DAMAGE OF HIGHWY BRIDGES DUE TO THE GREAT EAST JAPAN EARTHQUAKE

Jun-ichi Hoshikuma[1]

Abstract

The 2011 Great East Japan Earthquake struck the Tohoku and Kanto area on March 11, 2011. Immediately after the earthquake, research engineers in NILIM/PWRI were dispatched to Tohoku area to investigate the damage to bridges and suggest the technical advice to bridge administrators. On behalf of NILIM/PWRI joint reconnaissance team for bridge damage, author summarizes damage characteristic of bridges due to the earthquake in this paper. Technical issues learned from the damage of bridges were also described based on the investigation of bridge damage.

Introduction

The 2011 Great East Japan Earthquake occurred at 2:46 pm on March 11, 2011. The catastrophic damage resulting from strong ground motion and huge tsunami was caused in Tohoku and Kanto regions. More than 20,000 people were killed or missing and various infrastructures were damaged, especially in the coastal area of Iwate, Miyagi, Fukushima and Ibaraki Prefectures.

Many highway bridges were also 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 bridge damage and provided the technical supports and suggestions to bridge administrators, including Regional Bureaus of MLIT and some Local Governments. Furthermore, Task Committee G of UJNR Panel conducted the U.S.-Japan joint reconnaissance to bridge damage in early June, 2011.

This paper presents damage characteristic of bridges. Following topics were focused; bridge damage due to tsunami inundation or strong ground motion effect, verification of seismic performance of bridges retrofitted after the 1995 Hyogo-ken Nambu earthquake, validations of effectiveness of the current seismic design specification.

The 2011 Great East Japan Earthquake and Ground Motion

The main shock of this earthquake (Mw=9.0, focal depth=24km) occurred at 2:46 pm (JST) on March 11, 2011. 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 Tohoku and Kanto areas. Fig. 1 shows acceleration ground motion waveforms and spectral response accelerations at representative strong ground motion observation sites.

[7] Chief Researcher, Center for Advanced Engineering Structural Assessment and Research, PWRI

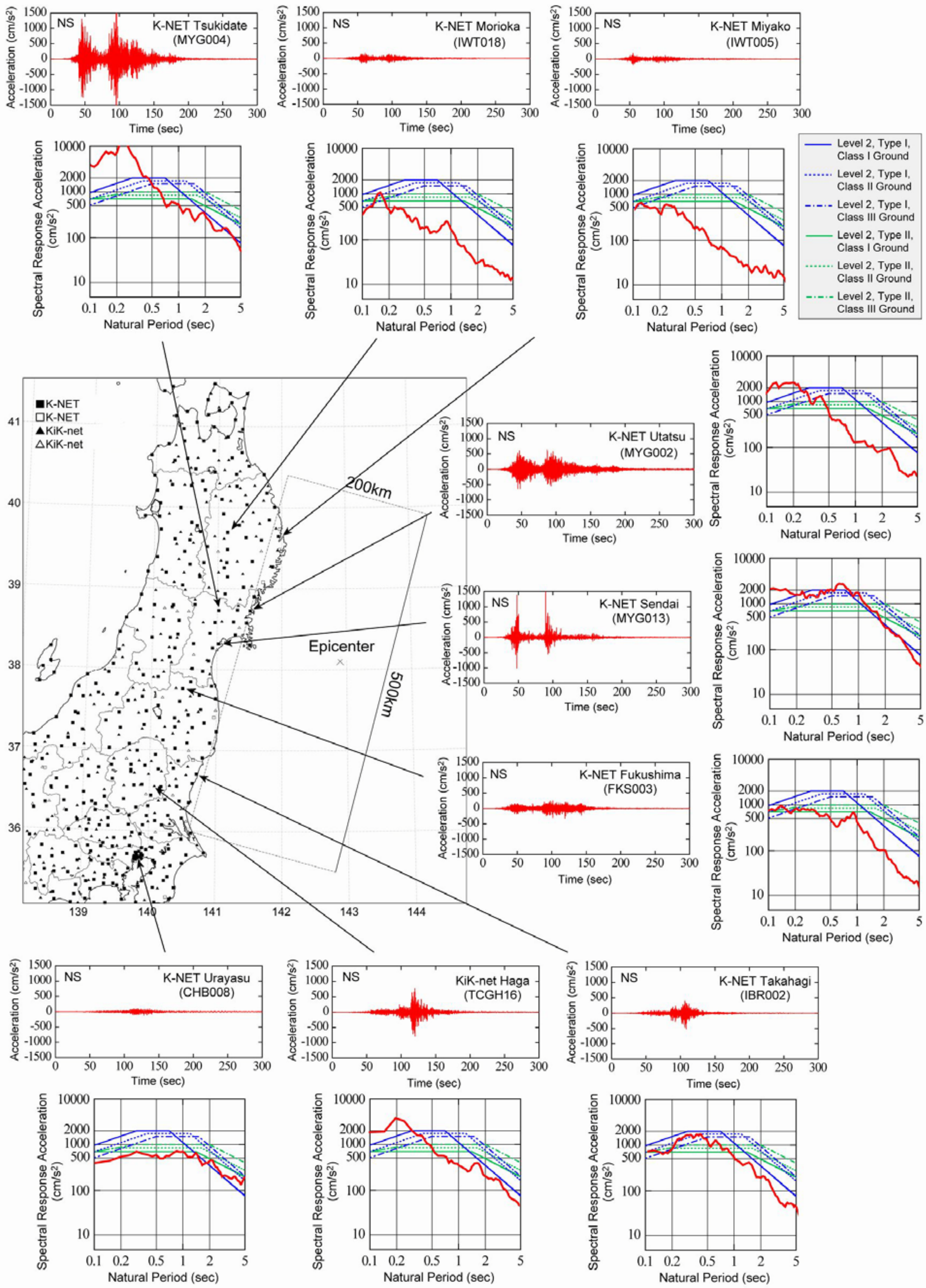

Fig. 1 Acceleration Waveforms and Spectral Response Acceleration at Main Shock (NS comp.)

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 epicenter. This is because large fault areas collapsed continuously. It was observed at very large maximum response acceleration at the range of short predominant period such as Tsukidate record. The maximum response accelerations at the range of natural periods from 1.0 to 2.0 seconds, which relatively correlate with damage of 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 Tohoku region were not so large. However, strong ground motions and large response accelerations were observed at the sites where located slightly far from epicenter such as Fukushima, Tochigi and Ibaraki prefectures.

Huge tsunami induced by main shock struck at Tohoku and Kanto coastal areas and exceeding 10m in height of wave were observed.

Moreover, aftershocks with the JMA magnitude of 7.0 or over were occurred three times within a day and total of 89 aftershocks with the magnitude of 6.0 or over were occurred until August 3.

Overview of Damage in Bridges

Damage of the highway bridges due to this earthquake can be categorized as effect of strong ground motion, effect of tsunami inundation, and effect of soil liquefaction. It should be noted in this earthquake that the intensive damage in highway bridges was mainly caused by tsunami inundation. Superstructures in twelve bridges including service road for pedestrian on national highway route 45 (main route along the Pacific coast of Tohoku Area) were washed away, which resulted in the traffic close after the earthquake. About 91 highway bridges in total were washed away due to tsunami inundation in Iwate, Miyagi, Fukushima, Ibaraki and Chiba prefectures. On the other hand, as long as we have investigated, 105 bridges survived even though the superstructures of these bridges were inundated with the tsunami. The backfill of abutment in some bridges were also washed out even though super- and substructures survived.

The ground motion effect to damage of bridges was less significant than the tsunami effect. One bridge (Rokko Ohashi Bridge, an old steel girder bridge supported by steel pile-bent columns located in Ibaraki prefecture) was collapsed due to the ground motion of the earthquake. Although the collapsed bridge was observed at the only Rokko Ohashi in the highway bridges, it was found in the bridge designed in accordance with pre-1980 design specifications that 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 have already observed in the past earthquakes. However, rupture of elastometric rubber bearings were observed at the Sendai-Tohbu viaduct designed based on Post-1995 design specifications.

After the Kobe Earthquake, the seismic retrofit project has been performed for existing bridges columns designed in accordance with pre-1980 specifications with high priority, to prevent the collapse of the bridge structure and unseating of the deck.

Almost of retrofitted bridge columns were not damaged due to the ground motion of the earthquake, which would exhibit the effectiveness of the seismic retrofit.

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

Bridge Damage Due to Ground Motion

Damage of Unretrofitted Bridges Designed in Accordance with Pre-1980 Design Specifications

Intensive damage due to the ground motion was developed in many unretrofitted bridges designed in accordance with pre-1980 design specifications. Almost of damage modes of those bridges have ever been observed in the past earthquakes. Photos 1 to 3 show the damage to reinforced concrete piers, steel bearing supports and the attachment of the bearing support to pier top or the superstructure, respectively. Although some bridge columns collapsed during the 1995 Kobe earthquake, the only one bridge collapsed during the 2011 Great East Japan earthquake. As shown in Photo 4, an old steel girder bridge supported by steel pile-bent columns (Rokko Ohashi Bridge) in the local roadway was collapsed due to the ground motion of the earthquake. Rokko Ohashi is to be replaced a new bridge and the new one was being constructed at the time of the earthquake.

Damage of 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 was specified for improving the confinement effect and shear capacity of the RC columns.

Photo 1 Damage of Reinforced Concrete Columns at Cut-off Section of Longitudinal Reinforcement

Photo 2 Damage of Steel Bearing Support

Photo 3a Damage of Pier Top

Photo 3b Damage of Attachment of Bearings to Superstructure

Photo 4 Collapse of Steel Pile-bent Columns

However, as shown in Photo 5, it should be noted that elastometric rubber bearings ruptured at the Sendai-Tohbu viaduct designed based on Post-1995 design specifications. Fig. 2 illustrates the structure of the Sendai-Tohbu viaduct and the positions of the elastometric rubber bearing with the rupture. As seen in Fig. 2, bridge structure in this junction is very complicate. Bridge width changes significantly in the 4-span continuous box girder with the change of the section. Span length of the section from P52 to 56 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 elastometric rubber bearings in the transverse direction was observed at the deck-end of P52 side in 4-span continuous box girder, though there are few damaged elastometric rubber bearings at the other deck-end side of the box girder on Pier 56. Photo 6 shows failure mode of the elastometric rubber bearing with the rupture. Comparison of the failure mode with the result of the shear loading test should be made, to estimate the capacity of the elastometric rubber bearing and the applied seismic force combination.

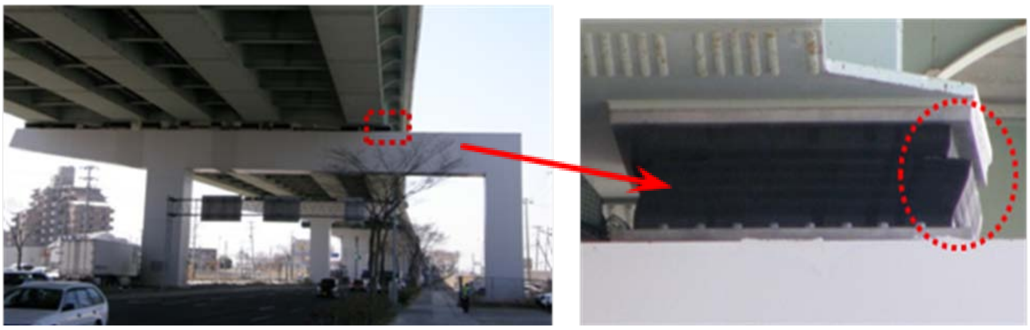

Photo 5 Rupture of Elastometric Rubber Bearings in Sendai Tobu Viaduct
(P54, Two-column Steel Frame)

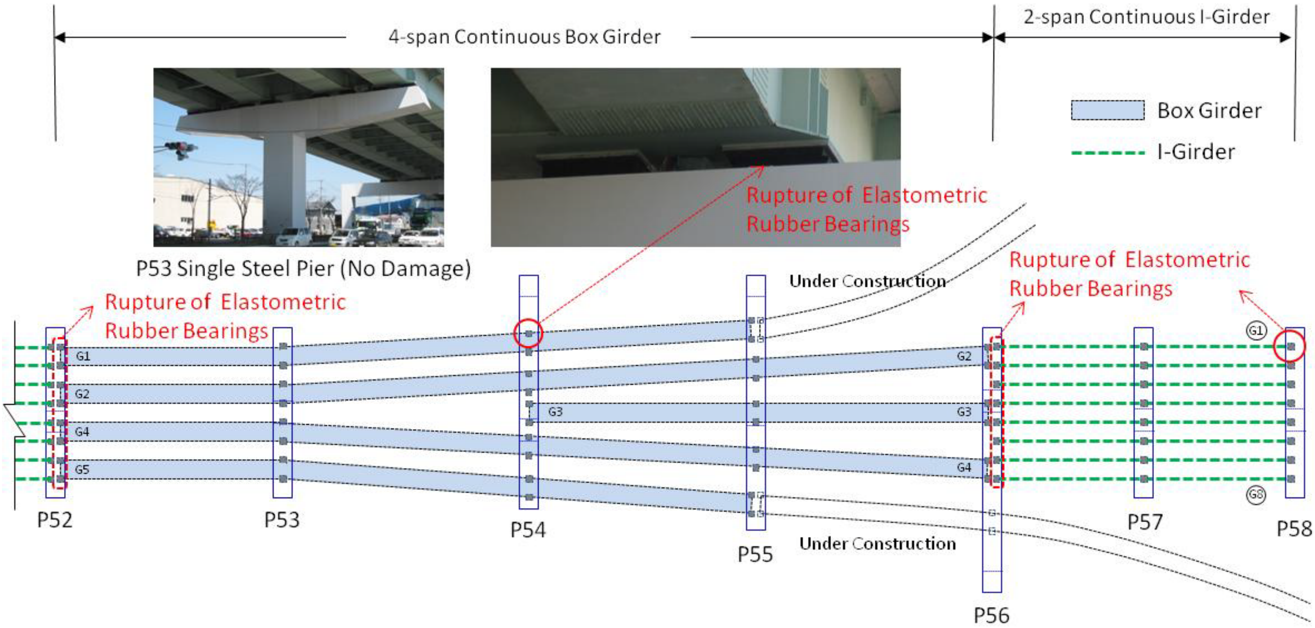

Fig. 2 Bridge Structure of Sendai-Tobu Viaduct and Position of Elastometric Rubber Bearing with Rupture

Photo 6 Details of Rupture of Elastometric Rubber Bearings

Damage of Retrofitted Bridges

Based on the lessons learned from the 1995 Kobe Earthquake, the seismic retrofit project has been performed for existing bridges 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.

Photo 7 exemplifies the effectiveness of the seismic retrofit for bridge columns. As seen in Photo 7, there are two adjacent river-crossing bridges. Since one bridge (Nakagawa Bridge) is on the designate emergency route, bridge columns designed with the 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 have not yet been retrofitted at the earthquake. Although Kunita Ohashi Bridge suffered from the vulnerable damage and thus lost the serviceability for the bridge, Nakagawa Bridge did not suffer from the damage and kept the serviceability soon after the earthquake. Seismic performance shown in these two bridges clearly exhibits the effectiveness of the seismic retrofit.

Photo 7 Comparison of Seismic Performance between Adjacent Two Bridges (Nakagawa Bridge and Kunita Ohashi Bridge)

On the other hand, there are a few remarkable damage examples in the retrofitted bridges. Photo 8 shows adjacent two reinforced concrete columns in Kameda Ohashi Bridge. Each column supports 2-span continuous steel box girder at the middle. The outbound column was designed with 1980 specifications and retrofitted by reinforced concrete jacketing for strengthening the cut-off section without increasing the flexural strength of the column base. Furthermore, additional shear keys were anchored to 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 Photo 8. The inbound column was designed with 1994 specifications basically and some modifications were made based on the 1995 tentative specifications (published soon after 1995 Kobe Earthquake). In the inbound column, elastometric rubber bearings were deformed in the transverse direction and the side stoppers were failed. Concrete of the beam edge portion attaching the supplemental shear keys also spalled off (see Photo 8) due to the transverse seismic force induced by the inertia of the superstructure, while vertical crack observed in the beam of the outbound column was not developed in the inbound beam.

Photo 8 Comparison of Damage Mode between Adjacent Two Bridge Columns

(Kameda Ohashi Bridge)

Photo 9 shows the failure of pier top. This type of damage has ever observed in the old bridge columns since the past earthquakes. Bearing capacity of the pier top was insufficient to transmit the seismic force to column. It should be noted in this bridge that the crack reaches the portion of the anchor bars of the steel bracket for attaching the unseating prevention devices. Since the unseating prevention devices should work at the unexpected situation of failure of bearing supports.

Photo 9 Crack to Pier Top Affects Attachment

Bridge Damage Due to Tsunami Inundation

Many bridges built along the coast line of the Pacific Ocean in Tohoku and Kanto areas were affected by the huge tsunami. Photos 10 and 11 show the typical damage modes of bridge due to the tsunami inundation.

The bridge shown in Photo 10a is a 6-span steel girder bridge (twin three-span-continuous girders), namely Koizumi Bridge. The steel girder was supported by RC pier walls with steel pipe piles. The pier walls with the fix bearings (P2 and P4) were retrofitted by FRP sheets, furthermore viscous dampers were installed to the deck-ends. The height of tsunami has been estimated exceeding 10 meter high in this area, which means the steel girder was completely inundated with tsunami. Tsunami effect was so significant to Koizumi Bridge that the whole girder were washed away upstream and one RC pier-wall (P3) was also washed away. The pier-wall of P3 supported two girders with movable bearing supports was found about 50m away from original position, while the pile foundation and the footing for P3 remained at the original position. The pier-wall failed at the bottom section as shown in Photo 10b. The probable reason for only P3 being washed away was due to smaller ultimate strength than the other pier-walls. It is also noted that backfill soil of abutment was also washed away at both sides.

Photo 11shows a 12-span PC single girder bridge, namely Utatsu Ohashi Bridge. Piers consisted of circular RC column (P1 and P2) and rectangular RC column with PC piles. Bridge columns were retrofitted by RC jacketing and the seat length was extended at the top of pier. Total of 8 spans (from P2 to P10) were washed away to the inland direction. It was found that additional concrete/steel shear keys, installed at the pier top for the seismic retrofit of existing bearings, were damaged and some hammerhead beams of piers in the inland side were cracked. A lot of diagonal cracks were also observed at both unseated and survived PC girders.

On the other hand, it is interesting to note that there are many survived bridges even though the superstructures of these bridges must be inundated with tsunami. Photo 12 exemplifies Yanoura Bridge survived washed away of superstructure due to the tsunami, while two spans of the water pipe bridge located adjacent to Yanoura Bridge were washed away. These survived bridges will hint the mechanism of resistance to the tsunami effect.

Impact of 2011 Great East Japan Earthquake on Seismic Design of Highway Bridges

The seismic performance of highway bridges, designed in accordance with the post-Kobe Japanese specifications, was very well and these bridges were functional without any long-term traffic stops after the earthquake. However, there are several important issues and lessons we should study and review for the latest seismic design specifications for highway bridges. Followings are the selected issues.

Photo 10a Washed Away of Steel Superstructure and Pier-wall (Koizumi Bridge)

Photo 10b Failure Section of Pier-wall (P3)

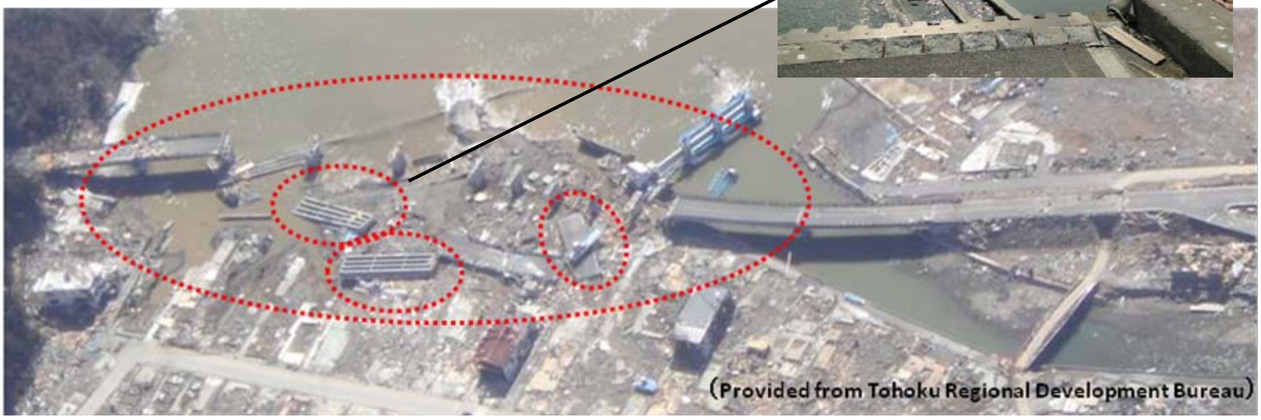

Photo 11 Washed Away of PC Superstructure (Utatsu Ohashi Bridge)

Photo 12 Bridge Survived Washed Away Due to Tsunami (Yanoura Bridge)

Ground Motion

In the 2011 Great East Japan Earthquake, many strong ground motion records were recorded and these records clearly showed that this earthquake generated ground motions with multiple pulses and thus the longer duration (more than 2 minutes) than other records observed in the past earthquakes. Similar ground motions were reported in the 2010 Chile Earthquake with the moment magnitude Mw 8.8. Therefore, the subduction-type earthquake with Mw of nearly 9 may induce the ground motion with long duration.

In general, the long duration would affect the number of cyclic inelastic response of the bridge system. Past experimental researches indicated that the loading pattern in the quasi-static cyclic loading test, particularly the number of cyclic loading affects the ductility capacity of flexural reinforced concrete column. In order to accommodate such effect into the seismic design, Japanese design specifications have determined two ductility/shear capacity factors based on the types of the ground motion, i.e. the subduction-type and the near-fault-type. Re-studies on the effect of the long duration will be required based on the ground motion observed in the 2011 Great East Japan Earthquake.

The long duration would also affect the soil liquefaction. Effect of the soil liquefaction on the seismic design of bridge foundation was introduced in the 1971 specifications in Japan based on the lessons learned from the 1964 Niigata Earthquake. Although there were no major liquefaction-induced damages in bridges during the 2011 Great East Japan Earthquake, the long duration effect on the bridge performance built on the liquefiable sandy soil condition should be verified through both geological and structural perspectives.

Since the ground motion effect propagated wide, bridge damage developed in wide area. Many ground motion records were also observed in wide area. It should be also important to study the relation between the properties of the ground motion and damage of bridges.

Tsunami Effect on Bridges

Superstructures in several bridges were washed away due to the tsunami effect. Backfill soil for the abutment was also washed out. Similar damage modes in bridge were also observed during the 2004 Indian Ocean Earthquake. Failure mechanism of bridge system due to the tsunami effect need to be studied, in which the resistance capacity of the existing bearing supports be analyzed based on both the washed-away and survived bridges. Also, more experimental researches on the bridge behavior due to the tsunami effect are required, to find the appropriate structural system for mitigating the tsunami effect.

On the other hand, the design concept of bridge for unexpected extraordinary event would be controversial, because structural resistance capacity has a limit. Basically, it would be one of the options for the extraordinary tsunami effect to avoid routing important highway network and locating important highway routes in the tsunami-inundation area. In terms of structural engineering, easy-to-recover bridge system employing the temporary structure with the accelerated construction technique is also another option.

Validations of Effectiveness of Seismic Retrofit

Seismic retrofit have been performed step-by-step since 1995 Kobe earthquake. Based on the lessons learned from the past earthquakes, bridge columns in the important highway network designed by pre-1980 specifications have been retrofitted with high prioritization. Many seismic vulnerable bridges in the important route such as National Highway Route 4, 6, 45 etc were retrofitted up to the date of the earthquake, which resulted in quick recovery of the functional highway network after the earthquake. It should be, however, important review to investigate details of the new type of damage in the retrofitted bridges and evaluate the seismic behavior of the bridge during the earthquake.

Conclusion Remarks

This report preliminarily summarized damage to highway bridge due to the 2011 Great East Japan Earthquake with focus of the seismic performance of the retrofitted bridges and tsunami effect. Based on the damage caused by the earthquake and tsunami, more analytical and experimental researches should be required to clarify the mechanism of the damage. Investigation results also indicate that subsidence of the backfill soil in the abutment has been remarkable with the improvement of seismic performance for bridge structures, though details are not reported in this paper. It would be important to ensure the seismic performance of both bridge structures and embankment for highway.

Acknowledgments

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