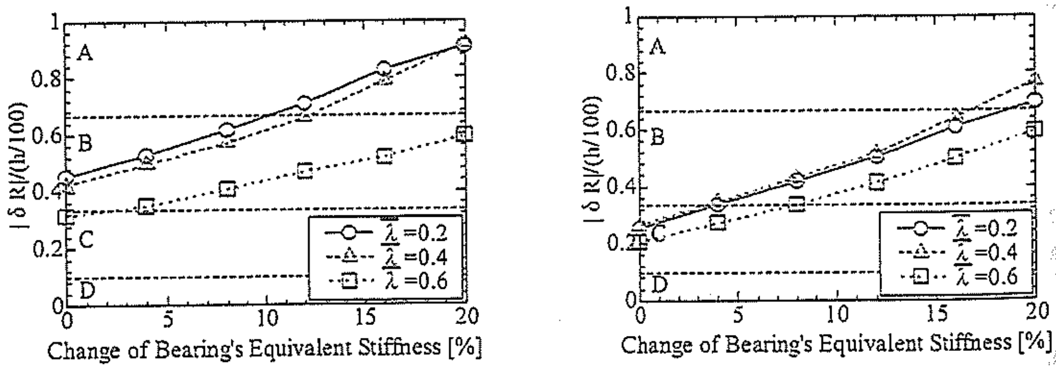


(a) R_f=0.35 (b) R_f=0.45

Fig.11 Seismic Response of Base-Isolated Single Steel Piers (Level 2, Type II)



(a) Type I, Ground Type I (b) Type I, Ground Type III



(c) Type II, Ground Type II (d) Type II, Ground Type III

Fig.12 Relations between residual displacement and rubber bearing stiffness (R_f=0.35)

ratio parameter $\bar{\lambda}$. It is discovered generally the residual displacement increases with parameter $\bar{\lambda}$. In most cases, the damage degree will upgrade to a higher level when the equivalent horizontal stiffness of natural rubber bearing increases 20%. In **Figs.10(a)**, for R_f=0.35, Ground Type I and III, $\bar{\lambda}$ between 0.2 and 0.6, the damage degree changes from Rank C to B or A. According to **Fig.7**, it is not allowable for important and the most important structure to exceed Rank C. Therefore, the natural rubber bearing cannot fulfill its task in these cases. For Ground Type II, the response is in the safe range even after deterioration. In **Figs.10(b)**, for R_f=0.45, the similar behaviors are found. It is not safe for Ground Type I and III, $\bar{\lambda}$ between 0.3 and 0.5 if natural rubber bearing is deteriorated.

In **Fig.11**, the cases with the earthquakes belonging to Level 2, Type II are analyzed. For

Ground Type I, the residual displacement is very small no matter before or after deterioration. However, for Ground Type II, before deterioration, the damage degree is in Rank B with $\overline{\lambda}$ between 0.2 and 0.5, which is allowable for important structures, but after deterioration, the damage degree upgrades to Level A. For Ground Type III, before deterioration the damage degree is in Rank C. If the bearing stiffness increases 20%, the residual displacement is found to increase by nearly 200%.

From above, it is known that for Level 2 earthquake, attention should be paid to Ground Type I and III with Type I earthquake, as well as Ground Type II and III with Type II earthquake. The relationship between residual displacement and equivalent horizontal stiffness change of natural rubber bearing in these cases are drawn in **Fig.12**. It can be seen the residual displacement increases with the equivalent horizontal stiffness of the rubber bearing. Considering the important structure, for Ground Type I with earthquake Type I, when bearing's equivalent stiffness increases by 12%, the pier with R_f=0.35 and $\overline{\lambda}$=0.2 will become unsafe. For Ground Type II with earthquake Type II, it will become unsafe when the bearing stiffness increases by 10%. From Fig.6(a), it is easy to find out after how many years the equivalent horizontal stiffness of a 600×600 NR bridge bearing under certain service conditions will increase to those critical values. Then the durability of the NR bearing can be evaluated based on the information like local temperature, bearing size, importance of the structure, location, ground type, and pier type.

CONCLUSIONS

Through thermal oxidation test, this research revealed the deterioration characteristics inside NR blocks. Based on the test results and FEM analysis, the future performance of NR bearing is estimated according to the local temperature. Dynamic analysis correlates the seismic response with the bearing's performance and provides a method to evaluate the durability of NR bearing. Major conclusions of this research are summarized as follows:

1) The deterioration characteristics inside NR block are clarified and a prediction model is established to estimate the property profile of aged NR bearing.
2) A FEM model is developed, which predicts about 20% increase of natural rubber bearing's shear stiffness after 100 years under 25℃.
3) Through dynamic analysis, it is discovered that the increase of the NR bearing's equivalent horizontal stiffness can enlarge the seismic response of the steel pier greatly, especially for earthquake Level 2, Type I with Ground Type I and III, as well as earthquake Level 2, Type II with Ground Type II and III. When bearing stiffness increases by 20%, damage degree will upgrade to a higher level in most cases.
4) The residual displacement increases with the equivalent horizontal stiffness of NR bearing. The durability of the NR bearing can be determined considering the damage degree.

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