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DAMAGE ANALYSIS OF SHINKANSEN VIADUCTS DURING THE 2011 GREAT EAST JAPAN EARTHQUAKE

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ABSTRACT: Retrofitted bridge fragility curves provide a powerful tool for assessing the effect of retrofit measures on seismic performance under a range of loading levels. Nonlinear dynamic analyses to develop the fragility curves of the steel-jacketed RC columns of the Shinkansen viaducts are performed in a Monte Carlo Simulation. The fragility curves in this study are represented by a lognormal distribution as a function of peak ground acceleration (PGA) or response acceleration. The improvement of seismic performance from retrofitting is quantified by comparing the fragility curve of the as-built viaduct with that of the retrofitted viaduct.

Key Words: 2011 Great East Japan earthquake, fragility curve, Monte Carlo Simulation, Shinkansen viaduct, steel jacketing, seismic reliability

INTRODUCTION

Bridges may be susceptible to damage during an earthquake event, particularly if they were designed without adequate seismic detailing. Reinforced concrete (RC) columns built using earlier designs often lack flexural strength, flexural ductility and/or shear strength. When these components are subjected to strong ground motions, they have the potential to exhibit brittle failure. Several recent destructive earthquakes in Japan (1995 Hyogoken-Nanbu earthquake, 2003 Sanriku-Minami earthquake, and 2004 Niigata-ken-Chuetsu earthquake) inflicted various levels of damage on the Shinkansen viaducts. The investigation of these negative consequences gave rise to serious discussions about seismic design philosophy and to extensive research activity on the retrofit of existing bridges. The seismic design methodology for new bridges was also improved. General column retrofits often include some type of encasement to improve the shear or flexural strength, flexural confinement and ductility capacity. Steel jackets are a common measure.

The Tohoku Shinkansen viaducts in eastern Japan, which were designed using specifications published in the 1970s, were damaged due to the 2003 Sanriku-Minami Earthquake because of the inadequacies of the shear design of RC components. After that earthquake, the RC bridge piers of the Tohoku Shinkansen viaducts were retrofitted with steel jacketing to prevent brittle failure from a severe earthquake and to enhance the ductility capacity. This work was finished before the 2011 Great East Japan earthquake. As reported in a JSCE damage investigation (Kawashima et al., 2011), the retrofitted Shinkansen viaducts experienced no damage, and the effectiveness of the seismic retrofit involving the application of steel jacketing to RC columns was demonstrated. However, there is a lack of understanding of the impact of these retrofits on the vulnerability of the viaduct. It is important to recognize the relationship between the damage to the Shinkansen viaduct with retrofitted RC columns

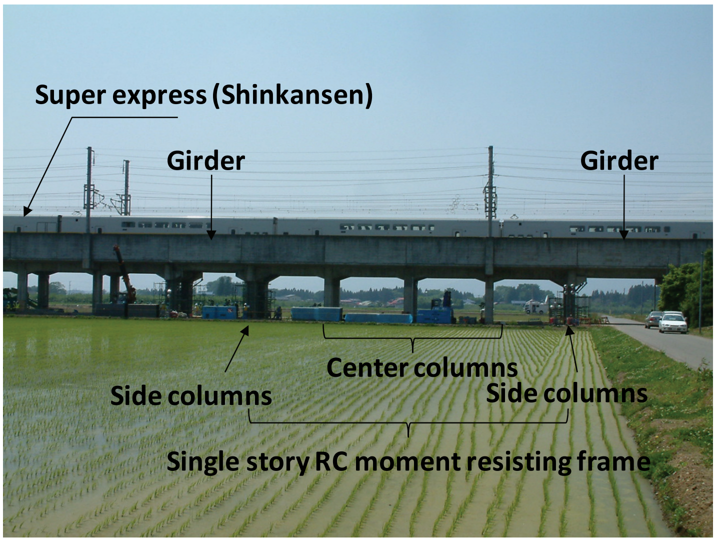

Figure 1. Single-story RC moment-resisting frame pier with a Gerber girder on both sides

and the ground motion intensity.

Seismic fragility curves are essential tools for assessing the vulnerability of viaducts. These curves describe the probability that the actual damage to a viaduct exceeds the damage criteria when the structure is subjected to a specific ground motion intensity. Fragility curves can offer a means of communicating the probability of damage over a range of potential earthquake ground motion intensities. This information is essential for seismic risk management and decision making on retrofit and mitigation strategies. Empirical fragility curves based on bridge damage data from past earthquakes have been developed for as-built bridges (e.g., Shinozuka et al., 2000). However, as a result of the limited empirical data available, developing the fragility curves for retrofitted bridges based on damage investigation is impossible. In the absence of adequate empirical data, analytical methods have been used to develop the curves (e.g., Karim and Yamazaki, 2003). Analytically derived fragility curves for retrofitted road bridges were developed by Kim and Shinozuka (2004) and Padgett and DesRoches (2007a, 2008, 2009). Demand and capacity are compared in a fragility analysis. In previous studies on analytically computed fragility curves, the demands were evaluated by a nonlinear dynamic analysis of the retrofitted bridge, and the capacities of each component were defined based on the results from past experiments and the results of an expert opinion survey (Padgett and DesRoches 2007b). In their fragility analysis, Padgett and DesRoches (2008) used the curvature ductility capacities of steel-jacketed columns with median values of 9.35 for slight damage, 17.7 for moderate damage, 26.1 for extensive damage and 30.2 for complete damage. However, these values could not be applied to develop the fragility curves of the Shinkansen viaducts in Japan with steel-jacketed RC columns.

In this paper, the moment-curvature relations of RC columns with steel jacketing based on experimental results are used. The limit states for the fragility analysis of the as-built and retrofitted viaducts are defined. The improvement due to steel jacketing is quantified by comparing the fragility curves of the viaduct before and after retrofit.

DAMAGE INVESTIGATION OF THE SHINKANSEN VIADUCTS WITH RETROFITTED RC BRIDGE PIERS SUBJECTED TO THE 2011 GREAT EAST JAPAN EARTHQUAKE

Seismic Retrofit Program of the Shinkansen Viaducts

Tohoku Shinkansen entered service in 1982 between Omiya and Morioka Stations. Because

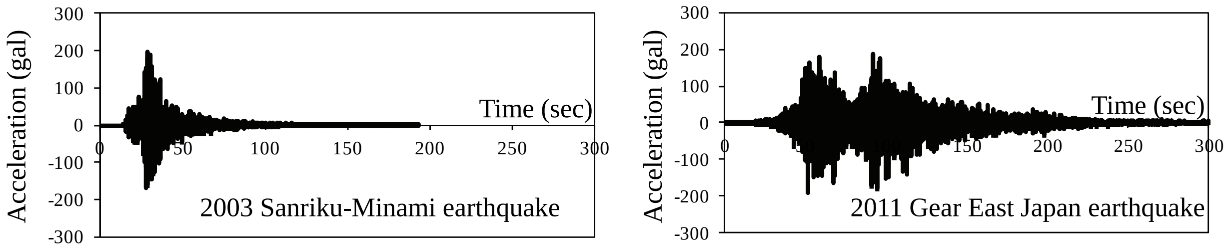

Figure 2. Comparison of the acceleration time history measured near the No. 5 Inohana viaducts during the 2003 Sanriku-Minami earthquake and the 2011 Great East Japan earthquake

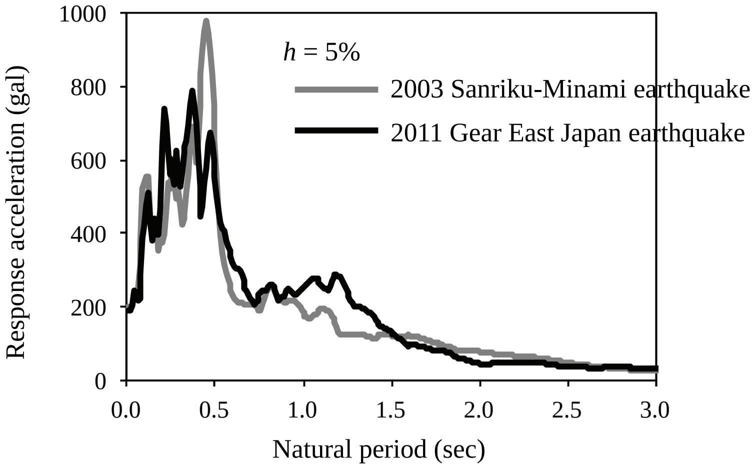

Figure 3. Response accelerations during the 2003 Sanriku Minami earthquake and the 2011 Great East Japan earthquake measured near the No. 5 Inohana viaduct

the Shinkansen viaducts were designed prior to the occurrence of the 1978 Miyagi-ken-oki earthquake, they have less shear reinforcement than is required by current code. Some viaducts of the Tohoku Shinkansen between the Morioka and Mizusawa-Esashi stations in Iwate-ken were extensively damaged during the 2003 Sanriku-Minami earthquake (JSCE, 2004).

It should be noted that all the viaducts that suffered damage during the 2011 Great East Japan earthquake had not yet been retrofitted. Most viaducts in Iwate-ken were single-story RC moment-resisting frames with a Gerber girder on both sides, as shown in Figure 1. Damage was concentrated at the side columns of the viaduct during the 2003 Sanriku-Minami earthquake. Because the side columns were made shorter than the center columns to fit under the Gerber girder, the ratio of the shear strength to the flexural strength was smaller in the side columns than in the center columns, which led to shear failure in the side columns.

After the 2004 Niigata-ken Chuetsu earthquake, the first seismic retrofit program was initiated for the Shinkansen viaducts, including the Tohoku Shinkansen. The objectives of the program were to enhance the seismic performance of the RC columns, which had insufficient shear strength. The program was completed in 2007 after retrofitting 12,500 columns. In 2009, the second retrofit program for enhancing the flexural strength and ductile capacity of the RC columns was initiated.

The No. 5 Inohana Viaducts

Figures 2 and 3 compare the acceleration time history and 5% damping response accelerations of the 2003 Sanriku-Minami earthquake with those of the 2011 Great East Japan earthquake. Both ground motions were measured at the same site near the No. 5 Inohana viaducts. As shown in Figure 2, the duration of the ground motion was very different between the 2003 Sanriku-Minami earthquake and the 2011 Great East Japan earthquake. However, the peak ground accelerations (PGAs) were almost

Figure 4. R13 to R15 of the No. 5 Inohana viaduct taken after the 2003 Sanriku-Minami earthquake and the 2011 Great East Japan earthquake
(Note: the viaducts were retrofitted before the 2011 Great East Japan earthquake)

the same. In addition, as shown in Figure 3, because the fundamental natural period of a single-story RC rigid frame ranges between 0.4 s and 0.6 s, it is reasonable to assume that the response accelerations of the No. 5 Inohana viaducts were nearly the same between the 2003 Sanriku-Minami earthquake and the 2011 Great East Japan earthquake.

Figure 4 shows the damaged states of the No. 5 Inohana viaducts taken after the 2003 Sanriku-Minami earthquake and the 2011 Great East Japan earthquake, respectively. Because of deficiencies in the number of ties used to prevent brittle failure, the RC columns of the No. 5 Inohana viaduct failed via shearing during the 2003 Sanriku-Minami earthquake. Under the first retrofit program described above, the viaducts were retrofitted by means of steel jackets for the RC columns so that they had sufficient shear capacity. Steel jackets with a thickness of 6 mm were used, and the gap between the as-built RC column and the steel jacket was approximately 30 mm. The gap was filled with a low-strength and highly liquid material. For road bridges in Japan, a steel jacket is used to increase the shear and flexural strengths and the ductile capacity (Kawashima, 2000). To increase the flexural strength, anchor bolts were provided to connect the bottom of the steel jacket to the footing. However, under the first program for the Shinkansen viaducts, because the steel jacket was only used to increase the shear strength and ductile capacity, it was not anchored to other components.

The retrofitted columns of the viaducts performed well, with almost no damage during the 2011 Great East Japan earthquake. The effectiveness of seismic retrofitting using steel jacketing to prevent significant damage to the Shinkansen viaducts was demonstrated.

FRAGILITY ANALYSIS OF THE SHINKANSEN VIADUCTS

Basic Equations for Developing Fragility Curves for As-Built and Retrofitted Viaducts

Fragility is defined as the conditional probability of the limit state $g_i \leq 0$ (representing the seismic

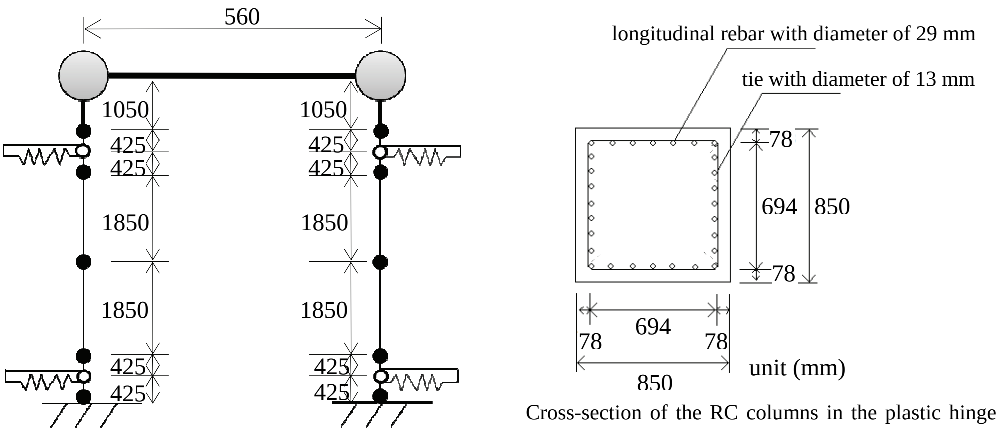

Figure 5. Analytical modeling for a single-story RC moment-resisting frame pier

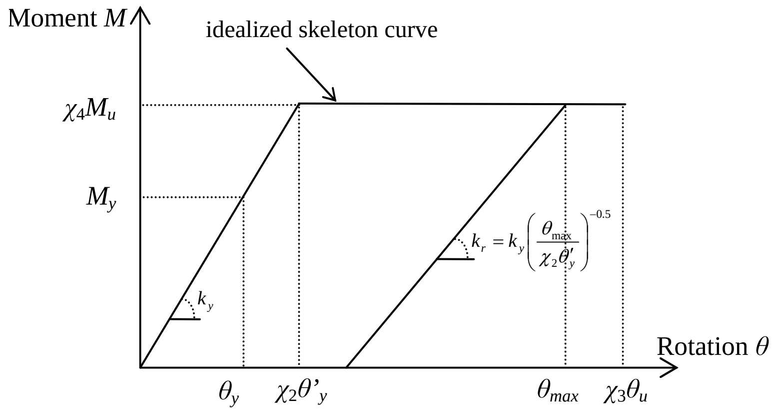

Figure 6. Moment and rotation relationship of a plastic hinge for an RC column with a steel jacket

demand placed on the structure exceeding its capacity for a given level of seismic intensity Γ, such as PGA and response acceleration). This can be expressed as follows:

$$Fragility = P[g_i \leq 0 \mid \Gamma = \gamma] \tag{1}$$

In this study, the limit states used in Equation (1) are determined for the as-built viaduct, as shown in the following equations:

$$g_1 = \chi_1 (V_c + V_s) - V_{D,b} \tag{2}$$

$$g_2 = \chi_2 \theta'_{y,b} - \theta_{D,b} \tag{3}$$

$$g_3 = \chi_3 \theta_{u,b} - \theta_{D,b} \tag{4}$$

where V_c and V_s are the shear strengths contributed by the concrete and the shear reinforcement, respectively; $V_{D,b}$ and $\theta_{D,b}$ are the peak shear forces of the RC column of the as-built viaduct and the rotation demand of the plastic hinge of the RC column of the as-built viaduct, respectively; $\theta'_{y,b}$ and $\theta_{u,b}$ are the yielding and ultimate rotations of the plastic hinge of the RC column of the as-built viaduct, respectively; and χ_1, χ_2, and χ_3 are the lognormal random variables representing the model uncertainties associated with the estimation of V_c+V_s, $\theta_{y,b}$, and $\theta_{u,b}$, respectively. The shear strength V_c+V_s in Equation (1) and the statistics of χ_1 are provided by Akiyama et al. (2010).

The Shinkansen viaducts were retrofitted by means of steel jacketing of the as-built RC columns. Because the viaduct has sufficient shear resistance after retrofitting, the limit state associated with shear failure was not considered for the retrofitted viaduct. The limit states for the retrofitted viaduct are provided by

$$g_4 = \chi_2 \theta'_{y,a} - \theta_{D,a} \tag{5}$$

$$g_5 = \chi_3 \theta_{u,a} - \theta_{D,a} \tag{6}$$

where $\theta'_{y,a}$ and $\theta_{u,a}$ are the yielding and ultimate rotation capacity of the plastic hinge of the steel-jacketed RC column, respectively, and $\theta_{D,a}$ is the rotation demand on the plastic hinge of the steel-jacketed RC column.

The fragility in Equation (1) is estimated by a Monte Carlo Simulation (MCS) and determined from the ratio of the number of times g_i (i=1, 2, 3, 4, 5) ≤ 0 (i.e., demand exceeds capacity) to the total number of MCSs. The variables $V_{D,b}$, $\theta_{D,b}$ and, $\theta_{D,a}$ in Equations (2) to (6) are calculated by a nonlinear dynamic analysis using a number of ground motions, as described later. If the fragility $P[g_i \leq 0 \mid \Gamma=\gamma]$ and seismic intensity γ relationship is modeled as a lognormal distribution based on the results of the fragility analysis using MCSs, then the fragility curve is represented by

$$F(\gamma) = \Phi\left[\frac{\ln\left(\gamma/\lambda\right)}{\varsigma}\right] \tag{7}$$

where λ and ζ are the median and log-standard deviation of the fragility curve and Φ[·] is the standard-normal distribution function. The parameters used to determine the lognormal distribution, (median λ and log-standard deviation ζ), are estimated by the method of maximum likelihood (Shinozuka et al., 2000).

The difference is negligible between $P[g_i \leq 0 \mid \Gamma=\gamma]$ (provided by MSC) and $F(\gamma)$ (in Equation (7)) at each seismic intensity γ. The number of samples was set to 1,000.

Dynamic Response Analysis of the Shinkansen Viaducts

The side columns of the single-story RC moment-resisting frame R15 of the No. 5 Inohana viaducts, as shown in Figure 4, are analyzed in this study (hereafter referred to as the “frame pier”). The seismic behavior of the frame pier is captured in the transverse direction. Figure 5 shows a two-dimensional response model to develop the fragility curves before and after the column retrofitting with a steel jacket. Within the hypothesis of a stiff soil, the bases of the frame piers are assumed to be fully fixed. The column is modeled as an elastic zone with a pair of plastic hinges at each end of the column. Each plastic hinge is modeled to consist of a nonlinear rotational spring, as shown in Figure 5. Figure 6 shows that the relationship between the moment and the rotation of the plastic hinge has a monotonic envelope, represented by the two lines, and that the stiffness beyond the yielding point is assumed to be zero. The rotation at the yielding point is provided by

$$\theta'_y = \left(M_u / M_y\right)\theta_y \tag{8}$$

where M_u is the ultimate moment of the plastic hinge of the as-built or retrofitted column and M_y and θ_y are the moment and rotation of the as-built or retrofitted column when the strain in the extreme tensile rebar reaches its yield point, respectively.

The rotation capacity $\theta_{u,a}$ is provided by Tamai et al. (1996) as follows:

$$\theta_{u,a} = \frac{4.66t}{b} + 0.024 \tag{9}$$

where t is the thickness of the steel jacket and b is the cross-sectional width of the as-built column.

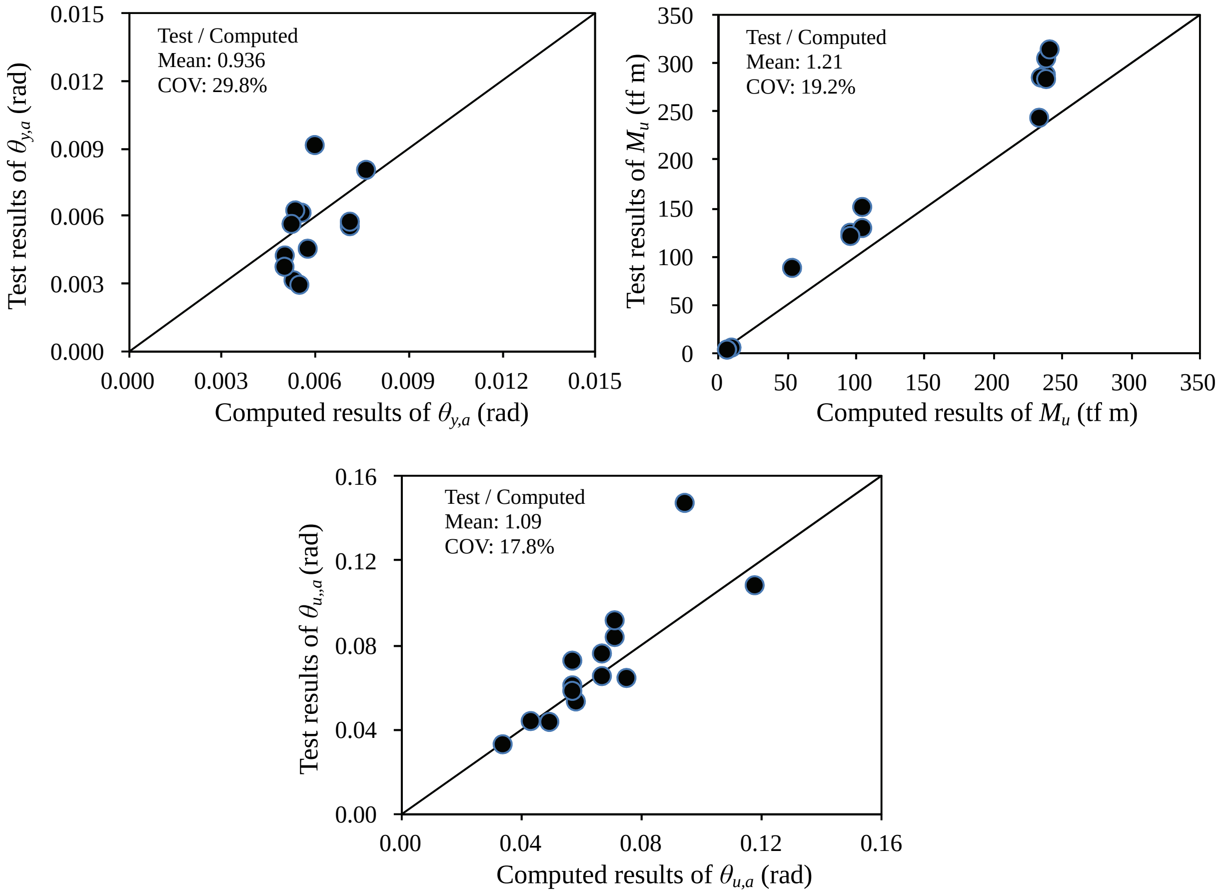

Figure 7. Comparison of the computed and experimental results from the evaluation of $\theta'_{y,a}$, $\theta_{u,a}$, and M_u

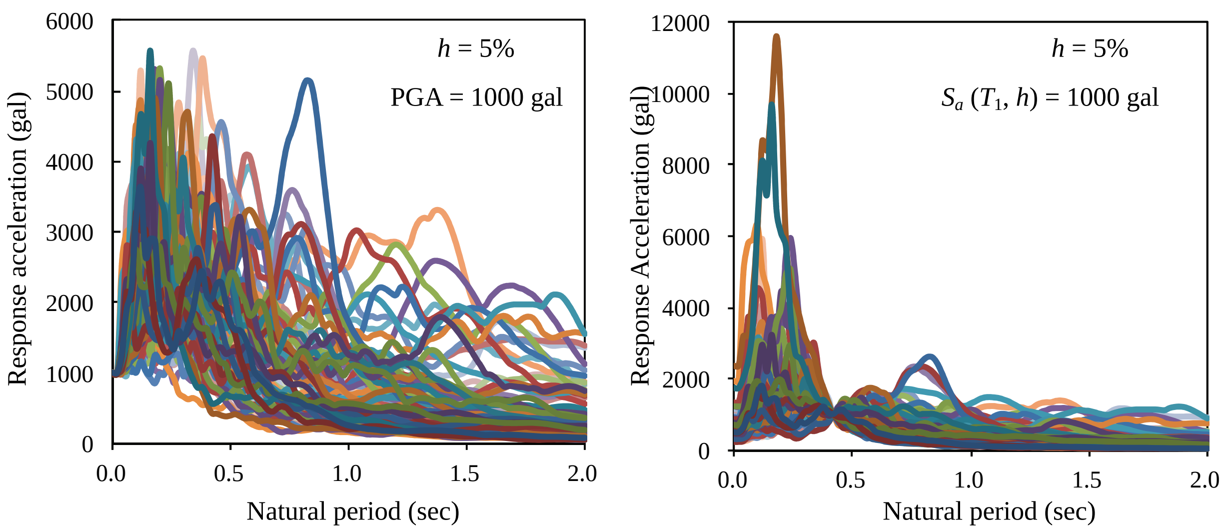

Figure 8. Response acceleration of 90 ground motions measured in Iwate-ken, Miyagi-ken, and Fukushima-ken during the 2011 Great East Japan earthquake

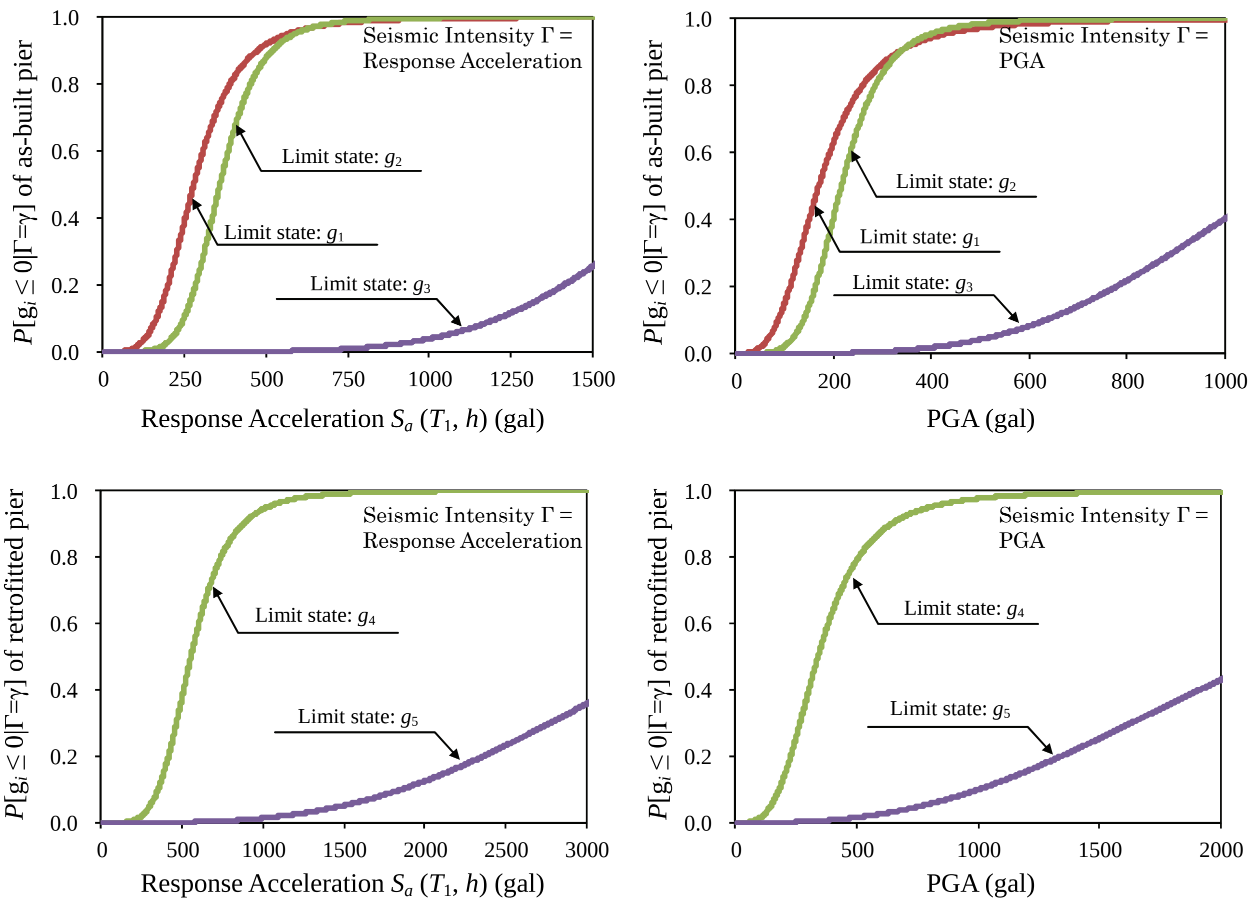

Figure 9. Fragility curves of a single-story RC moment-resisting frame pier
with and without a steel jacket

Based on a comparison of experimental results, Tamai et al. (1996) reported that the lateral force acting on a steel-jacketed RC column decreases sharply beyond the rotation capacity $\theta_{u,a}$.

Based on the experimental results of steel-jacketed columns subjected to cyclic loading, Tamai et al. (1998) reported that the hysteretic model of steel-jacketed columns is based on that proposed by Takeda et al. (1972). In this study, the unloading stiffness k_r is given by

$$k_r = k_y \left(\frac{\theta_{\max}}{\chi_2 \theta'_y} \right)^{-0.5} \tag{10}$$

where k_y is the yielding stiffness and $\theta_{\max}$ is the maximum curvature attained in the direction of loading.

The statistics of χ_2, χ_3, and χ_4 can be obtained by comparing the experimental results of steel-jacketed RC columns subjected to cyclic loading with computed values. Experimental results reported by Tamai et al. (1998) are used in this study. The number of specimens used was 12. The statistics of model uncertainties associated with the evaluation of $\theta'_{y,b}$ and $\theta_{u,b}$ are assumed to be the same as those associated with the evaluation of $\theta'_{y,a}$ and $\theta_{u,a}$. Figure 7 shows the comparison of experimental and computational results. Because the steel jacket was not anchored to the other components, the ultimate moment M_u of the retrofitted RC columns was calculated by removing the existence of the steel jacket from consideration. The steel jacket serves to increase the shear strength, which may affect the flexural strength. The experimental ultimate (=maximum) moments of steel-jacketed RC columns are underestimated, as shown in Figure 7. The statistics in Figure 7 are used as the model uncertainties. In addition, the uncertainties associated with material strength are considered when M_y, M_u, θ_y, and θ_u are calculated. The parameters associated with material strength are shown in Akiyama et al. (2010).

Ninety ground motions measured in Iwate-ken, Miyagi-ken, and Fukushima-ken during the 2011 Great East Japan earthquake were used in the nonlinear dynamic analysis performed on the frame pier. In the MCS, one horizontal component from the ground motion record is randomly selected. The seismic intensity Γ in the fragility analysis is the PGA or S_a (T_1, h), where S_a (T_1, h) is the h-damped spectra acceleration at the fundamental frequency T_1 ($\approx$ 0.43 sec) of the analyzed frame pier for the considered earthquake record. Each ground motion is amplified such that the PGA or S_a (T_1, h) is equal to a specified seismic intensity. Figure 8 shows the response acceleration of 90 ground motions that are amplified such that PGA = 1000 gal or S_a (T_1, h) = 1000 gal.

Fragility Curves of the As-Built and Retrofitted Shinkansen Viaducts

In the MCS, g_i provided by Equations (2) to (6) is calculated by using the maximum value of the peak shear forces of the RC columns and the ductility demands sustained by the plastic hinges. Figure 9 shows the fragility curves illustrating the vulnerability of the as-built and retrofitted frame pier over a range of the seismic intensities PGA or S_a (T_1, h).

Because RC columns of the as-built frame pier do not have sufficient shear reinforcement and the shear strength is less than the flexural strength, the fragility curve associated with g_1 is located to the left of that associated with g_2. From Figure 3, it might be roughly estimated that the as-built frame pier was subjected to ground motion with PGA ≈ 200 gal during the 2011 Great East Japan earthquake. If the frame pier had not been retrofitted, there is a high probability that it would have suffered severe damage due to shear.

Comparing the fragility curves associated with the limit states g_2 and g_3 for the as-built frame pier to those associated with g_4 and g_5 for the retrofitted frame pier, the fragility enhancement is found to be more significant for the severe-damage state. The physical improvement to seismic vulnerability from steel jacketing is evident when enhanced fragility curves are plotted as a function of PGA or S_a (T_1, h). The enhanced curves shift to the right relative to those associated with the frame piers before retrofit. In 2004, Kim and Shinozuka presented the fragility curve of a typical California-type multiframe concrete bridge based on a two-dimensional response analysis and an MCS. They developed the fragility curves of a bridge with steel jacketing, indicating the states of damage as none, slight, moderate, extensive, and complete collapse. It is interesting to note that the fragility curve associated with g_5 in Figure 9 is similar to the one associated with the extensive damage state presented by Kim and Shinozuka (2004), even though the bridge types and ground motions used in this and Kim and Shinozuka's studies are different.

CONCLUSIONS

The effectiveness of the seismic retrofit to the Shinkansen viaduct was demonstrated in the 2011 Great East Japan earthquake. The RC columns retrofitted by means of steel jacketing experienced no damage during this earthquake. Some of the Shinkansen viaducts were not retrofitted in the first seismic retrofit program because they were identified as having sufficient shear reinforcement to prevent brittle failure. They were to be retrofitted in the second seismic retrofit program. Some of the viaducts that were not retrofitted were severely damaged during the 2011 Great East Japan earthquake. It is important that rapid progress is made in the seismic retrofit program for the Shinkansen viaducts.

Although the viaduct with retrofitted RC columns sustained no damage during the 2011 Great East Japan earthquake, it should be noted that the ground motions at the site of the Shinkansen viaducts induced by the 2011 Great East Japan earthquake were not strong compared with the seismic design force in the current seismic design code. To evaluate the likelihood of damage of the as-built and retrofitted viaduct over a range of potential earthquake ground motion intensities, seismic fragility curves were developed. Lognormal distribution functions were derived by a nonlinear dynamic analysis and an MCS. The simulated fragility curves of the retrofitted viaduct show a great improvement in seismic performance compared with those of the as-built viaduct.

In this study, the limit states for the viaduct are defined in terms of structural capacities such as shear strength and rotation ductility. However, the limit states should be provided by considering the motion performance of Shinkansen and postevent rehabilitation. The investigation of fragility curves using these limit states is a future challenge.

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