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DAMAGE OF BRIDGES DUE TO THE 2011 GREAT EAST JAPAN EARTHQUAKE

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ABSTRACT: 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 due to the 1978 Miyagi-ken-oki earthquake so that the effect of 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 video movies.

Key Words: Great East Japan earthquake, seismic damage, bridges, ground motions, tsunami, seismic design code

INTRODUCTION

The Great East Japan earthquake (M_w9.0) occurred at 14:46 (local time) on March 11, 2011 along the Japan Trough in the Pacific Ocean. The fault zone extended 450km and 200km in the north-south and west-east directions, respectively. Extensive damage occurred in a wide region in the east Japan.

As shown later the seismic design codes of bridges were extensively enhanced since 1990. 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. Thus, the Great East Japan earthquake was a valuable occasion to evaluate the effectiveness of recent progress of seismic design codes 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.

It was the first time to have extensive damage to bridges by tsunami in recent years. No single word about tsunami is included in the current design codes for bridges. Of course, extensive damage occurred in the past, but it was probably regarded as unavoidable natural disasters before the World War II. After the World War II, there were tsunami earthquakes, but extensive damage did not occur to transportation facilities.

This paper presents ground-motion-induced damage and tsunami-induced damage of road bridges in the north Miyagi-ken and south Iwate-ken (refer to Fig. 1) during the 2011 Great East Japan earthquake.

Fig. 1 Bridges in the north Miyagi-ken and the south Iwate-ken

BRIEF HISTORY OF JAPANESE SEISMIC DESIGN FOR BRIDGES

For evaluating damage of bridges, it is important to know what design codes were used for constructing bridges which suffered damage. Japanese history of seismic design of bridges since the end of 19 century may be classified into the following four stages.

Stage I

The Stage I corresponds to the days between Meiji Revolution and the end of Second World War II (1886-1945) during which seismic design was not considered or was poorly considered. It was the 1923 Great Kanto Earthquake when Japanese first realized that technologies imported from the European countries were insufficient for mitigating damage of structures due to an earthquake. Obviously the technologies imported from the European countries did not include consideration for the seismic effects. The Great Kanto earthquake was the starting point for Japanese to develop our own seismic design practice. At the Stage I, damage always resulted from settlement, overturning and excessive drift of foundations. Countermeasure for damage and cost saving for the use of expensive steel rebars led to the practice of constructing large and stiff foundations and piers, and this concept became the main stream of seismic design in Japan.

Stage II

The Stage II corresponds to the days of the 1964 Niigata earthquake. Extensive damage of bridges occurred due to liquefaction. Terminology of "liquefaction" came after the Niigata earthquake leading to extensive research for the mechanism of liquefaction. The original concept of "unseating prevention devices" was proposed by Japanese field engineers. They proposed that if adjacent decks were tied together by cables or if a deck was connected to a substructure, a total collapse of bridges could be prevented. This concept was first incorporated in the 1971 Design guidelines for seismic design of bridges (JRA 1971). Now it is widely adopted not only in Japan but also worldwide. There are essentially no bridges in Japan which do not have unseating prevention devices. Various unseating prevention devices are currently used.

Stage III

After the Stages I and II, foundations and piers were strengthened, the effect of soil liquefaction was included in seismic design, and unseating prevention devices were implemented. As a consequence, damage extended in an unexpected direction though the existing damage was mitigated. The days between 1978 and 1995 when we had extensive damage to piers and steel bearings corresponds to Stage III.

During the 1978 Miyagi-ken-oki earthquake (M_{JMA}7.4), extensive damage concentrated to piers and bearings though damage of foundations due to excessive displacement and liquefaction was gradually mitigated. In the 1982 Urakawa-oki earthquake, extensive shear failure occurred at Shizunai Bridge at cut-off of main reinforcements with insufficient development. Obviously damage shifted from the previous weak links to the next weak links. Though the allowable shear strength of concrete was overestimated and the development of longitudinal bars at cut-off points was insufficient, they did not lead to shear failure of piers in the Stages I and II since the concrete section was large. However as population in cities increased, a space limitation under bridges restricted the size of piers in viaducts in urban areas. The same restriction was imposed to river crossing bridges for smoother river flow. Thus, column and pier section had to be reduced in size such that they became flexible piers. Under such a condition, overestimation of the concrete shear capacity and insufficient development at cut-off predominantly contributed to resulting in shear failure in columns and piers.

Steel bearings which accommodate only limited relative displacement between superstructure and substructure suffered extensive damage. Side blocks with poor capacity were always sheared off, and anchor bolts were pulled out from concrete bases in past earthquakes. Because an elastic static analysis based on a 0.15-0.3g elastic static seismic force was used, the seismic force demand for steel bearings was inadequate. There was an argument that weak bearings were fuse to limit an excessive transfer of the inertia force from decks to substructures so that collapse of substructures could be prevented. However damage of weak bearings resulted in excessive drift of superstructures, and repair of steel bearings in a wide damaged region required long down time resulting in suspension of transportation.

The shift of damage to piers and bearings was first noticed in the 1978 Miyagi-ken-oki earthquake and it extensively occurred during the 1995 Kobe earthquake.

Stage IV

The importance of considering the realistic design ground motions and ductility capacity by preventing shear failure for piers and columns was recognized since the 1978 Miyagi-ken-oki earthquake. The first seismic design code which included the design requirement for ductility capacity was issued in 1990 as Part V Seismic design, Design specifications of highway bridges (JRA 1990). An inelastic static analysis as well as the Type I ground motions as shown in Fig. 2 was incorporated in the 1990 code, where the Type I ground motion represents the ground motions which are induced by an M8 subduction earthquake. In design, the ground motions which were possibly developed in Tokyo during the 1923 Great Kanto earthquake is used as the Type I ground motions. As well as the Type II ground motions which was incorporated in the 1995 Guide specifications (JRA 1995) and 1996 code (JRA 1996), the introduction of realistically high intensity ground motions and an inelastic static analysis extensively enhanced the seismic performance of brides designed in accordance with the post-1990 codes. The days after the 1995 Kobe earthquake corresponds to the Stage IV.

Various research for seismic isolation was on going in parallel with the preparation of 1990 design code (TRCNLD 1988, PWRI 1992). The first seismic isolated bridge with use of lead rubber bearings (Miyagawa bridge, Shizuoka-ken) and high damping rubber bearings (Yama-age Bridge, Tochigi-ken) was constructed in 1991 and 1992, respectively. Implementation of elastomeric bearings, LRB and HDB was initiated in the early 1990s (Unjoh et al 2010). Those bearings were implemented not only in seismic isolated bridges but also in multi-span continuous bridges for distribution of the inertia force to piers. The implementation of elastomeric bearings, LRB and HDR extensively mitigated damage of steel bearings during the 2011 Great East Japan earthquake.

Furthermore, a new evaluation analysis for the inertia force for multi-span continuous bridges was incorporated in the 1990 code. Prior to the 1990 code, the lateral force was evaluated by only multiplying a reaction force and a seismic coefficient by disregarding the overall system response. In the 1996 Part V Seismic design (JRA 1996), the Type II ground motions, which represents typical near-field ground motions induced by an M7 event (ground motions developed during the 1995 earthquake), an evaluation for residual displacement (Kawashima, MacRae, Hoshikuma, Nagaya 1998), and an enhancement for lateral force demand for bearings and unseating prevention devices were incorporated (Kawashima 2000, Kawashima 2006a). Thus, the post-1990 design codes (1990 code and 1996 code) contributed to construction of bridges with enhanced seismic performance. As a result, ground-motion-induced-damage of bridges which were designed in accordance with the post-1990 codes was minor as will be described later.

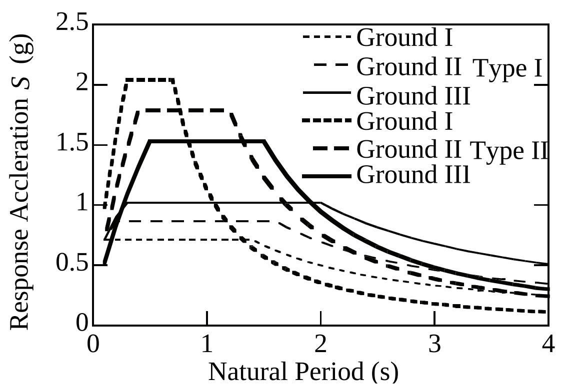


Fig. 2 Type I and II design ground motions

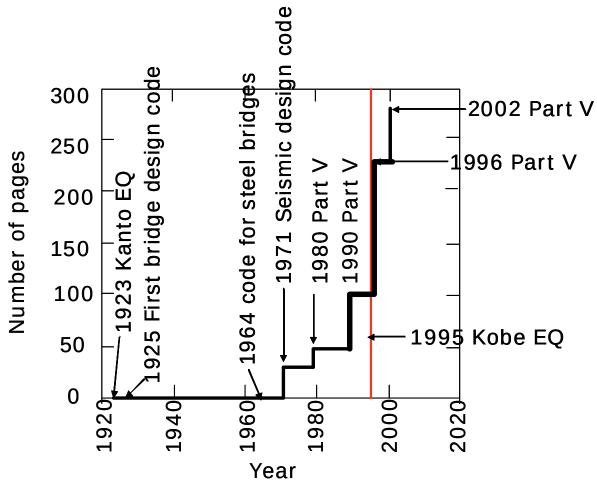


Fig. 3 Progress of seismic design in terms of number of pages related to seismic design

Pre-1990 and post-1990 seismic design codes

Fig. 3 shows the progress of seismic design of bridges in terms of number of pages of design codes which are relate to seismic design. Of course only an increase of number of pages in codes does not lead to better seismic design, but it can be realized how the knowledge on seismic design was accumulated in the past. It should be noted that only a 3-4 page description was provided in the 1964 Design specifications of highway bridges, which was referred to in design of a large number of bridges. Many bridges which collapsed or suffered extensive damage during the 1995 Kobe earthquake were constructed in accordance with the 1964 design code (JRA 1964). A combination of a static elastic analysis and an allowable stress design approach (seismic coefficient method) was used until 1990 (pre-1990 codes). The static elastic method is still used now but a combination of an inelastic static analysis and the Type I and II design ground motions is the main stream in the post-1990 codes. It should be noted that elastic and inelastic dynamic response analyses are conducted on routine basis for bridges with complex structural response after 1995.

Seismic retrofit of existing bridges was conducted for reinforced concrete piers which had cut-offs of longitudinal bars with insufficient development since the 1980s (Kawashima 2006b). Over 30,000 piers were so far retrofitted since 1995. However there still remain a number of piers which require retrofitting. Moreover, seismic retrofit of foundations has be conducted only for few bridges.

GROUND MOTIONS AND DESIGN GROUND MOTIONS

A number of strong motion accelerations were recorded in the damaged areas by the National Institute of Earth Science and Disaster Prevention and Japan Meteorological Agency. Since most of them were recorded on stiff sites, they cannot be directly compared to the Type I and Type II ground motions. Fig. 4 shows some accelerations recorded along the Pacific Coast. Ground accelerations continued over 300s, and had at least two wave groups reflecting the fault rupture process. The highest peak ground acceleration was 27.0 m/s^2 which was recorded at Tsukidate City. However this high acceleration was resulted from a single pulse with high frequency components. The response acceleration at 1.0s period

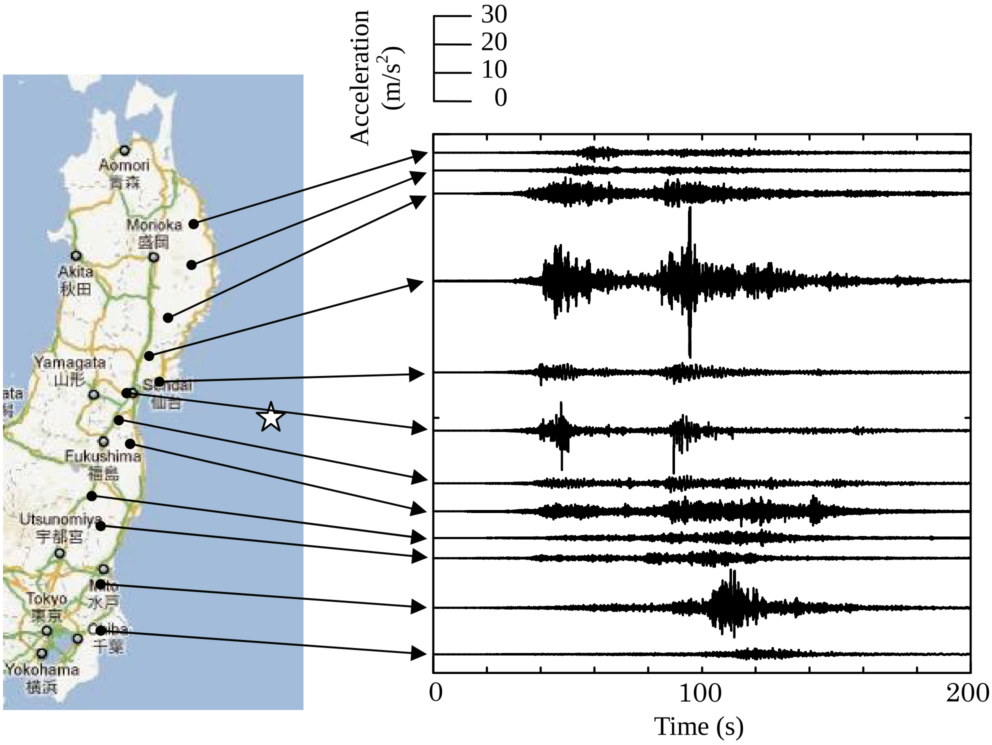


Fig. 4 Accelerations recorded along the Pacific Coast by NIED

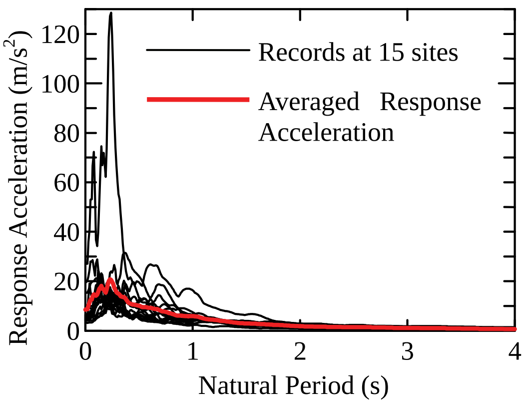
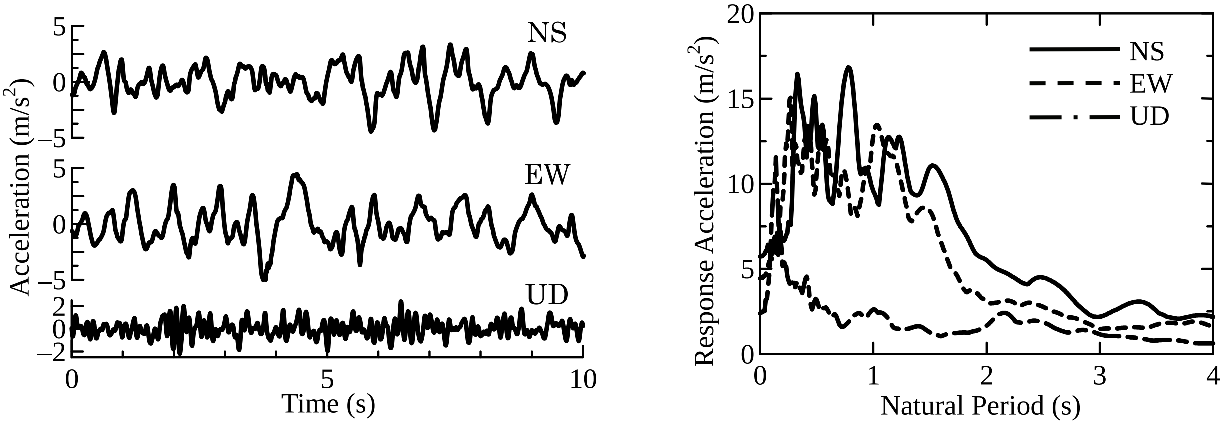


Fig. 5 Response accelerations (ξ =0.05) at 15 sites at the flat land north of Sendai

was only 5.1m/s^2 as shown in Fig. 5. Consequently damage of buildings and bridges was very limited in Tsukidate. This clearly shows that PGA cannot be a reliable index for seismic design.

At soft soil sites in the north Sendai City such as Osaki City, Tome City, Wakuya City and Ichinoseki City, ground accelerations having higher response accelerations at 0.5-1.5s period range were recorded. In particular, the response acceleration was slightly over 16 m/s^2 at Osaki as shown in Fig. 6. Thus it is considered that the response accelerations at high intensity areas during the 2011 Great East Japan earthquake was close to but smaller than the Type II design ground motions.



(a) Accelerations during 10s around the peak values (b) Response accelerations

Fig. 6 Acceleration record at Furukawa, Osaki City (K-NET)

DAMAGE DUE TO GROUND MOTIONS

Damage of bridges which were constructed in accordance with the pre-1990 codes

Extensive damage occurred at the bridges which were designed in accordance with the pre-1995 design code and not yet 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 was resulted from an overestimated shear capacity and an inadequate development of longitudinal bars at cut-off which were the common practice in the pre-1980 design codes. This mode of damage occurred extensively during the 1995 Kobe, Japan earthquake (Kawashima and Unjoh 1997). Extensive investigation was directed to clarify the failure mechanism of such damage (for example, Kawashima, Unjoh and Hoshikuma 1995), including a series of large scale shake table experiments using E-Defense (Kawashima et al 2009). 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 (Akimoto et al 1990), 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, but damage still continued to occur at the bridges which were not yet 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 using reinforced concrete jacketing, they did not suffer damage again. 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

Photo 1 Flexure-shear failure of columns due to termination of longitudinal bars with insufficient development (Fuji Bridge) (courtesy of Dr. 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

to failure by allowing no relative displacements at pin bearings and relative displacements accommodated in 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 again as shown in Photo 4. It was a critical zone due to concentration of seismic force, dead load reaction and PC anchor force.

Tennoh Bridge built in 1959 which suffered extensive damage during the 1978 Miyagi-ken-oki earthquake suffered extensive damage again during the 2011 Great East Japan earthquake at the same

(a) 1978 Miyagi-ken-oki earthquake

(b) 2011 Great East Japan earthquake

Photo 4 Damage of a PC box girder bridge at the end support, Yuriage Bridge

Photo 5 Rupture and buckling of upper braces, Tennoh Bridge, National Road No. 45

(a) rupture of a lower brace

(b) Disconnection at the end of a lower brace

Photo 6 Rupture of a lower brace and lost of connection at the end, Tennoh Bridge

members. Photos 5 and 6 show rupture and local buckling of upper and lower braces. An end of a lower truss brace with a gusset plate was completely disconnected due to corrosion. Because similar disconnection was observed at other lower braces, it is likely that a large torsion response of the truss bridge due to deterioration of torsional rigidity resulted in extensive rupture and buckling of upper and lower braces. This truss bridge was critical for collapse during the earthquake.

Performance of bridges which were retrofitted

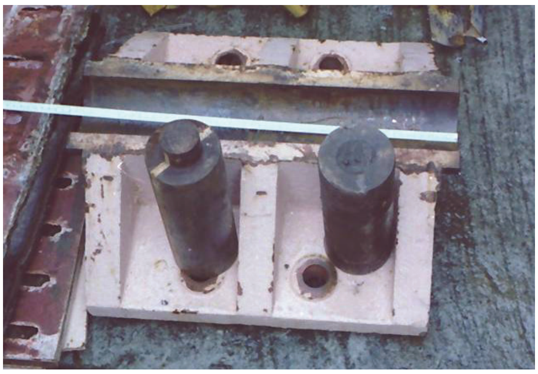
Damage of bridges which were already retrofitted suffered virtually no damage. For example, Sendai Bridge which is an extremely important bridge in Sendai City suffered extensive damage at reinforced concrete piers and steel bearings as shown in Photo 7(a) during the 1978 Miyagi-ken-oki earthquake.

(a) 1978 Miyagi-ken-oki earthquake

(b) 2011 Great East Japan earthquake

Photo 7 Effect of seismic retrofitting of piers, Chiyoda Bridge, National Road No. 4

Photo 8 Elastomeric bearings which were set for replacement of original steel bearings suffered no damage, Sendai Bridge



(a) Rupture of a pin in a pin bearing

(b) Pull-out of anchor bolts due to rocking response of a lower bearing

Photo 9 Damage of steel bearings during the 1978 Miyagi-ken-oki earthquake, Shin-Iino Bridge, National Road No. 45

However this bridge suffered no damage during the 2011 Great East Japan earthquake, because columns were retrofitted as shown in Photo 7(b) and the original steel bearings were replaced with elastomeric bearings as shown in Photo 8.

Shin-Iino-gawa Bridge suffered extensive damage at steel pin and roller bearings as shown in Photo 9 during the 1978 Miyagi-ken-oki earthquake. It was retrofitted prior to the 2011 Great East Japan earthquake: 1) some reinforced concrete piers were retrofitted using steel jacketing, 2) nonlinear viscous dampers were installed, and 3) steel bearings were replaced with elastomeric beatings as shown in Photo 10. As a result, the bridge suffered no damage during the 2011 Great East Japan earthquake.


(a) An elastomeric bearing without damage


(b) A damper for seismic retrofit

Photo 10 Elastomeric bearings and viscous dampers which were set for seismic retrofit did not suffer damage during the 2011 Great East Japan earthquake

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 during the Great East Japan earthquake. For example, Photo 11 shows Shin-Tenno Bridge constructed in 2002 suffered no damage. Elastomeric bearings and new cable restrainers which satisfy the requirements of the post-1990 design code were set. This bridge was located only 200m upstream of Tenno Bridge which suffered extensive damage during the Great East Japan earthquake (refer to Photos 5 and 6).

Photo 12 shows Higashi-Matsushima Bridge which was constructed in 2007. No damage occurred in this bridge.


(a) Bridge after Great East Japan Earthquake


(b)Elastomeric bearing without damage

Photo 11 Shin-Tennoh Bridge

(a) General view

(b) Deck end supported by three elastomeric bearings

Photo 12 Higashi-Matsushima Bridge

Elastomeric bearings generally performed quite well under the extreme ground motions as shown above. However it should be noticed that elastomeric bearings ruptured in some bridges. For example, several elastomeric bearings ruptured such that a deck offset by 0.5m in the transverse direction as shown in Photo13(a) at Sendai-Tobu Viaduct. Rubber layers detached from steel plates in addition to rupture of rubber layers as shown in Photo 13(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 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 (Quan and Kawashima 2009).

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

(b) Rupture of an elastomeric bearing

Photo 13 Damage of elastomeric bearings at Sendai Tobu Expressway

TSUNAMI INDUCED DAMAGE

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. Smaller and shorter span bridges which were built in the early days were generally vulnerable to tsunami effect. Bridges which were taller than tsunami waves did not suffer damage. Very short and short span bridges generally suffered less damage by tsunami probably because tsunami wave front did directly hit bridges and they were well constraint by abutments.

Back fills and embankment were eroded and washed away at many bridges. Though repair of back fills and embankment is easier than repair of bridge structures, protection should be considered in the future.

Utatsu bridge

Utatsu Bridge built in 1972 at Minami-sanriku Town over Irimae Bay suffered extensive damage by tsunami as shown in Photo 14. 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 as shown in Fig. 7. Diaphragms were set between PC girders at the ends and mid points. The girders had an inclination as large as 6% in the transverse direction due to curved alignment of the bridge. For example, the inclination was 4.8%, 3%, 3%, 1.1% (sea side down) at D3, D4, D5, and D6, respectively, then it changed to 2-5%, 9%, 4% and 2% (sea side up) at D8, D9 & D10, D11, and D12, respectively.

The bridge was seismically retrofitted a few years ago. Columns were retrofitted by steel jacketing and unseating prevention devices were installed. The decks D1, D2, D11 and D12 remained but Decks D3-D10 were washed away. Decks D3-D7 were simply supported pre-tensioned PC girder bridges. As a part of the seismic retrofit, cable restrainers were set for tying together between D3-D7, and three steel stoppers were provided at the end of D3 and D7 for preventing excessive longitudinal deck movements. Though cable restrainers ruptured between D4 and D5, D3-D4 and D5-D6-D7 were still tied together after floated as shown in Photo 15. D8, D9 and D10 overturned when they were floated as shown in Photo 16.

Photo 14 Utatsu Bridge (Google)

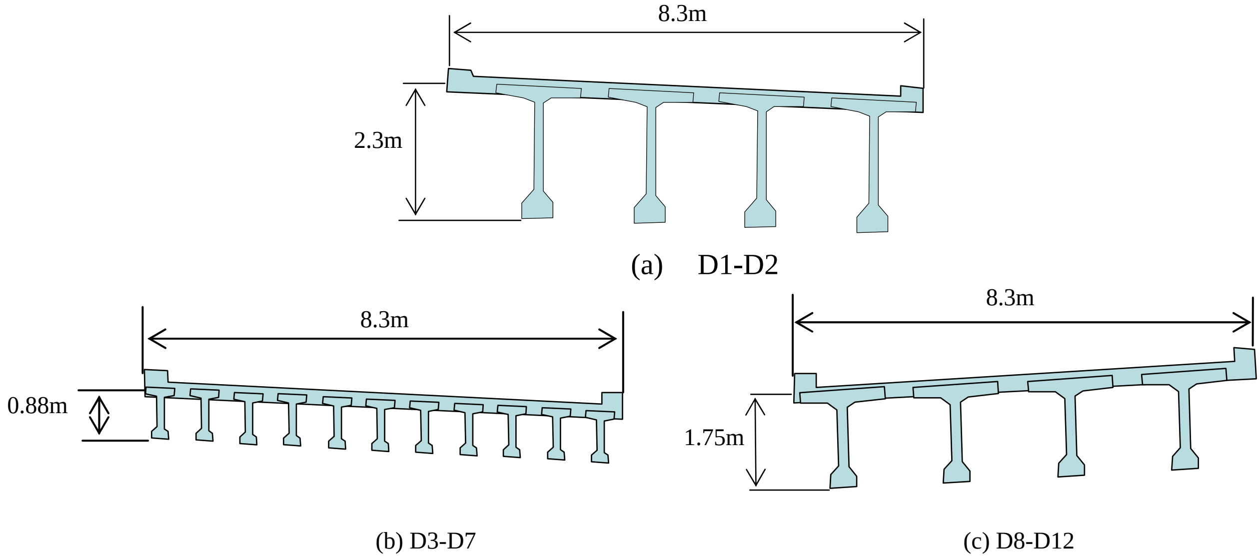


Fig. 7 PC Deck Sections, Utatsu Bridge (the left and right correspond to the land side and sea side, respectively, when viewed from Sendai side)

Only P2 suffered flexural failure as shown on Photo 17(a) at the land side. The column was retrofitted using a steel jacket. The damage was probably caused by ground-motion-induced seismic force based on an analysis of the moment capacity of retrofitted section. Since new ties in the jacket were flare welded, the ties were still confining the column though they yielded. D2 did not suffer damage and three stoppers for D2 on P2 were intact. On the other hand, three stoppers for D3 on P2 suffered damage as shown in Photo 17(b). This indicates that D3 offset when D3 was dragged laterally due to tsunami.

Photo 18 (a) shows two steel longitudinal stoppers on Abutment 2. The main function of the stoppers was to prevent excessive deck displacement in the longitudinal direction, but they also restricted deck displacement in the transverse direction. Photo 18 (b) and (c) shows three stoppers and four pot bearings on P10. Three longitudinal stoppers did not suffer damage at all. All upper bearings were detached from the lower bearings and floated away together with D10. 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 two lower bearings were in their original position without damage. From the fact that three stoppers neither suffered damage nor tilted, it was likely that D10 was uplifted over the stoppers before floated.

Photo 15 Decks D5-D7

Photo 16 Overturned Deck D8

(a) P2 from D2 side

(b) P2 from D3 side

Photo 17 Flexural failure of P2

Photo 18(d) shows three steel longitudinal stoppers and four seat extenders for D9 on P9, and Photo 18 (e) shows three stoppers for D8 on P7. The stoppers for D8 and D9 neither tilted nor suffered damage. This also indicates that D8 and D9 were first uplifted before they were floated. Photo 18 (f) shows four stoppers for D7 on P7. The land side stopper slightly tilted with other three stoppers being not damaged at all. It indicates that D7 (one of the shortest deck) was uplifted toward the land side at the sea side but insufficiently at the land side before it was floated. Other shorter span decks (D2-D6) were simply dragged laterally.

A video was taken from A2 by a local resident. This video shows a whole process of rising tsunami water level until the bridges was completely covered by tsunami as shown in Photo 19. Since the bridge failed after it was completely covered by tsunami, it is not known when the decks were uplifted and floated. Since tsunami flow was not fast at both sides of the bay, this saved D1-D2 and D11-D12 from collapse. Tsunami flow velocity was about 6m/s.

Fig. 8 shows possible failure mechanisms due to tsunami. As mentioned earlier, overturning of foundations due to scouring did not occur. The above mentioned damage indicates that Utatsu Bridge suffered damage due to the mechanism of (b) and (c).

If decks uplift before floated under tsunami action, it may be effective to install restrainers in the vertical direction between decks and substructures so that decks can be tied down to substructures (tsunami unseating prevention devices). 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 upward uplifting force is limited, such a strengthening is not generally required for most bridges unless bridges were not very old.

(a) Longitudinal and transverse stopper at A2

(b) P10

(c) Steel bearings and stoppers on P10

(d) Steel stoppers and seat extenders on P9

(e) P7 from D8 side

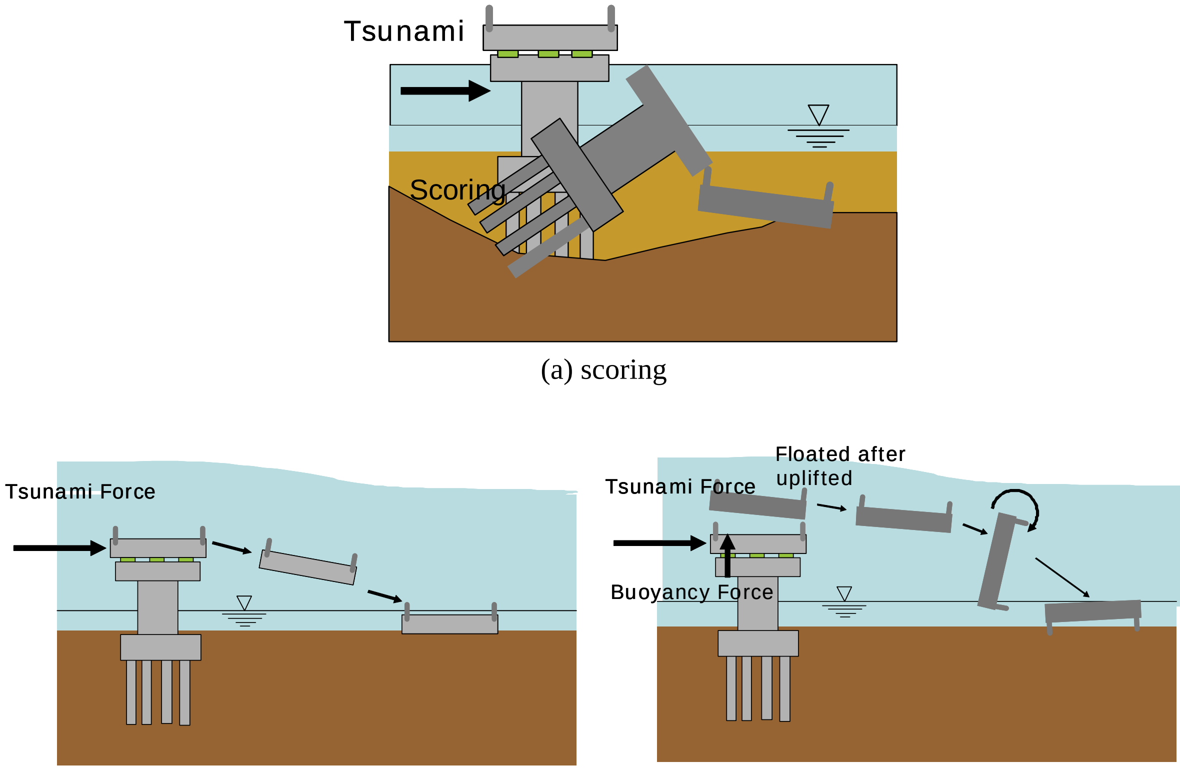
(f) P7 from D7 side

Photo 18 Damage of stoppers

(a) Tsunami reached the bottom of decks

(b) The bridge was completely covered by tsunami

Photo 19 Video taken by a local resident near A2



(a) scoring

(b) Transverse drag

(c) Floated after uplifted

Fig. 8 Possible damage mechanisms by tsunami

Bridges which survived tsunami

There are a number of bridges which survived tsunami though they were completely covered by tsunami. For example, Yanoura Bridge on National Road No. 45 in Kamaishi City was a 108.6m long three span simply supported curved steel deck girder bridge as shown in Photo 20. It crossed Koshi River. Pile foundations were used for two abutments and two piers. Ten 30m long and 1m diameter cast in place piles were driven to support the footings. A video was taken from Kamaishi Port Office which was located at the left bank 140m from the bridge as shown in Photo 21. A Toyota rent-a-car office and Ozawa Building were located near the bridge at the left and right bank, respectively.

At about 15:00, the first tsunami attacked the bridge as shown in Photo 22. Tsunami reached nearly the top of an entrance gate of Toyota rent-a-car office. Since the tsunami reached mid-height of the second story of Ozawa Building, it is evaluated that the maximum covering depth of tsunami above the bridge surface was about 5m. It is noted from the video that few debris was included in tsunami which hit the bridge because this bridge was located at the mouth of Koshi River.

As shown in Photo 20, Yanoura Bridge suffered only minor damage on hand rails.

Photo 20 Yanoura Bridge, National Road 45, Kamaishi City

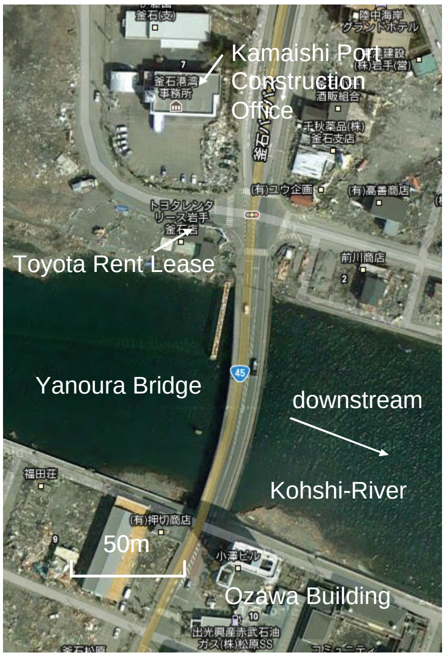


Photo 21 Location of Yanoura Bridge, Kamaishi Port Office, Toyota rent-a-car office and Ozawa Building

(a) Tsunami almost covered Yanoura Bridge

(b) Yanoura Bridge completely covered by tsunami

Photo 22 Tsunami attack to Yanoura Bridge

A preliminary evaluation of uplift and floating of Utatsu Bridge

A preliminary analysis was conducted to evaluate possible uplift of decks of Utatsu Bridge. As shown in Photo 16, since the PC girder decks had lateral diaphragms at both ends and mid spans, they were vulnerable to uplift 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 sand and mud included in tsunami water. 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 three sections shown in Fig. 6. F_u is not larger than W_d, but F_u becomes closer to W_d as the deck height increases (D1-D2 and D8-D12). Thus it is considered possible that D8-D10 were uplifted before floated if some uplift force due to tsunami additionally applied to the decks.

Similarly, tsunami drag force vs. lateral resistance of a deck was evaluated. 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, Shoji et al 2009) 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 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, an accuracy of Eq. (2) for representing hydrodynamic force due to tsunami for a deck is not verified. The inclination of decks in the transverse direction was not considered in analysis.

On the other hand, the lateral resistance of a deck was evaluated from the design seismic lateral force of a deck in the transverse direction F_{br} as

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

where k_h is elastic seismic coefficient used in design, W_d is dead weight of a deck 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. 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 deck evaluated for three types of deck. Since the drag force F_{df} is in proportion to the deck height, F_{df} becomes closer to the estimated lateral resistance of decks F_{br} at D1-D2 and D8-D12. However F_{df} is only 45% of F_{br} at D3-D7 which were probably dragged by tsunami based on the field investigation. More precise evaluation for tsunami effect is required.

Table 1 A preliminary evaluation for deck uplift and shear resistance

(a) Uplift vs. dead weight

Decks	D1-D2	D3-D7	D8-D12
Trapped air volume per deck V_{ta} (m^3)	400	55	240
Estimated uplift force by trapped air per deck F_u (kN)	4300	580	2500
Deck weight W_d (kN)	5800	1600	3600

(b) Dragged force vs. lateral resistance

Decks	D1-D2	D3-D7	D8-D12
Deck height and length (m)	2.5 x 40.7	1.0 x 14.4	1.85 x 29.8
Hydrodynamic force per deck F_{df} (kN)	2560	360	1390
Lateral capacity of bearings per deck F_{br} (kN)	2890	800	1800

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 presented. Based on the findings presented herein, the following conclusions may be tentatively deduced:

1) Ground-motion-induced damage of bridges which were built in accordance with the post-1990 design code was very 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 during this earthquake. Since the ground motions during the 2011 Great East Japan earthquake was nearly equal or smaller than the type II ground motions, it is the expected level of seismic performance. However effectiveness of the measures provided in the post-1990 design codes against stronger than code specified ground motions has to be verified.

2) On the other hand, 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 bridges which were uplifted before floated. On the other hands, there were a number of bridges which survived though they were completely covered by tsunami. Tsunami effect has to be studied so that it can be included in design in the future.

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