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SEISMIC BEHAVIOR OF RETROFITTED BRIDGES DURING THE 2011 GREAT EAST JAPAN EARTHQUAKE

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ABSTRACT: In order to examine the effectiveness of the seismic retrofit for bridges through the experience of the 2011 Great East Japan Earthquake, authors report the seismic behavior of some retrofitted bridges during the earthquake with comparison of the damage of unretrofitted bridges, and summarize that the seismic retrofit for bridge columns has been effective to prevent vulnerable failure in columns. It should be also remarkable that the structural members attached with the additional shear keys or the unseating prevention devices were damaged in some retrofitted bridges.

Key Words: the 2011 Great East Japan earthquake, retrofitted bridge, shear keys, unseating prevention devices

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 damaged in these areas due to both large ground motion and tsunami inundation (Hoshikuma 2011). This paper focuses on the damage to retrofitted bridges due to the ground motion effect. There were some retrofitted bridges with damage during the earthquake, while most of bridge damage due to the ground motion effect was observed in the bridges designed with pre-1980 specifications. Seismic retrofit projects have been performed to bridges step-by-step since the 1995 Kobe earthquake, to prevent fatal damage of bridges due to the ground motion observed in the 1995 Kobe earthquake. Based on the lessons learned from the past earthquakes, bridge columns designed with pre-1980 specifications have been retrofitted with high prioritization. It should be noted that such retrofitted bridges were actually excited due to the 2011 Great East Japan earthquake.

This paper presents the seismic behavior of some retrofitted bridges during the earthquake with comparison of the damage of unretrofitted bridges, so that examine the effectiveness of the seismic retrofit for bridge columns with insufficient development length of the cut-off longitudinal reinforcement bars at mid-height section. Remarkable damage of the structural members attached with the additional shear keys or the unseating prevention devices in the retrofitted bridge is also introduced in this paper.

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. The location of some bridges shown later in this paper is also marked on Fig. 1.

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.

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.

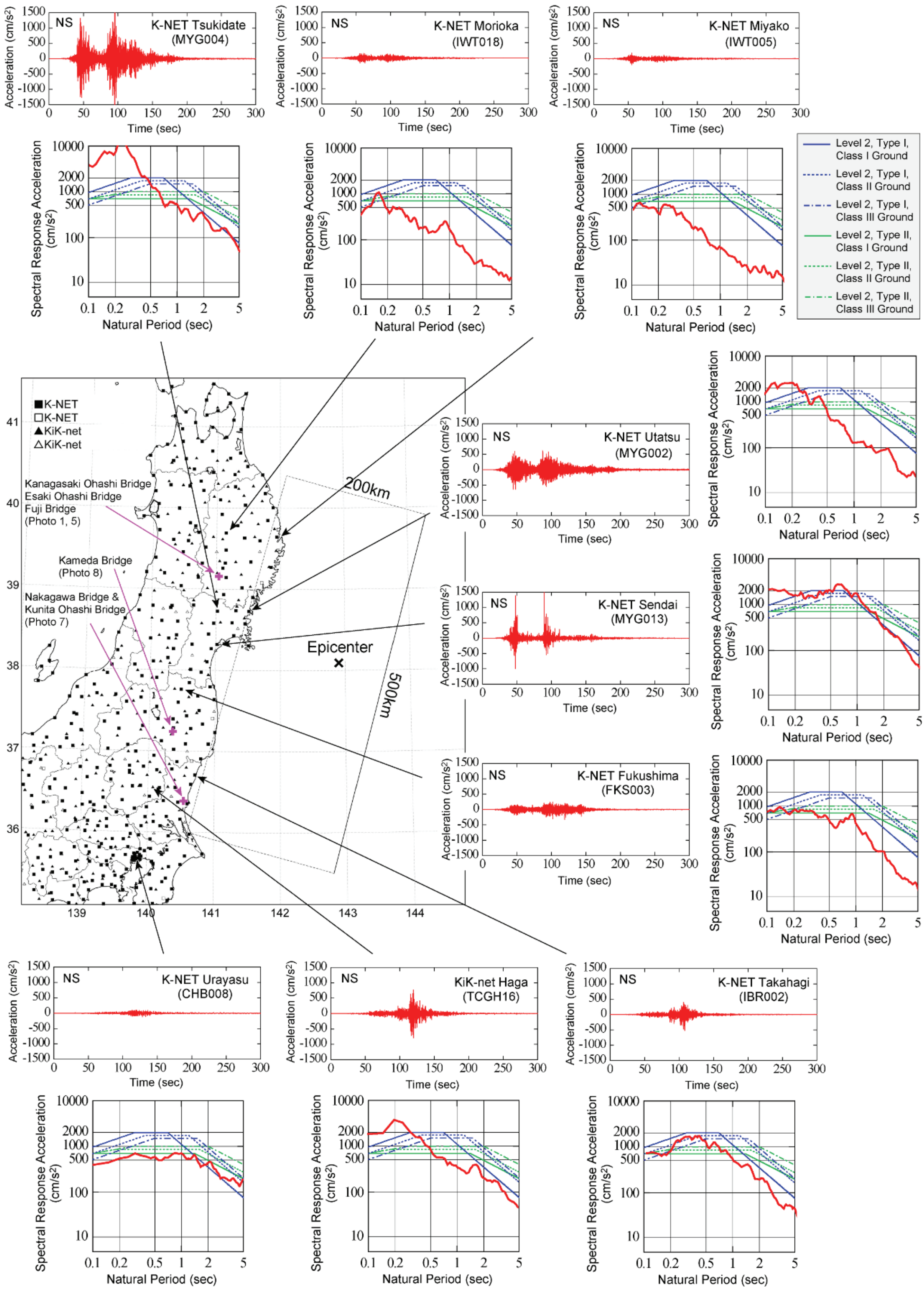


Fig. 1 Acceleration Waveforms and Spectral Response Acceleration at Main Shock (NS comp.)

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.

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

Photo 2 Damage of Steel Bearing Support

Photo 3 Damage of Pier Top

Photo 4 Residual Inclination of Substructure and Damage of Pile Cap for Caisson Foundation

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. Photo 4 shows the residual inclination of substructure due to the damage of the pile cap for caisson foundation.

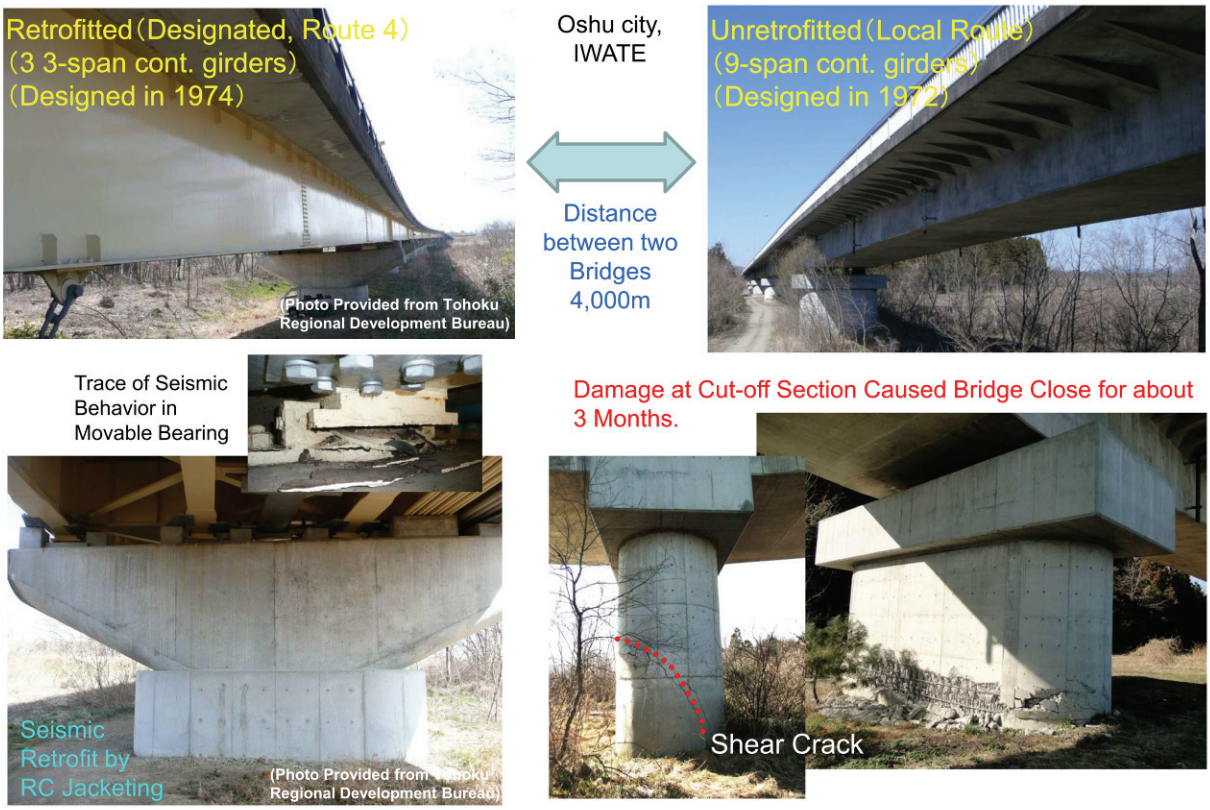


Photo 5 Comparison of Seismic Performance between Adjacent Two Bridges
(Kanagasaki Ohashi Bridge and Esaki Ohashi Bridge)

Comparison of Seismic Performance of Bridge between Retrofitted and Unretrofitted

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 5 exemplifies the effectiveness of the seismic retrofit for bridge columns. The unretrofitted bridge (Esaki Ohashi Bridge) shown in the right side of Photo 5 suffered from severe shear damage in concrete columns. Esaki Ohashi Bridge is 9-span continuous concrete box girder bridge designed in 1972 design specification. Near Esaki Ohashi Bridge (as close as 4,000m), there is the other bridge (Kanagasaki Ohashi Bridge) as shown in the left side of Photo 5, where this is three 3-span continuous steel girders bridge designed in 1974 and the columns were retrofitted by concrete jacketing. No structural damage was observed in this retrofitted bridge. Comparison of the seismic performance with these two bridges indicates that the seismic retrofit for bridge columns work effectively, although structural type of these bridges are different and thus the natural period is not equivalent between two bridges.

Photo 6 shows the other example of the comparison of the damage between seismic retrofitted bridge and unretrofitted bridge. As seen in Photo 6, 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.

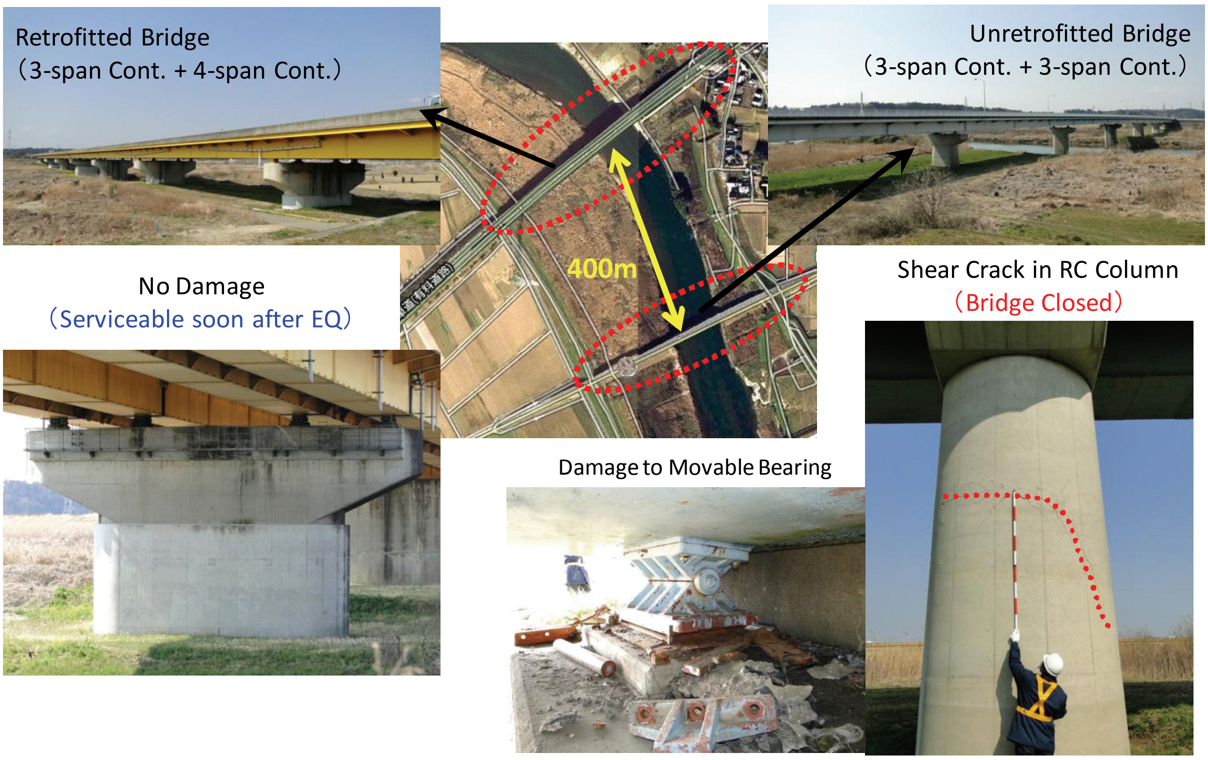


Photo 6 Comparison of Seismic Performance between Adjacent Two Bridges
(Nakagawa Bridge and Kunita Ohashi Bridge)

Damage of Cap Beam in Retrofitted RC Column

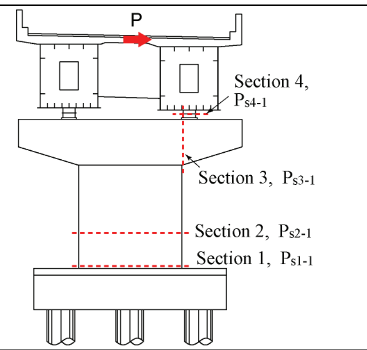
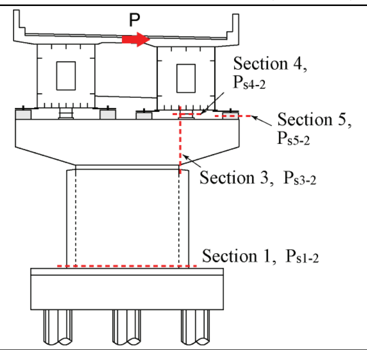
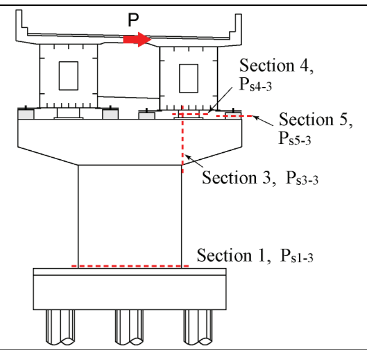
There are a few remarkable damage examples in the retrofitted bridges. Photo 7 shows adjacent two reinforced concrete columns in Kameda Ohashi Bridge. Each column supports 2-span continuous steel box girder at the middle. The outbound column was designed with 1980 specifications and retrofitted by reinforced concrete jacketing for strengthening the cut-off section without increasing the flexural strength of the column base. Furthermore, additional shear keys were anchored to cap beam to supplement the strength of the existing steel bearings. Although no damage was found to the steel bearings and shear keys, vertical cracks of nearly 10mm width were observed at the section of cap beam as shown in Photo 7. 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 cap beam edge portion attaching the supplemental shear keys also spalled off (see in Photo 7) due to the transverse seismic force induced by the inertia of the superstructure, while vertical crack observed in the cap beam of the outbound column was not developed in the inbound cap beam.

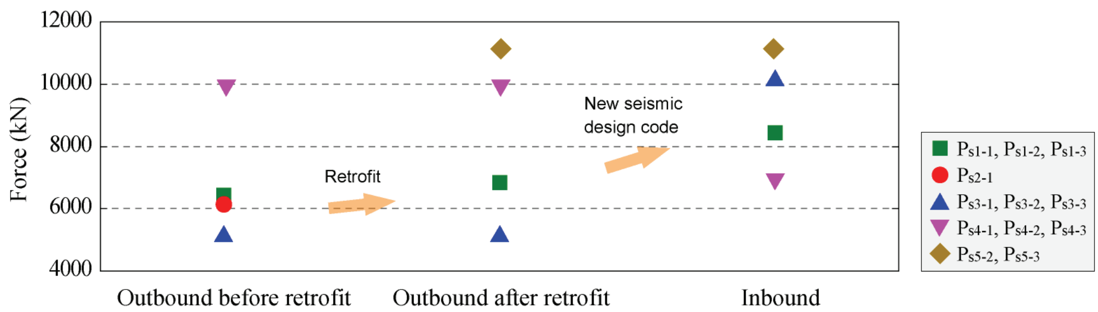
Amount of tensile steel bar in the cap beam was different between two columns. Tensile reinforcement ratio of the cap beam is 0.40% in the inbound column, while 0.24% in the outbound column. The section of the cap beam has been designed predominantly by the live load rather than the seismic effect. Actually the design live load is different between two columns, because the design specifications of the live load revised and larger live load was considered into the design of the inbound column. The difference of strength of the cap beam would affect the failure mode for the large seismic lateral force.

Table 1 shows the analytical result of the strength capacity of some sections in two columns when the transverse seismic force applies at the superstructure. The result of the push-over analysis indicates that the section of the cap beam shoulder (Section 3 in Table 1) in the outbound column is the first failure section regardless the effect of the seismic retrofit, while the section of the column base is the first failure section so that the cap beam be protected in the inbound column. These results coincide with actual behavior of the columns observed after the earthquake.

Photo 7 Comparison of Damage Mode between Adjacent Two Bridge Columns
(Kameda Ohashi Bridge)

Table 1 Comparison of Strength in Columns of Kameda Ohashi Bridge

Outbound before retrofit	Outbound after Retrofit (current)	Inbound
		
$P_{s1\text{-}1}$ = 6,440 kN	$P_{s1\text{-}2}$ = 6,850 kN	$P_{s1\text{-}3}$ = 8,450 kN
$P_{s2\text{-}1}$ = 6,117 kN	-	-
$P_{s3\text{-}1}$ = 5,100 kN	$P_{s3\text{-}2}$ = 5,100 kN	$P_{s3\text{-}3}$ = 10,100 kN
$P_{s4\text{-}1}$ = 10,040 kN	$P_{s4\text{-}2}$ = 10,040 kN	$P_{s4\text{-}3}$ = 7,040 kN
-	$P_{s5\text{-}2}$ = 11,110 kN	$P_{s5\text{-}3}$ = 11,110 kN
$P_{s3\text{-}1} < P_{s2\text{-}1} < P_{s1\text{-}1} < P_{s4\text{-}1}$	$P_{s3\text{-}2} < P_{s1\text{-}2} < P_{s4\text{-}2} < P_{s5\text{-}2}$	$P_{s4\text{-}3} < P_{s1\text{-}3} < P_{s3\text{-}3} < P_{s5\text{-}3}$



$P_{s1\text{-}1}$, $P_{s1\text{-}2}$, $P_{s1\text{-}3}$: Flexural strength of the base section of the column (section 1)

$P_{s2\text{-}1}$: Flexural strength of the cut-off section of the longitudinal reinforcement (section 2)

$P_{s3\text{-}1}$, $P_{s3\text{-}2}$, $P_{s3\text{-}3}$: Flexural strength of the section of the cap beam shoulder (section 3)

$P_{s4\text{-}1}$, $P_{s4\text{-}2}$, $P_{s4\text{-}3}$: Shear strength of the shear key for the rubber bearing (section 4). This shear key was designed based on earthquake with high probability of occurrence for the bridge service life (level 1 earthquake ground motion)

$P_{s5\text{-}2}$, $P_{s5\text{-}3}$: Shear strength of the shear key (section 5). This shear key was designed based on strong earthquake with low probability of occurrence for the bridge service life (level 2 earthquake ground motion)

Damage of Steel Truss Attached with Cable Restrainer

Photo 8 shows the other example of the damage in the retrofitted bridge. Osaragi Ohashi Bridge is a 3-span continuous steel truss bridge and the cable restrainers are installed so as to prevent from unseating of the superstructure in the event of unexpected failure of bearing supports during an earthquake. In this steel truss bridge, the cable restrainers were attached to the lower chord with steel bracket and additional steel plate for strengthening of the section of attachment as shown in Photo 8. Any bearing supports didn’t suffer from the damage due to the earthquake and there was no evidence of the superstructure movement, which indicated that the cable restrainers didn’t work during the earthquake. However, the lower chord was slightly buckled at near the section of attachment for the cable restrainer. This damage was caused by the seismic force transmitted from the fix bearing support (not from the cable restrainer). Significant change of the stiffness and strength at the section of the attachment may cause the buckling due to the cyclic loading.

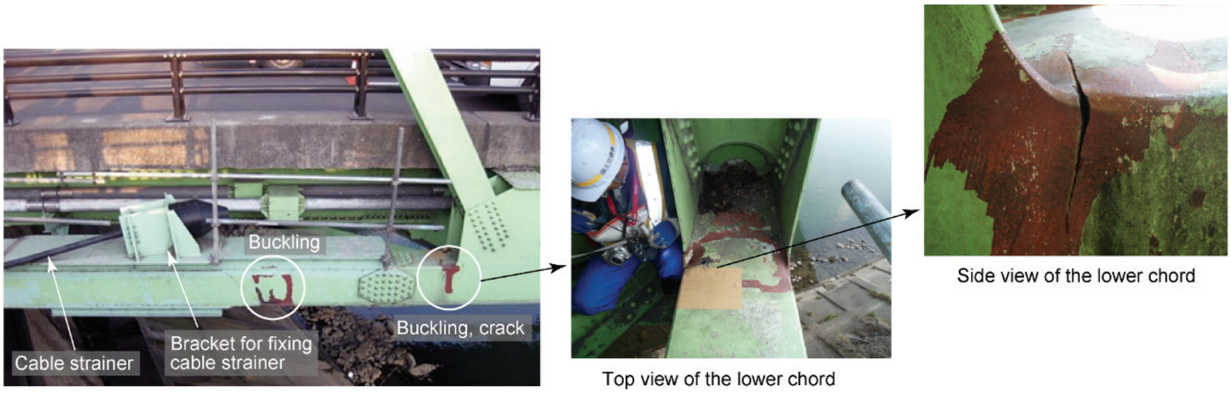


Photo 8 Damage of the lower chord of truss bridge (Osaragi Ohashi Bridge)

IMPACT OF 2011 GREAT EAST JAPAN EARTHQUAKE ON SEISMIC DESIGN OF HIGHWAY BRIDGES

The seismic performance of retrofitted highway bridges was very well except a few bridges described above. The retrofitted 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.

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 (Chen 2010a, Kawashima 2010b). 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.

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 described in this paper and evaluate the seismic behavior of the bridge during the earthquake.

CONCLUSION REMARKS

This report summarized damage to highway bridge due to the 2011 Great East Japan Earthquake with focus of the seismic performance of the retrofitted bridges. Based on the damage caused by the earthquake, 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.

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