DAMAGE TO ROAD BRIDGES INDUCED BY GROUND MOTION IN THE 2011 GREAT EAST JAPAN EARTHQUAKE

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In the 2011 Great East Japan Earthquake, many civil infrastructures were heavily damaged by the tsunami, but most structures designed with post-1990 code were not damaged by ground motion. However, unretrofitted road bridges designed with pre-1980 code were damaged and it should be noted that some bridges designed with post-1995 code were severely damaged. In this earthquake, cracks in elastomeric bearings were observed in a wide region in the east part of Japan, and rupture of elastomeric bearings occurred in some bridges. This paper summarizes the characteristics of damage to road bridges induced by ground motion, especially focusing on damage to the rubber bearings of bridges.

Key Words: road bridges, ground motion, 2011 Great East Japan Earthquake, rubber bearings

1. INTRODUCTION

The 2011 Great East Japan Earthquake (The 2011 off the Pacific coast of Tohoku Earthquake) with moment magnitude of 9.0 occurred at 14:46 (local time in Japan) on March 11, 2011 along the Japan Trough in the Pacific. It was the sixth largest earthquake ever recorded in the world. The fault zone extended 450 km and 200 km in the north-south and west-east directions, respectively. Extensive damage occurred in a wide region in the east part of Japan.

Many highway bridges were damaged in these areas due to both large ground motion and tsunami inundation. Soon after the earthquake occurred, NILIM and CAESAR in PWRI jointly investigated the bridge damage and provided technical support and suggestions to bridge administrators. The Earthquake Engineering Committee, JSCE, dispatched reconnaissance teams to investigate the damage mechanism. This paper presents the damage characteristics of road bridges induced by ground motion based on these survey teams.

2. GROUND MOTION

In the 2011 Great East Japan Earthquake, maximum seismic intensity was observed at Tsukidate, Kurihara City in Miyagi Prefecture (seismic intensity of JMA was 7) and large seismic intensities were observed in the Tohoku and Kanto areas. Fig.1 shows acceleration ground motion waveforms and spectral response accelerations at representative strong ground motion observation sites.

It should be noted that: 1) strong ground motion records with long duration were observed, and 2) there were multiple pulses in some ground motion records observed near the epicenter. This was because large fault areas collapsed continuously. Very large maximum response acceleration was observed at the range of short predominant period such as those in the Tsukidate record. The maximum response accelerations at the range of natural periods from 1.0 to 2.0 seconds, which relatively correlate with damage to ordinary road bridges, were equal or slightly less than those of the 1995 Hyogo-ken Nambu earthquake. Ground motions and maximum response accelerations at the coastal area of the Tohoku region were not so large. However, strong ground motions and large response accelerations
were observed at the sites located slightly far from the epicenter, such as Fukushima, Tochigi and Ibaraki Prefectures. Moreover, aftershocks with JMA magnitude of 7.0 or higher occurred three times within a day and a total of 89 aftershocks with magnitude of 6.0 or higher occurred until August 3.
3. CHARACTERISTICS OF DAMAGE TO ROAD BRIDGES INDUCED BY GROUND MOTION

(1) Overview
Damage to highway bridges due to this earthquake can be categorized as the effects of strong ground motion, tsunami inundation, and soil liquefaction. It should be noted that in this earthquake, the intensive damage in highway bridges was mainly caused by tsunami inundation. The ground motion effect on damage to bridges was less significant than that of the tsunami effect. One bridge (Rokko Ohashi Bridge, an old steel girder bridge supported by steel pile-bent columns located in Ibaraki Prefecture, Fig. 2) collapsed due to the ground motion of the earthquake. Although only the Rokko Ohashi collapsed among the highway bridges, damage to bridges designed in accordance with pre-1980 design specifications were found as follows: damage to RC columns at section of cut-off of longitudinal rebars, damage to RC pier-wall with small amount of reinforcement, damage to steel bearings and attachment of bearings, damage to bracing and steel members, and subsidence of backfill soil of abutment. These damage modes had been observed in the past earthquakes. However, rupture of elastomeric bearings were observed at the Sendai-Tobu Viaduct designed based on post-1995 design specifications.

After the Kobe Earthquake, seismic retrofitting were performed on existing bridge columns designed in accordance with pre-1980 specifications with high priority, to prevent the collapse of the bridge structure and unseating of the deck. Most retrofitted bridge columns were not damaged due to the ground motion of the earthquake, which shows the effectiveness of the seismic retrofit.

Soil liquefaction was widely observed particularly in the Tokyo Bay area. Although the effect of the soil liquefaction on the bridge damage was minor, subsidence of backfill soil of abutment due to soil liquefaction effect developed in some bridges. Deck-end gap was shortened resulting from the movement of the substructure, which caused damage to steel bearings and cracks in parapet walls.

(2) Damage to unretrofitted bridges designed in accordance with pre-1980 design specifications
The Rokko Ohashi Bridge (Fig. 2), constructed in 1968, collapsed due to the ground motion of the earthquake. The bridge was located at Kitaura, Ibaraki Prefecture. The superstructure was made of 21—span simple steel girders (404.63 m) and the substructure was made of steel pile-bent columns. Three simple girders and two columns collapsed. Based on the survey, the remaining columns were
inclined in the transverse direction, but no severe damage was found in the lateral beams of the columns. Since the bridge was over 40-years-old, it needed to be replaced with a new bridge and the new one was being constructed at the time of the earthquake.

Intensive damage due to the ground motion developed in many unretrofitted bridges designed in accordance with pre-1980 design specifications. Most damage modes in those bridges had ever been observed in the past earthquakes. **Figs. 3 to 5** show the damage to reinforced concrete piers, steel bearing supports and the attachment of the bearing support to the pier top or the superstructure, respectively.

**Fig. 3** Damage to reinforced concrete columns at cut-off section of longitudinal reinforcement.

**Fig. 4** Damage to steel bearing support.

**Fig. 5a** Damage to pier top.

**Fig. 5b** Damage to attachment of bearing to superstructure.

(3) **Damage to bridges designed in accordance with post-1995 design specifications**

There were few intensively damaged bridges designed in accordance with post-1995 design specifications, where the seismic design acceleration increased based on the ground motion records of the 1995 Kobe earthquake and the details of the transverse steel were specified to improve the confine-
ment effect and shear capacity of the RC columns. However, it should be noted that elastomeric bearings ruptured at some bridges designed based on post-1995 design specifications. The details are described in the next section.

(4) Damage to retrofitted bridges

Based on the lessons learned from the 1995 Kobe Earthquake, the seismic retrofit project has been performed for existing bridge columns designed in accordance with pre-1980 specifications with high priority, to prevent the collapse of the bridge structure and unseating of the deck. During the 2011 Great East Japan Earthquake, many retrofitted bridges were given a shake due to the ground motion.

Fig. 6 illustrates the effectiveness of the seismic retrofit for bridge columns. As seen in Fig. 6, there are two adjacent river-crossing bridges. Since one bridge (Nakagawa Bridge) is on the designated emergency route, bridge columns designed with pre-1980 specifications have already been retrofitted by reinforced concrete jacketing. The other bridge (Kunita Ohashi Bridge) is on the local roadway and the bridge columns had not yet been retrofitted at the time of the earthquake. Although Kunita Ohashi Bridge suffered vulnerable damage and thus lost its serviceability, Nakagawa Bridge did not suffer from the damage and continued to be serviceable soon after the earthquake. Seismic performance shown by these two bridges clearly exhibits the effectiveness of the seismic retrofit.

On the other hand, there are a few remarkable damage examples in the retrofitted bridges. Fig. 7 shows two adjacent reinforced concrete columns in Kameda Ohashi Bridge. Each column supports a 2-span continuous steel box girder at the middle. The outbound column was designed with 1980 specifications and retrofitted by reinforced concrete jacketing to strengthen the cut-off section without increasing the flexural strength of the column base. Furthermore, additional shear keys were anchored to the column top to supplement the strength of the existing bearings. Although no damage was found to the steel bearings and shear keys, vertical cracks of nearly 10mm width were observed at the beam section as shown in Fig. 7. The inbound column was designed with 1994 specifications basically and some modifications were made based on the 1995 tentative specifications (published soon after the 1995 Kobe Earthquake). In the inbound column, elastomeric bearings were deformed in the transverse direction and the side stoppers failed. Concrete of the beam edge portion attaching the supplemental shear keys also spalled off (see Fig. 7) due to the transverse seismic force induced by the
inertia of the superstructure, while the vertical crack observed in the beam of the outbound column did not develop in the inbound beam.

**Fig. 8** shows the failure of the pier top. This type of damage has never been observed in the old bridge columns since the past earthquakes. The bearing capacity of the pier top was insufficient to transmit the seismic force to the column. It should be noted that in this bridge, the crack reaches the portion of the anchor bars of the steel bracket that attaches the unseating prevention devices. Thus, the unseating prevention devices should work during the unexpected failure of the support bearings.

4. BRIDGES WITH DAMAGED RUBBER BEARINGS

(1) JARA seismic design specifications revised after 1995

The 1995 Kobe Earthquake caused destructive damage to highway bridges, including the collapse of superstructures at 25 sites. Based on the lessons learned from the earthquake, the Japan Road Association’s (JARA) Specifications for Highway Bridges, Part V: Seismic Design was significantly revised in 1996. The intensive earthquake motion with a short distance from the inland earthquakes with Magnitude 7 classification has been considered in the design. As the design philosophy for large earthquakes, the JARA code does not allow critical damage to standard bridges but allows limited damage to important bridges. The 1990 JARA code required that RC piers be designed with the ductility design method, but the 1996 JARA code expanded the scope of applicability of the ductility design method to include not only RC piers but also steel piers and other structural elements such as foundations, unseating prevention systems and bearings that are seriously affected by earthquakes.

Elastomeric bearings and seismic isolators are installed to improve the seismic performance of bridges. In the past earthquakes, rupture of only one rubber bearing was reported in the 1995 Kobe
Earthquake\textsuperscript{41}, but after the earthquake there was no report of significant damage to rubber bearings and their high deformation performance protected the function of bridges even if abutments moved. However, in the Great East Japan Earthquake, damage to rubber bearings in many bridges was reported.

(2) Tobu Viaduct (Sendai Tobu Toll Road)

Tobu viaduct (Fig. 9) is located between Sendai Higashi IC and Sendai Port Kita IC in Sendai Tobu Tall Road (opened in 2001). However, in Fig. 9, it should be noted that elastomeric bearings were designed based on post-1995 design specifications. Fig. 10 illustrates the structure of the Sendai-Tobu Viaduct and the positions of the elastomeric bearing with rupture. As seen in Fig. 10, the bridge structure in this junction is very complicated. The bridge width changes significantly in the 4-span continuous box girder with the change in the section. The span length of the section from P52 to P56 is around 70m, while that from P56 to P58 is 39m. P52 and P53 are single-column hammerhead steel piers, while P54, P55 and P56 are two-column steel frames. Rupture of the elastomeric bearings in the transverse direction was observed at the deck-end of the P52 side in the 4-span continuous box girder, though there are few damaged elastomeric bearings at the other deck-end side of the box girder on P56. It is pointed out that one of the possible reasons for the rupture of elastomeric bearings is that the interaction between adjacent bridges with different natural periods was not properly considered in the design of elastomeric bearings\textsuperscript{42}.

(3) Rifu Viaduct (Sendai Hokubu Toll Road)

Rifu Viaduct is located between Rifu JCT and Rifu Shirakashi IC on Sendai Hokubu Toll Road (opened in 2002). Several elastomeric bearings ruptured in the transverse direction as shown in Fig 11. Fortunately, the ruptured bearings did not cause the deck to collapse. Fig. 12 shows the location of the rupture of elastomeric bearings and the crack pattern of damaged bearings is shown in Fig. 13. The urgent safety inspection after March 11 reported that three bearings ruptured. After the aftershock of Magnitude 7.4 on April 7, urgent inspection was
conducted again and another eight bearings were found to have ruptured. Since the section area of the elastomeric bearings was larger than those of the conventional steel bearings, it could support the girders after the breakage of the bearings, but could not resist the lateral force. The big aftershock gave us another important lesson on the need to provide not only vertical support but also lateral resistance in the restoration process.

Some damaged bearings caused by environmental deterioration were repaired (Fig. 13). Experimental tests of elastomeric bearings installed in the Tobu and Rifū Viaducts showed that some of the bearings were ruptured at lower than 250% rubber strain. It is not clear whether the cracks caused by the environment initiated the rupture of the bearings or not, but it is important to evaluate the effect of the environment on the seismic performance of elastomeric bearings.

(4) Shin Nakagawa Bridge (Higashi Mito Toll Road)

Shin Nakagawa Bridge (Fig. 14) is the cable-stayed bridge located at Mito City in Ibaragi
Prefecture. The steel box girder is supported by one tower and five piers. The bridge was completed in 1999, but originally designed based on 1990 design specifications. After the 1995 Kobe Earthquake, the design of the bridge was checked by dynamic analysis based on the supplementary specifications for reconstruction of damaged bridges (1995). The four elastomeric bearings were installed on the piers and the bearings on three piers were ruptured in the longitudinal direction (Fig. 15). The bearings deformed about 45 cm and hit the prevention devices due to excessive movement.

(5) Asahi Viaduct (Hitachi By-pass of Route 6)
Asahi Viaduct (Fig. 16) is located at Hitachi City in Ibaragi Prefecture. The viaduct consists of 4-span and 7-span continuous PC box girders (Fig. 17). The bridge was designed based on 2002 design specifications and completed in 2008. Two and three lead-rubber bearings (seismic isolators) were installed on the piers and the abutments, respectively, and one bearing on abutment As1 was largely cracked during the earthquake (Fig. 18). Since the bridge was located along the coastline, the bearings were coated by rubber to prevent corrosion. The side stoppers of the bearings bit into the rubber coat and prevented the deformation of the bearings.

(6) Kinko JCT Viaduct (Kanagawa Route 1 Yokohama Line, Metropolitan Expressway)
Kinko JCT (Fig. 19) is located at Yokohama City in Kanagawa Prefecture. The viaduct was designed in 1970, and the superstructure was a simple steel I girder and supported by rubber pad-type bearings. The bearings were replaced in 1999. Due to the ground motion of the earthquake, one bearing moved away from the original position although the side blocks were not damaged.

5. GROUND MOTIONS AT BRIDGES WITH DAMAGED ELASTOMERIC BEARINGS

(1) Recorded data at permanent observations
Fig. 20 shows the distribution of seismograph stations around the bridges with damaged elastomeric bearings. The stations are located around the bridge sites within 5km. The response spectra are shown in Fig. 21. In the Sendai area, K-net Sendai and NEXCO Sendai Higashi are similar to the Design Spectra of Level II – Type II (Soil Type I), but the other ground motions are smaller than those of the Design Spectra especially in 0.5-5 seconds. According to these spectra, recorded ground motions do not have killer components to general bridges with elastomeric bearings.

(2) Estimated waveforms based on records of temporary aftershock observations
The ground motions strongly affected the site condition. To estimate strong ground motions at the
damaged bridges with high accuracy, temporary aftershock observations were conducted at the bridge sites\textsuperscript{10}. Effects at the bridge sites were evaluated based on observation records. Strong ground motions of the main shock in the 2011 Great East Japan Earthquake were estimated considering the empirical site amplification and phase effects.

The response spectra of the estimated waveforms are shown in Fig. 22. In the case of Tobu Viaduct, the estimated response spectrum is similar to the recorded data at K-net Sendai. In the other cases, the estimated response spectra in 0.5-2.0 seconds are much greater than the recorded data. Especially the estimate response spectrum of Asahi Viaduct is larger than the design spectra. These estimations indicate that the peak of the response spectra around 1.0 second has the potential to severely vibrate the bridges with elastomeric bearings.
Fig. 20 Location of bridges with damaged elastomeric bearings and seismograph stations.

Fig. 21 Response spectra of recorded ground motion with JARA 2002 Design Spectra.
6. CONCLUSIONS

This paper summarized the damage to road bridges induced by the ground motion in the 2011 Great East Japan Earthquake with focus on the damage to rubber bearings. Based on the damage caused by the earthquake, more analytical and experimental researches are required to clarify the mechanism of the damage. The following can be concluded:

1. Most structures designed with post-1990 code were not damaged by ground motion. Investigation results also indicate that subsidence of the backfill soil in the abutment has been remarkable with the improvement of seismic performance of bridge structures.

2. Most structures designed with post-1990 code were not damaged by ground motion. Investigation results also indicate that subsidence of the backfill soil in the abutment has been remarkable with the improvement of seismic performance of bridge structures.

3. In some bridges designed with post-1995 code, elastomeric bearings and dampers were severely damaged. Cracks in elastomeric bearings were observed in a wide region in the east part of Japan, and in the Sendai area, the rupture of bearings occurred in two highway viaducts. The damage was concentrated near the edge of girders.

4. Some damaged elastomeric bearings were repaired for environmental deterioration. It is not clear whether the cracks due to environmental action initiated the rupture of the bearings or not, but it is important to evaluate the effect of environmental action on the seismic performance of elastomeric bearings.

5. Since there were no measurement system for almost all bridges, it is very difficult to know the seismic behavior without any estimation. Some measurement systems are needed to at least record the maximum deformation and the direction in these bearings.

6. Recorded ground motion continued over 300 seconds, and the peak ground acceleration was very large. Although most recorded ground motions at JMA and K-net had relatively small power in the natural period of 0.5 – 2.0 seconds, except for K-net Sendai and NEXCO Sendai Higashi IC. But based on the temporary after-
shock observation at the damaged bridge sites, the response spectra of the estimated ground motions show the peak in the period of 1.0 second and they have the potential to severely vibrate the bridges with elastomeric bearings.

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