Damage of Road Bridges by 2011 Great East Japan (Tohoku) Earthquake

K. Kawashima
Tokyo Institute of Technology

H. Matsuzaki
Tohoku University

SUMMARY:
This paper presents damage of bridges during the 2011 Great East Japan earthquake. Since the bridges in the north Miyagi-ken and south Iwate-ken suffered extensive damage during the 1978 Miyagi-ken-oki earthquake, damage of bridges during the 2011 Great East Japan earthquake is evaluated in comparison with the damage during the 1978 Miyagi-ken-oki earthquake so that effect of the recent progress of seismic design can be evaluated. Tsunami-induced damage was extensive for bridges along the Pacific Coast. Typical feature of tsunami-induced damage is presented based on field investigation and video movies.

Keywords: Great East Japan earthquake, seismic damage, bridges, tsunami, design code

1. INTRODUCTION

Extensive damage occurred to bridges in a wide region from Tohoku to Kanto due to the M_w9.0 March 11, 2011 Great East Japan earthquake. Seismic design code of bridges was extensively enhanced since 1990 because of incorporation of the inelastic static analysis, enhancement of ductility capacity of reinforced concrete and steel columns, enhancement of unseating prevention devices, design for residual displacement, seismic isolation, and elastomeric bearings including lead rubber bearings and high damping rubber bearings for replacement of vulnerable steel bearings (Kawashima 2000, Kawashima 2006a, Unjoh et al. 2010). Among a number of bridges which suffered damage during the 2011 Great East Japan earthquake, the bridges in the north Miyagi-ken and south Iwate-ken suffered extensive damage during the 1978 Miyagi-ken-oki earthquake. Extensive seismic retrofit of columns which are vulnerable to shear failure due to insufficient development of longitudinal bars at cut-offs has been conducted since the end of 1980s (Kawashima 2006b).

Thus, the Great East Japan earthquake was a valuable opportunity to evaluate the effectiveness of recent progress of seismic design code by comparing damage due to two earthquakes. Damage of bridges is shown here for two categories: bridges which were designed in accordance with the pre-1990 design codes and the post-1990 design codes. Effect of the seismic retrofit is also presented.

Fig. 1 shows Type I and Type II design response spectra which were introduced since 1990 and 1995 (JRA 1990, JRA 1996). Type I design ground motions typically represent the ground motions induced by M8 subduction events nearly 50km from coast while Type II design ground motions typically represent the ground motions induced by M7 near-field inland events. Duration of ground accelerations can be 200-300s in Type I ground motions though it is short in Type II ground motions.

A number of strong motion accelerations with the highest PGA of 27.0m/s^2 (Tsukidate City) was recorded in a wide area. Short period components such as 0.2s were predominant in most ground accelerations recorded by K-net, Kik-net and JMA-net since most sensors were installed at rock sites for aiming of recording rock accelerations. Thus the information from K-net, Kik-net and JMA-net was misleading the nature of ground accelerations such that ground accelerations included only short

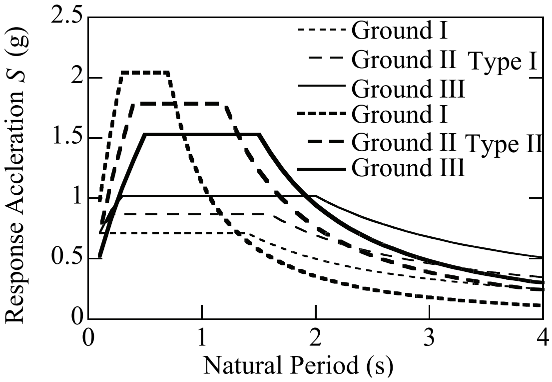


Figure 1. Type I and II design response accelerations

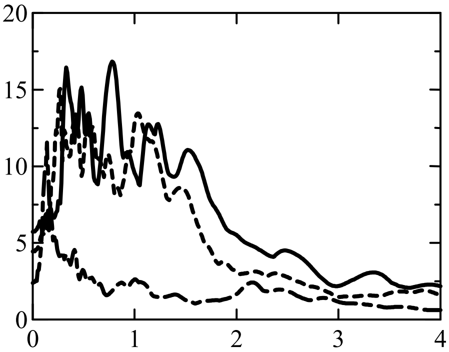


Figure 2. Response acceleration (Furukawa, Osaki City (K-net))

period components. Structures suffered only minor damage at the above mentioned Tsukidate City where 27.0m/s^2 PGA was recorded. It is obvious that short period components result in only limited damage to engineering structures no matter how PGA was high. However there were several sites where sensors were installed at weak soil sites resulting in large response accelerations at 0.5-2s. For example, Fig. 2 shows response accelerations at Osaki-City (K-net) which had 15m/s^2 or higher response accelerations at 0.5-1.5s. Thus it may be considered that the response accelerations at 0.5-1.5s which are important for ordinary bridges were nearly the same level or slightly smaller than the design acceleration response shown in Fig. 1.

On the other hand, it was the first time in Japan to have extensive damage to bridges by tsunami in recent years. No single word about tsunami was included in the design codes. Of course, extensive damage occurred in the past, but it was regarded as an unavoidable natural disaster in the early days.

This paper presents ground-motion-induced damage and tsunami-induced damage of road bridges in the north Miyagi-ken and south Iwate-ken during the 2011 Great East Japan earthquake.

2. GROUND-MOTION-NDUCED DAMAGE

2.1. Damage to bridges which were constructed in accordance with the pre-1990 design codes

Extensive damage occurred at the bridges which were designed in accordance with the pre-1990 design code and not yet been retrofitted in accordance with the post-1990 design codes. For example, Photo 1 shows flexural-shear failure of reinforced concrete piers at Fuji Bridge. The damage occurred due to an overestimated concrete shear capacity and an inadequate development of longitudinal bars at

Photo 1. Shear failure due to insufficient termination of longitudinal bars at cut-offs (Fuji Bridge, courtesy of Hoshikuma, J., PWRI)

Photo 2. Yuriage Bridge

(a) 1978 Miyagi-ken-oki earthquake

(b) 2011 Great East Japan earthquake

Photo 3. Damage of steel roller and pin bearings Yuriage Bridge

cut-off which were the common practice prior to 1980. Such a failure occurred extensively during the1995 Kobe, Japan earthquake (Kawashima and Unjoh 1997). Extensive investigation was directed to clarify the failure mechanism of such damage (Kawashima et al 1995) including a series of large scale shake table experiments using E-Defense. It should be note that damage progresses very sharply once shear cracks were initiated under this failure mechanism. Seismic retrofit was initiated in the 1980s, and it was accelerated after the 1995 Kobe earthquake. Over 30,000 columns were so far retrofitted. Consequently, during the 2011 Great East Japan earthquake, damage due to this mechanism did not occur at the bridges which were retrofitted, while the damage still continued to occur at the bridges which were not yet been retrofitted.

Yuriage Bridge (refer to Photo 2) suffered extensive damage at reinforced concrete hollow and solid columns, an end of prestressed concrete girders, and steel pin and roller bearings during the 1978 Miyagi-ken-oki earthquake as shown in Photo 3(a). Since the damaged columns were repaired and retrofitted by reinforced concrete jacketing, they did not suffer damage during the 2011 Great East Japan earthquake. However steel pin and roller bearings suffered extensive damage again in the similar mode as shown in Photo 3 (b). It is obvious that steel pin and roller bearings are vulnerable to seismic action, because the stress builds up to failure by allowing no relative displacements at pin bearings and relative displacements accommodated by roller bearings are insufficient to real relative displacement under a strong excitation. Furthermore, exactly the same end of a prestressed concrete girder which suffered damage during the 1978 Miyagi-ken-oki earthquake suffered damage.

2.2. Performance of retrofitted bridges

Bridges which had been retrofitted suffered virtually no damage. For example, Sendai Bridge which is an extremely important bridge in Sendai City suffered extensive damage at reinforced concrete piers and steel bearings as shown in Photo 4(a) during the 1978 Miyagi-ken-oki earthquake. However this bridge suffered no damage during the 2011 Great East Japan earthquake, because columns were retrofitted as shown in Photo 4(b) and the original steel bearings were replaced with elastomeric bearings.

Shin-Iino-gawa Bridge which carries National Road 45 over Kitakami River suffered extensive damage at steel pin and roller bearings as shown in Photo 5(a) during the 1978 Miyagi-ken-oki earthquake. It was retrofitted prior to the 2011 Great East Japan earthquake such that 1) several reinforced concrete piers were retrofitted using steel jacketing, 2) nonlinear viscous dampers were installed between a superstructure and a substructure, and 3) steel bearings were replaced with elastomeric beatings as shown in Photo 5(b). As a result, the bridge suffered no damage during the 2011 Great East Japan earthquake.

2.3 New bridges constructed in accordance with the post-1990 codes

Bridges which were designed in accordance with the post-1990 codes suffered essentially no damage

(a) 1978 Miyagi-ken-oki earthquake

(b) 2011 Great East Japan earthquake

Photo 4. Effect of seismic retrofitting of piers, Chiyoda Bridge, National Road 4

(a) Rupture of a pin in a pin bearing during 1978 Miyagi-ken-oki earthquake

(b) An elastomeric bearing which did not suffer damage during 2011 great east Japan earthquake

Photo 5. Effect of seismic retrofitting of bearings, Shin-Iino Bridge, National Road 45

(a) Bridge after Great East Japan Earthquake

(b) Elastomeric bearing and cable restrainer without damage

Photo 6. Shin-Tenoh Bridge, National Road 45

during the Great East Japan earthquake. For example, Photo 6 shows Shin-Tenno Bridge that was constructed in 2002. This bridge suffered no damage. Elastomeric bearings and new cable restrainers which satisfied the requirements of the post-1990 design code were set. The bridge was located only 200m upstream of Tenno Bridge which suffered extensive damage during the Great East Japan earthquake.

Elastomeric bearings generally performed quite well under the extreme ground motions. However it should be noticed that elastomeric bearings ruptured in several bridges. For example, several elastomeric bearings ruptured such that a deck offset by 0.5m in the transverse direction as shown in Photo 7(a) at Sendai-Tobu Viaduct. Rubber layers detached from steel plates in addition to rupture of

(a) Offset of left girder by 0.5m due to rupture of elastomeric bearings (NEXCO East)

(b) Rupture of an elastomeric bearing

Photo 7. Damage of elastomeric bearings at Sendai Tobu Expressway

rubber layers as shown in Photo 7(b). Though detailing is not yet released, there may be two possible reasons for the damage. The first is a miss design and fabrication of the elastomeric bearings. The second is an interaction between adjacent decks. Since an expansion joint must have constrained relative displacement between adjacent decks in the transverse direction, it is possible that a larger displacement demand of a deck is imposed to an adjacent deck resulting in larger shear deformation in the damaged bearings.

3. TSUNAMI INDUCED DAMAGE

3.1. Bridges which suffered damage by tsunami

A number of bridges suffered damage by tsunami. Overturning of substructures due to scouring did not occur in road bridges though it happened in railway bridges. Bridges which were built in the early days were generally vulnerable to tsunami effect since connection of spans to substructures was weak. Bridges which were taller than tsunami waves did not suffer damage. Many short bridges with short spans generally survived tsunami though they were completely covered by tsunami because tsunami wave did directly hit spans and they were well constraint by abutments. Back fills and embankment were eroded and lost at many bridges. Though repair of back fills and embankment was easier than repair of bridge structures, an appropriate protection should be considered in the future.

3.2. Utatsu bridge

Utatsu Bridge built in 1972 carrying National Road 45 at Minami-sanriku Town over Irimae Bay suffered extensive damage by tsunami as shown in Photo 8. It was a 303m long 12 simply supported PC girder bridge consisting of 3 types of superstructures with spans ranging from 14.4m to 40.7m. Diaphragms were set between PC T-beam girders at the ends and mid points. The concrete decks had an inclination as large as 6% in the transverse direction due to curved alignment of the bridge.

The bridge was retrofitted a few years ago. Columns were retrofitted by reinforced concrete jacketing and unseating prevention devices were installed. The spans S1, S2, S11 and S12 remained but spans S3-S10 were washed ashore. Spans S3-S7 were simply supported pre-tensioned PC T-beam girder bridges. As a part of the seismic retrofit, cable restrainers were set for between S3-S7, and three steel stoppers were provided at the end of S3 and S7 for preventing excessive longitudinal deck movements. Though cable restrainers ruptured between S4 and S5, S3-S4 and S5-S6-S7 were still tied together after floated ashore as shown in Photo 9. S8, S9 and S10 laid down upside down as shown in Photo 10.

Photo 8. Utatsu Bridge (Google)

Photo 9. Spans S5-S8

Photo 10. Overturned Span S8

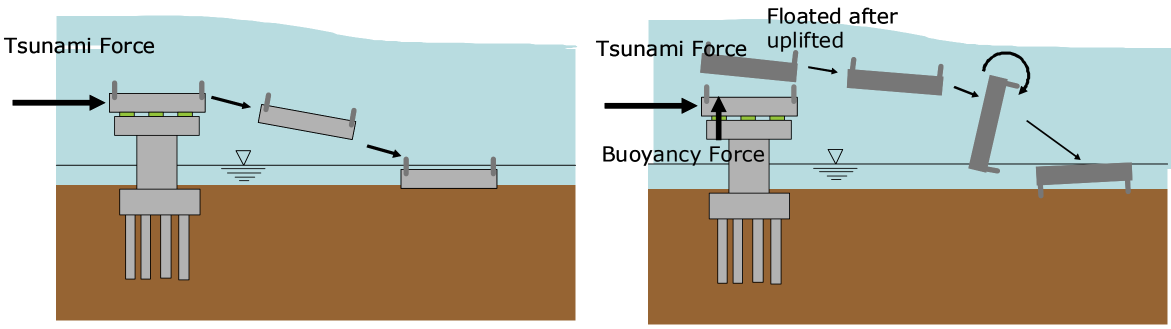
(a) P10

(b) Steel bearings and stoppers on P10

Photo 11. Stoppers and bearings on P10

(a) Tsunami reached the bottom of girders (b) The bridge was completely under water

Photo 12. Utatsu bridge, National Road 45, under tsunami water



(a) Transverse drag (b) Floated after uplifted

Figure 3. Failure mechanisms of spans due to tsunami effect

Photo 11 shows three stoppers and four pot bearings on P10. Three longitudinal stoppers did not suffer damage at all. All four upper bearings were detached from the lower bearings and remained at the end of S10. Three side stoppers in the sea side were removed due to rupture of four anchor bolts each. The sea most lower bearing slightly uplifted but other three lower bearings were in their original position without damage. From the fact that three stoppers neither suffered damage nor tilted, it was likely that S10 was uplifted higher than the top of three stoppers before floated. Other shorter spans (S2-S6) were simply dragged laterally by tsunami.

A video was taken at a slope near the north abutment A2 by a local resident. This video shows a whole rising process of tsunami until the spans were completely covered by tsunami as shown in Photo 12. Since tsunami reached the top of a 6m tall pole shown at the center of the photo, it is estimated that the deck surface was saturated 6m deep. Since the spans failed after they were completely under water, it is not known when the spans were uplifted and floated. At both ends of the bay, tsunami flow was not as fast as the flow at the center, which saved spans S1-S2 and S11-S12 from collapse. Tsunami flow velocity at the center of the bay was about 6m/s.

Damage of Utatsu Bridge may be summarized as shown in Fig. 3. If spans uplift before floated under tsunami action, it may be effective to install restrainers in the vertical direction between girders and substructures so that bridge spans can be tied down to substructures. Such a tsunami unseating prevention device may be cheaper and effective as a countermeasure for tsunami. Since installation of tsunami unseating prevention devices imposes an additional upward force to substructures, substructures have to be strengthen if they do not have enough capacity. However because an uplifting force due to tsunami action does not reach several times the span weight, strengthening of substructures is not generally required at most bridges except very old bridges.

Photo 13. Yanoura bridge after tsunami, National Road 45

Photo 14. Yanoura bridge 5m under water

3.3. Bridges which survived tsunami

There are a number of bridges which survived tsunami though they were completely saturated by tsunami. For example, Yanoura Bridge that carried National Road 45 over Koshi River in Kamaishi City was a 108.6m long curved three span simply supported steel girder bridge. Photo 13 shows the bridge after tsunami. Two abutments and two piers were supported by four pile foundations with ten 30m long and 1m diameter cast in place reinforced concrete piles.

A video was taken by Kamaishi Port Office, Ministry of Land, Infrastructure and Transportation, located at the left bank 140m from the bridge. At about 15:00, the first tsunami reached the bridge as shown in Photo 14. The spans were already completely underwater. Based on the fact that tsunami reached the mid-height of the second story of a building close to the bridge, it is known that the deck surface was 5m underwater. Though a large amount of wooden house originated debris was included in tsunami which propagated upstream along both sides of the river, few debris was transported by tsunami which propagated in the river because the bridge was located the most downstream.

As shown in Photo 13, a white center stripe did not drift indicating that the spans did not offset in the transverse direction. The bridge suffered only minor damage on hand rails. The bridge remained open for use by emergency vehicles following the earthquake.

4. PRELIMINARY EVALUATION FOR UPLIFT AND FLOATING OF UTATSU BRIDGE

A preliminary analysis was conducted to evaluate possible uplift of spans of Utatsu Bridge. As shown in Photo 10, since the PC T-beam girders had diaphragms at both ends and mid spans, they were subjected to uplift force due to trapped air under the deck (Chen 2007). The uplift force by trapped air F_u was estimated as

$$F_u = V_{ta} \times w_w \tag{1}$$

where, V_{ta} is trapped air volume per deck and w_w is unit weight of tsunami water. It was assumed that w_w is 10.78 kN/m^3 by considering inclusion of sand and mud in tsunami. Deck weight W_d was evaluated from the design document.

Table 1(a) shows a comparison of uplift force by trapped air F_u and deck weight W_d for the three sections. F_u is not larger than W_d, but F_u becomes closer to W_d as the girder height increases (S1-S2 and S8-S12). Thus it was possible that S8-S10 were uplifted before floated if some additional tsunami hydrodynamic force applied to the decks.

Similarly, the tsunami drag force and the lateral resistance was evaluated bridge spans. The tsunami drag force F_{df} was assumed to be evaluated from hydrodynamic water pressure based on the Design specifications of highway bridges (JRA 2002, Kosa et al 2010) as

$$F_{df} = \frac{1}{2}\rho_w c_d v_w{}^2 A_d \tag{2}$$

in which ρ_w ($\rho_w = w_w / g$), g is the gravity acceleration, c_d is dag coefficient, v_w is tsunami velocity, and A_d is the side area of a deck. The drag coefficient c_d is assumed as 1.4 for a rectangular section based on the Design specifications of highway bridges (JRA 2002). It should be noted that since Eq. (2) represents hydrodynamic force acting to a pier in a river flow, scattering of Eq. (2) must be large. The inclination of decks in the transverse direction was not included in analysis.

On the other hand, since the steel bearings were the most weak link connecting a bridge to the pier cap, the lateral resistance of a span was evaluated based on the design seismic lateral force of bearings in the transverse direction F_{br} as

$$F_{br} = \alpha k_h W_d \tag{3}$$

where k_h is elastic seismic coefficient, W_d is dead weight of a span and α is an over-strength factor for steel bearings. Since Utatsu Bridges was designed in accordance with the Pre-1990 design code, k_h was assumed as 0.25 and the over-strength factor α was assumed to be 2.0.

Table 1(b) shows the tsunami drag force F_{df} and the lateral resistance F_{br} of a span evaluated for three types of girder section. Since the drag force F_{df} is in proportion to the girder height, F_{df} becomes closer to the estimated lateral resistance of a span F_{br} at S1-S2 and S8-S12. On the other hand at S3-S7 which were estimated to be dragged by tsunami based on the field investigation, F_{df} is only 45% of F_{br}. Thus the preliminary evaluation on the flow of spans by Eqs. (2) and (3) provides favorable agreement wit the damage.

Table 1. A preliminary evaluation on the uplift and shear resistance per span

(a) Uplift vs. dead weight

Spans	S1-S2	S3-S7	S8-S12
Trapped air volume V_{ta} (m^3)	400	55	240
Estimated uplift force by trapped air F_u (kN)	4300	580	2500
Deck weight W_d (kN)	5800	1600	3600

(b) Dragged force vs. lateral resistance per span

Spans	S1-S2	S3-S7	S8-S12
Deck height and length (m)	2.5 x 40.7	1.0 x 14.4	1.85 x 29.8
Hydrodynamic force F_{df} (kN)	2560	360	1390
Lateral capacity of bearings F_{br} (kN)	2890	800	1800

5. CONCLUSIONS

Ground-motion-induced and tsunami-induced damage of road bridges in the north Miyagi-ken and south Iwate-ken during the 2011 Great East Japan earthquake was studied. Based on the findings presented herein, the following conclusions may be deduced:

1) Ground-motion-induced damage of bridges which were built in accordance with the post-1990 design code was limited. Thus enhancing the shear and flexural capacity as well as ductility capacity of piers and extensive implementation of elastomeric bearings were effective for mitigating damage of bridges during this earthquake. Since the ground motions during the 2011 Great East Japan earthquake was nearly equal to or smaller than the type II design ground motions, it is the anticipated seismic performance of bridges which were built based on the post-1990 design code. However effectiveness of the measures provided in the post-1990 design codes against stronger than code specified ground motions has to be verified.

2) Bridges which were built in accordance with the pre-1990 code and which were not yet retrofitted suffered similar damage developed during the 1978 Miyagi-ken-oki earthquake. Appropriate seismic retrofit is required for those bridges.

3) Tsunami-induced-damage was extensive to bridges along the Pacific Coast. There were at least two failure mechanisms; 1) simple drag in the transverse direction, and 2) spans uplift before floated. There were no road bridges which suffered damage by scoring. On the other hand, there were a number of bridges which survived tsunami action though they were completely underwater.

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