

## Damage of Bridges during 2011 Great East Japan Earthquake

by

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### ABSTRACT

This paper presents damage of road and railway bridges during the Great East Japan earthquake and tsunami on March 11, 2011 based on a JSCE damage investigation. Ground motion induced damage and tsunami induced damage of both road and railways bridges are presented.

**KEYWORDS:** Great East Japan Earthquake, Damage Investigation, Seismic Damage, Bridges, Tsunami, Seismic Design, Seismic Retrofit

### 1. INTRODUCTION

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

The authors were dispatched by Japan Society of Civil Engineers for field damage investigation to bridges in Miyagi-ken and Iwate-ken between March 29-April 3, 2011. In addition to the first investigation, damage investigations were conducted several times. Since 1978 Miyagi-ken-oki earthquake and 2003 Sanriku-Minami earthquake affected this region, an emphasis was placed in the damage investigation to compare damage among 2011 Great East Japan earthquake and two previous earthquakes. This paper presents ground motion induced damage and tsunami induced damage of bridges during 2011 Great East Japan earthquake.

### 2. GROUND MOTIONS AND TSUNAMI

#### 2.1 Ground Motions

A number of strong motion accelerations were recorded by the National Institute of Earth Science and Disaster Prevention and Japan Meteorological Agency. Fig. 1 shows measured accelerations along the Pacific coast. Ground accelerations continued over 300s, and had at least two groups reflecting the fault rupture process. The highest peak ground acceleration of  $27.0 \text{ m/s}^2$  was recorded at Tsukidate. However the high acceleration was resulted from a single pulse with high frequency components, and the response acceleration at 1.0 s was only  $5.1 \text{ m/s}^2$ . Damage of buildings and other infrastructures was minor in Tsukidate.

Fig. 2 shows acceleration response spectra at 15 sites in the flat region north of Sendai. It is general trend that high frequency components were predominant in the measured accelerations. However Fig. 3 shows ground accelerations and response accelerations of the records at Furukawa where soil condition is very weak such that the shear wave velocity is 80m/s at 2m thick top soil and 120 m/s between 2 m and 17 m below the ground surface. The response accelerations in the lateral components were nearly  $15 \text{ m/s}^2$  at period between 0.2 s and 0.8 s, and  $3\text{-}5 \text{ m/s}^2$  at 2 s period.

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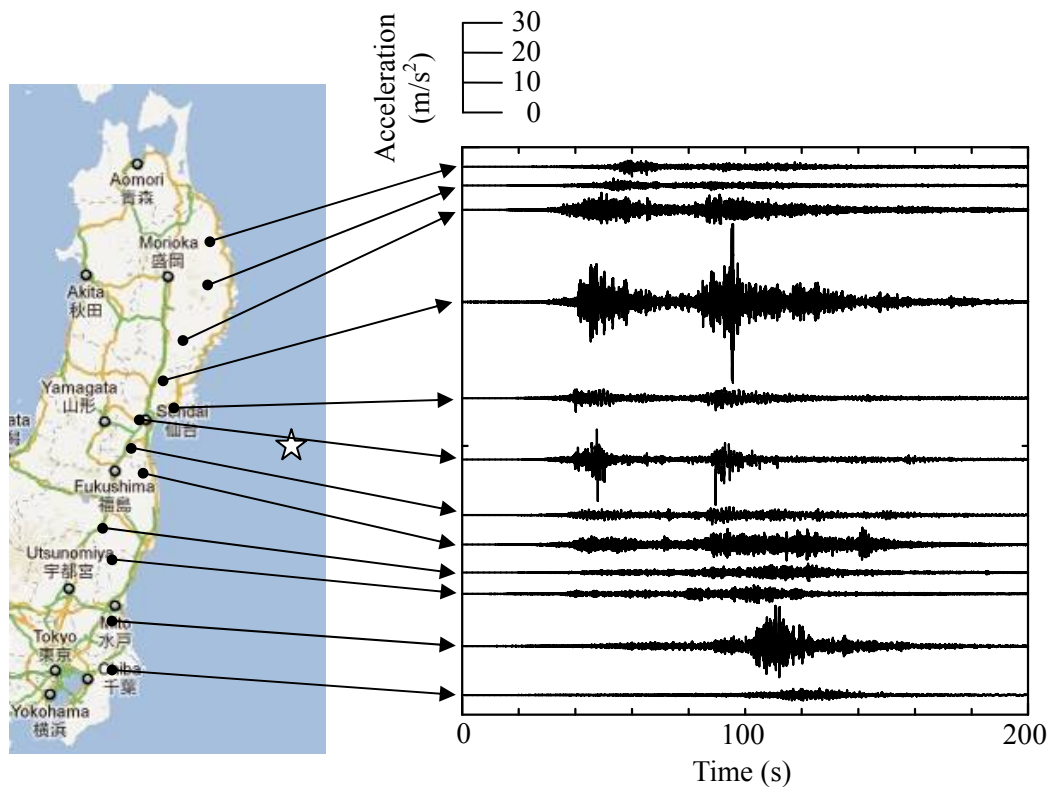


Fig. 1 Accelerations recorded by NIED

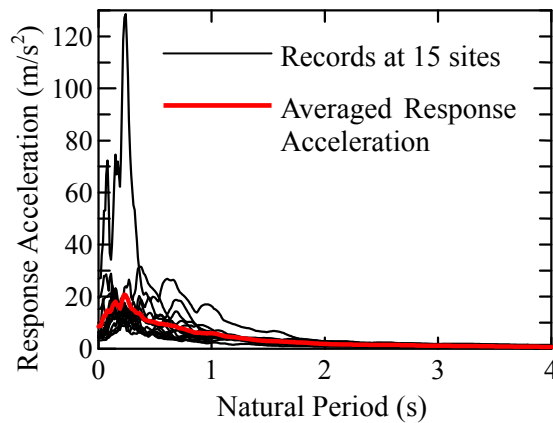


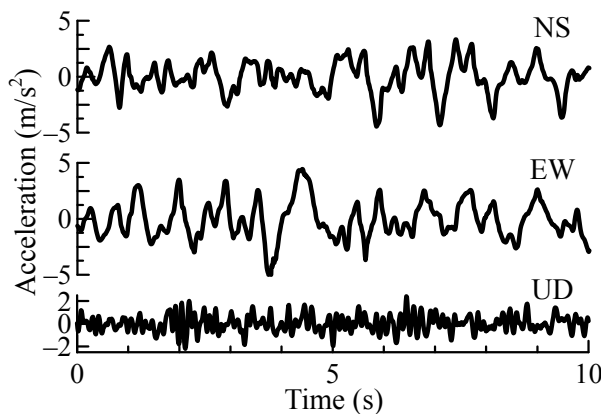
Fig. 2 Response accelerations ( $\xi=0.05$ ) at 15 sites at the flat land north of Sendai City

Ground accelerations with similar trend were recorded at other soft soil sites such as K-NET Ichinoseki and Sendai and JMA Tome and Wakuya.

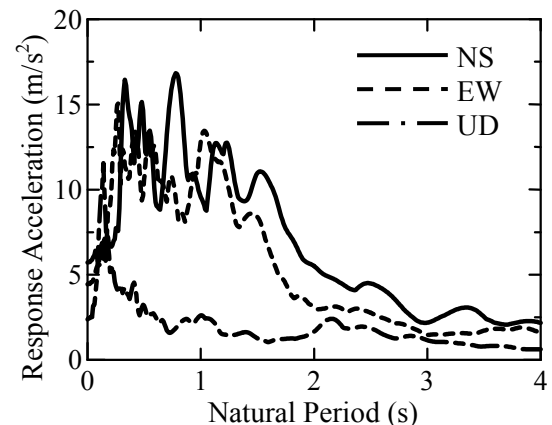
## 2.2 Tsunami

Tsunami attacked coastal region as soon as 30 minutes after the earthquake. Tsunami inundation

area reached as far as 10 km inland, engulfing virtually everything including peoples and structures. Japan Meteorological Agency (JMA) tidal stations recorded high tsunami at many locations. The highest tsunami was recorded 8.5m at Miyako. However due to saturation of the instrument, it must be higher than 8.5m. Other recorded heights at JMA stations include over 8.0 m in Ofunato and over 7.3 m in Soma. It



(a) Accelerations during 10 s around the peak values



(b) Response accelerations

Fig. 3 Acceleration record at Furukawa, Osaki City (K-NET)

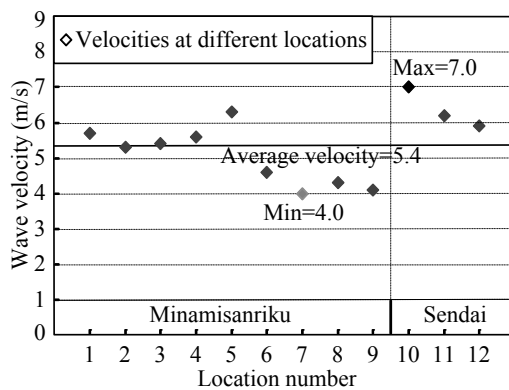


Fig. 4 Estimated tsunami particle velocity

is estimated that real tsunami height was as high as 15m at Onagawa fishery port.

For evaluating tsunami actions to bridges, it is important to know tsunami flow velocity. For this purpose, 12 videos which recorded tsunami flow in Minami-Sanriku Town and Sendai City were analyzed. Particle velocity was estimated based on the time required for a piece of debris to flow between two distinguished points. The distance between the two points and the time were measured using Google Earth's distance measurer and the video's timer, respectively.

Fig. 4 shows tsunami particle velocities evaluated at 12 locations. The maximum, average and the minimum tsunami particle velocities were 7.0m/s, 5.4m/s and 4.0m/s, respectively.



Photo 1 Shear failure of a column due to termination of longitudinal bars with insufficient development (Fuji Bridge) (courtesy of Dr. Hoshikuma, J., PWRI)

### 3. GROUND MOTION INDUCE DAMAGE OF ROAD BRIDGES

#### 3.1 Damage of Bridges which were not yet Retrofitted

Ground motion induced damage of road bridges was generally less significant. However extensive damage occurred at the bridges which were designed according to the pre-1990 design codes (JRA 1990) and were not yet retrofitted in accordance with the post-1990 design codes. For example, Photo 1 shows shear failure of a reinforced concrete column resulted from insufficient development at cut-off of longitudinal bars. This mode of damage occurred



Photo 2 Yuriage Bridge



Photo 3 Damage of pin and roller bearings

extensively in the 1995 Kobe, Japan earthquake [Kawashima and Unjoh 2007]. Extensive investigation was directed to clarify the failure mechanism of such columns [for example, Kawashima, Unjoh and Hoshikuma 2005], including a project consisting of series of large scale shake table experiments using E-Defense [Kawashima et al 1999]. Over 30,000 columns were so far retrofitted since 1995 Kobe earthquake. Consequently, during this earthquake, damage due to this mechanism was not predominant in the bridges which were retrofitted, but damage occurred at the bridges which were not yet appropriately retrofitted.

Yuriage Bridge as shown in Photo 2 suffered extensive damage at reinforced concrete hollow and solid columns, supports of prestressed concrete girders, and steel pin and roller bearings during 1978 Miyagi-ken-oki earthquake. Since the damaged columns were repaired and strengthened by reinforced concrete jacketing, they did not suffer damage this time. However pin and roller bearings suffered damage again in



Photo 4 Damage of PC girders near the support



Photo 5 Buckled truss braces

the similar way as shown in Photo 3. It is obvious that pin and roller bearings are vulnerable to seismic action, because the stress builds up until failure by allowing virtually no relative displacement at pin bearings and relative displacement accommodated in roller bearings is insufficient to realistic relative displacement developed under a strong excitation.

Furthermore, the same supports of prestressed concrete girder which suffered damage in 1978 Miyagi-ken-oki earthquake suffered again as shown in Photo 4. Taking account of likely concentration of seismic force at this region and the importance of anchorage of PC cables, more rigorous repair had have to be conducted after 1978 Miyagi-ken-oki earthquake.





Photo 6 Damage of a pier during 1978 Miyagi-ken-oki earthquake (Sendai Bridge)



Photo 7 Retrofitted pier of Sendai Bridge which did not suffer damage during 2011 Great East Japan earthquake

Tennoh Bridge built in 1959 suffered extensive damage during 1978 Miyagi-ken-oki earthquake. This bridge suffered extensive damage again during this earthquake at the same members; truss braces and pin and roller bearings as shown in Photo 5.

On the other hand, damage of bridges which were already retrofitted suffered virtually no damage. For example, Sendai Bridge which is a symbolic bridge in Sendai suffered extensive damage at columns and bearings as shown in Photo 6 during 1978 Miyagi-ken-oki earthquake. However this bridge suffered no damage during this earthquake, because columns were retrofitted as shown in Photo 7 and steel bearings were replaced with elastomeric bearings.

### 3.2 New Bridges Constructed by Post-1990 Codes

Since 1990, the seismic design code was extensively upgraded [JRA 1990]. Before 1990, only elastic static and dynamic analysis was used assuming unrealistically small seismic design force. However after 1990, inelastic static and dynamic analyses based on Type I/Level 2 design ground motions (middle-field ground motions by M8 events) and an evaluation method of inertia forces considering multi-span continuous effect were introduced, and introduction of those provisions much enhanced the ductility capacity of columns and the seismic performance of bridges. Furthermore in the 1996 code [JRA 1996, Kawashima 2000] which was revised taking account of damage experience of 1995 Kobe earthquake, the Type II/ Level 2 design ground motions (near-field ground motions by M7 events), seismic isolation and use of elastomeric bearings were incorporated. Furthermore, strength of unseating prevention devices was enhanced.

As a consequence of the upgrading of seismic measures, damage of bridges which were built or were retrofitted in accordance with the post-1990 design codes suffered essentially no damage during this earthquake.

For example, Photo 8 shows Shin-Tenno Bridge which was constructed in 2002 suffered no damage. This bridge was located 150 m upstream of Tenno Bridge which suffered damage during 1978 Miyagi-ken-oki earthquake and suffered damage again at almost the same components. Photo 9 shows an end of girder at the left bank where it was supported by elastomeric bearings. New cable restrainers which satisfy the requirements by the post-1990 design code are set. No damage occurred in this bridge.

Elastomeric bearings including lead rubber bearings and high damping rubber bearings performed much better than vulnerable steel bearings. However it should be noticed that elastomeric bearings ruptured in several bridges. For example, at Sendai-Tobu viaduct as shown in Photo 10, several elastomeric bearings ruptured



Photo 8 Shin-Tenno Bridge



Photo 9 An end of deck supported by elastomeric bearings and unseating prevention devices

such that the deck offset in the transverse direction and settled aside the ruptured bearings as shown in Photo 11. Rubber layers detached from steel plates as well as rupture inside rubber layers as shown in Photo 12. Since extensive number of elastomeric bearings including high damping rubber bearings and lead rubber bearings are used, the damage should be critically investigated. It is pointed out that one of the possible reasons for the damage is that the interaction between adjacent bridges with different natural periods was not properly considered in design of elastomeric bearings. Since an expansion joint constrained relative displacement between adjacent decks in the transverse direction, it is likely that larger displacement demand of an adjacent deck is imposed to the elastomeric bearings which were designed based on smaller displacement demand [Quan and Kawashima 2009].



Photo 10 Piers where elastomeric bearings ruptured (Sendai-Tobu viaduct)



Photo 11 Transverse offset of a girder due to rupture of elastomeric bearings



Photo 12 One of elastomeric bearings ruptured

#### 4. GROUND MOTION INDUCED DAMAGE OF SHINKANSEN VIADUCTS

##### 4.1 Seismic Retrofit Program of Shinkansen

Tohoku Shinkansen started the service in 1982 between Omiya and Morioka Stations. Since Shinkansen viaducts were designed prior to the



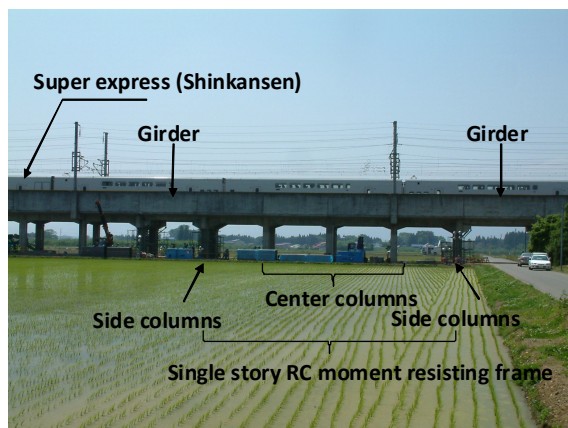


Photo 13 Single story RC moment resisting frame pier with a Gerber girder at both sides



Photo 14 Shear failure of RC columns at Odaki viaducts after 2003 Sanriku-Minami earthquake

occurrence of 1978 Miyagi-ken-oki earthquake, they have smaller amount of shear reinforcements than required by the current code. Some viaducts of Tohoku Shinkansen between Morioka and Mizusawa-Esashi stations in Iwate-ken were extensively damaged during 2003 Sanriku-Minami earthquake [JSCE 2004].

It should be noted that all viaducts which suffered damage during 2011 Great East Japan earthquake had not yet been retrofitted. Most viaducts in Iwate-ken were single story RC moment resisting frame with a Gerber girder at both sides. Damage concentrated at the side columns during 2003 Sanriku-Minami earthquake as shown in Photo 13. Since the side columns were shorter than the center column, a parameter  $\alpha$  defined as a ratio of the shear capacity to the flexural capacity was smaller in the side columns than the center columns, which



Photo 15 No. 3 Odaki viaducts which were retrofitted after 2003 Sanriku-Minami earthquake performed well during 2011 Great East Japan earthquake

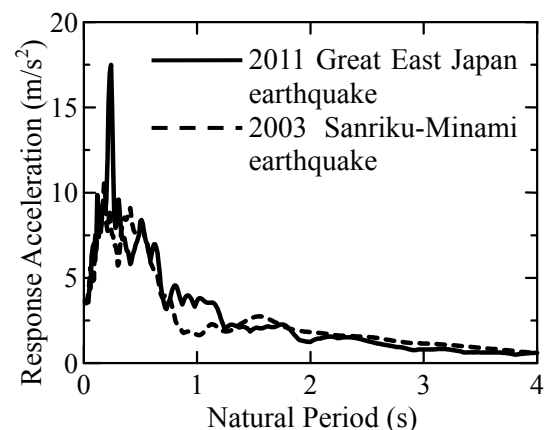


Fig. 5 Response accelerations in 2003 Sanriku-Minami earthquake and 2011 Great East Japan earthquake



Photo 16 Damage of R7 column, No. 1 Nakasone viaduct during 2011 Great East Japan earthquake



Photo 17 Close view of damage of R7-1 and R7-2 columns

led shear failure in the side columns.

After 2004 Niigata-ken Chuetsu earthquake [Huyck et al 2006], the first seismic retrofit program was initiated for Shinkan-sen viaducts including Tohoku Shinkansen. Objectives of the program were to enhance the seismic performance of the columns with insufficient shear capacity. After retrofitting 12,500 columns, the program was completed by 2007. In 2009, the second retrofit program for enhancing columns flexural capacity was initiated.

#### 4.2 No. 3 Odaki Viaducts

No. 3 Odaki viaducts in Iwate-ken failed in shear as shown in Photo 14 during 2003 Sanriku-Minami earthquake. The viaducts were retrofitted so that they had sufficient shear capacity under the first retrofit program after 2004 Niigata-ken Chuetsu earthquake. The retrofitted columns of the viaducts performed well with almost no damage during 2011 Great East Japan earthquake as shown in Photo 15.

Fig. 5 compares the 5% damping response accelerations between 2003 Sanriku-Minami earthquake and 2011 Great East Japan earthquake. The records were measured at approximately 3km from No. 3 Odaki viaducts. Since the fundamental natural period of a single story RC rigid frame ranges between 0.4s to 0.6s,

it is reasonable to considered that the response acceleration of No. 3 Odaki viaducts was nearly the same between 2003 Sanriku-Minami earthquake and 2011 Great East Japan earthquake. It is considered that the seismic retrofit for No. 3 Odaki viaducts was effective for preventing significant damage during this earthquake.

#### 4.3 No. 1 Nakasone Viaducts

No. 1 Nakasone Viaducts, constructed in 1978, is located between Kitakami and Shin-Hanamaki Stations. They had the similar structural shape with the No. 3 Odaki viaducts. No columns were retrofitted during the first seismic retrofit program since it was evaluated that the parameter  $\alpha$  was not small enough.

During 2011 Great East Japan earthquake, side columns suffered extensive damage as shown in Photo 16. All columns lost even the bearing capacity for vertical load. It was fortunate enough not to totally collapse because the viaduct was supported by eight columns. Side columns in other viaducts also suffered extensive damage. Shear failure occurred at the upper part of columns as shown in Photo 17. The damage was so extensive that original shear cracks could not be identified because of crash and spill out of the core concrete.



Table 1 Numbers of bridges which were washed away by tsunami

Types of bridges	Number of bridges washed away
Railways	101
National roads	9
Prefectural roads	14
City or town roads	More than 200
Total	324



Photo 18 Damage of Utatsu Bridge

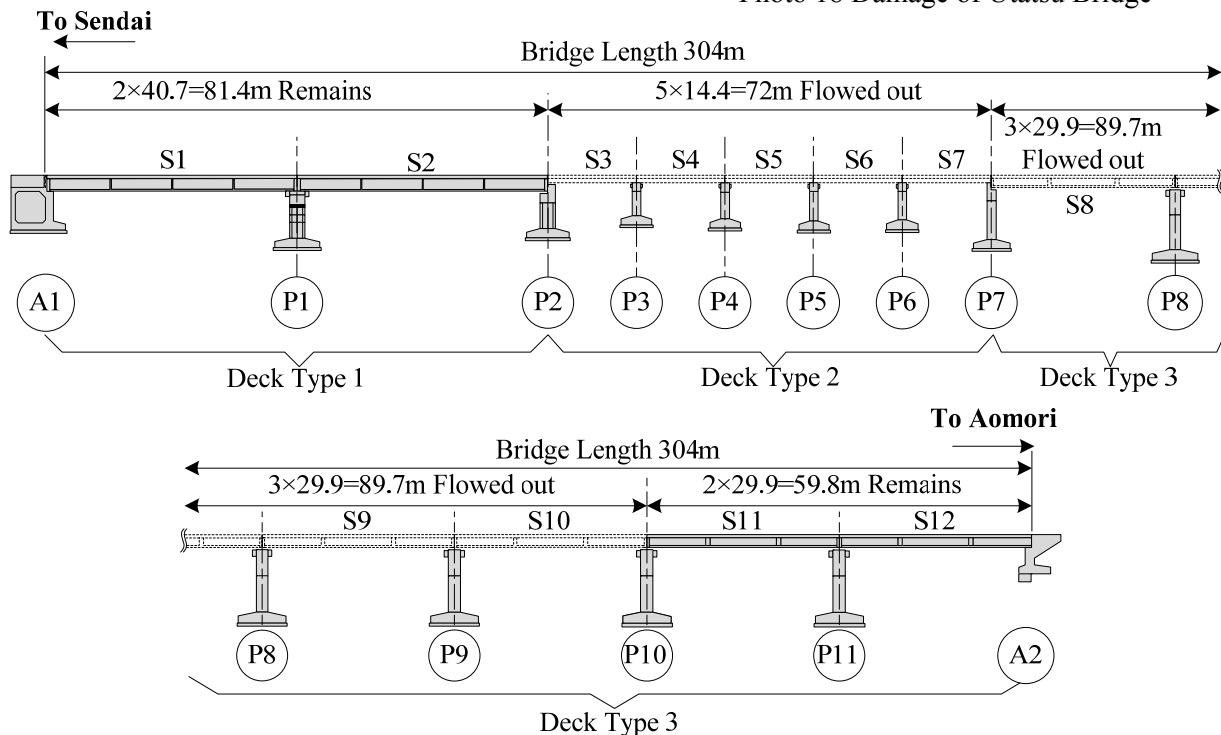


Fig. 6 Utatsu bridge

In the JR seismic evaluation, it was evaluated that shear failure occurred if the parameter  $\alpha$  was smaller than 0.9. From the fact that the columns in No. 1 Nakasone viaducts failed in shear although  $\alpha$  was not as small as 0.9, it is recommended to revise the criteria of failure mode in the future seismic retrofit program.

## 5. TSUNAMI INDUCED DAMAGE OF BRIDGES

### 5.1 Typical Damage to Road Bridges

More than 300 bridges were washed away by the

tsunami. Jurisdiction of the bridges which were washed away by tsunami can be classified as shown in Table 1. A large number of bridges which suffered damage were either on railways or regional roads. Smaller and shorter span bridges which were built in the early days were vulnerable to tsunami effect. It was generally seen that either tall bridges or short bridges did not suffer damage by tsunami because tsunami front did not reach the bridges or over passed. The bridges with mi-range height suffered extensive damage because debris or ships directly attacked decks [JSCE 2011].

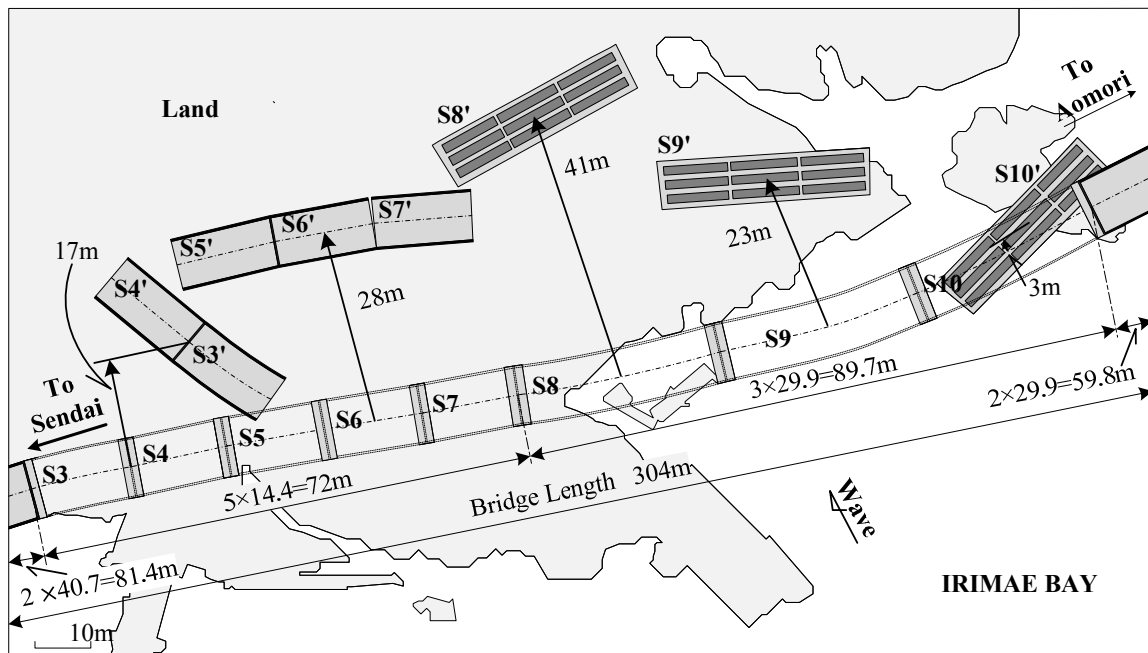


Fig. 7 Outflow of superstructure



Photo 19 Effective restrainers to tie together adjacent decks

Utatsu Bridge at Minami-sanriku Town over Irimae Bay suffered extensive damage by tsunami as shown in Photo 18. It consisted of 3 types of superstructures with spans ranging from 14.4m to 40.7m, as shown in Fig. 6. The superstructures from S3 to S10 were completely washed away from their supports in the transverse direction due to tsunami while the superstructures S1, S2, S11 and S12 were not



Photo 20 An overturned superstructure

washed away. The outflow displacements of S3~S10 are shown in Fig. 7. It should be noted that the spans located at the center such as S5-S7 and S8 were flowed 28 m and 41 m away from the original position, while the spans located at the sides such as S1, S2, S11 and S12 were not washed away. It is noted that S3, S4 and S4, and S5, S6 and S7 which flowed out together because they were tied by cable restrainers for preventing excessive superstructure response under a large seismic excitation as shown in Photo 19. S8, S9 and S10 overturned during being floated as shown in Photo 20.

Photo 21 shows the top of a pier after



Photo 21 Steel devices for extending seat length (short device) and steel stoppers for preventing excessive longitudinal deck response due to ground motions (tall device)

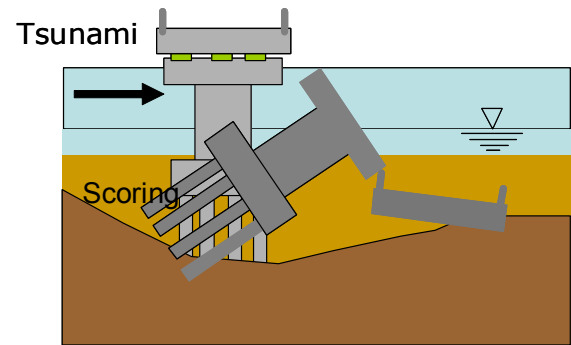


Photo 22 Failure of a RC side stopper, and four steel stoppers for preventing excessive longitudinal deck response which were not damaged

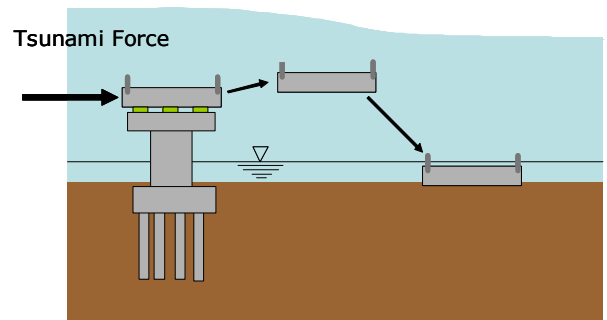


Photo 23 An upper steel bearing after damaged

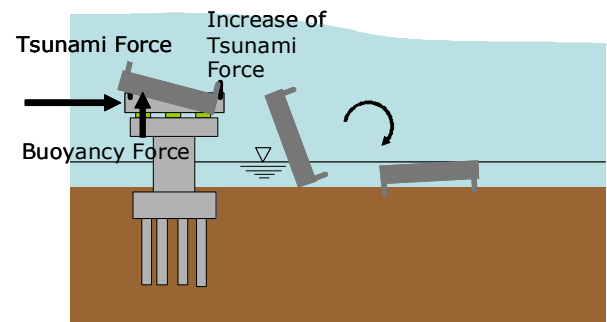
superstructures were washed away. Two types of steel devices were set as an unseating prevention device in this column; one is the devices aiming of increasing seat length required by the recent code, and the other is the devices which were set



(a) Scoring



(b) Transverse offset



(c) Offset after overturning

Fig. 8 Possible mechanism of bridge damage by tsunami

for preventing excessive deck displacement in the longitudinal direction. It is important to know that none of those devices tilted or were detached from the pier which must have happened if the decks were simply washed away laterally. It is likely that the decks were uplifted by tsunami buoyancy force and then they were washed away. Steel plate bearings used in this bridge was very simple as shown in Photo 22 such that both uplift and lateral force capacities were limited.

This is also the case at a column shown in Photo 23 in which four stoppers did not tilt. But a RC side stopper at the land side collapsed probably



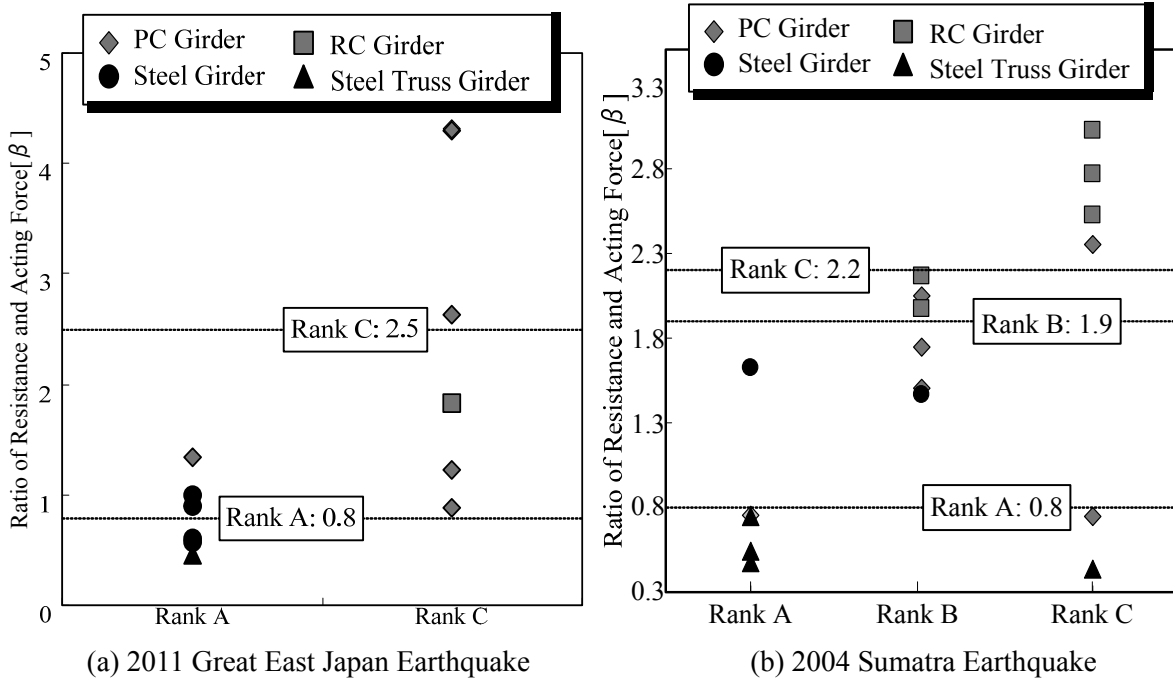


Fig. 9 Correlation of  $\beta$  value with damage ranks

due to a transverse force which applied from the deck. It is likely that due to tsunami force the deck uplifted at the sea side first being supported only at the land side, which resulted in larger tsunami force. Thus the side stopper at the land side collapsed due to excessive concentration of tsunami force.

Fig. 8 shows possible mechanism of damage of bridges due to tsunami. As mentioned earlier, overturning of foundation due to scouring did not occur in road bridges. It is likely that damage of decks in Photos 20 and 21 occurred due to mechanism shown in Fig. 8 (b) and (c), respectively.

In spite of the extensive damage of superstructures, none of piers suffered damage due to tsunami at Utatsu bridge.

## 5.2 An Evaluation of Tsunami Induced Damage

A preliminary analysis was conducted to evaluate tsunami induced damage of bridges by assuming that superstructures were simply laterally washed away without uplift. A parameter  $\beta$  was defined as [Kosa et al 2010]

$$\beta = \frac{S}{F} \quad (1)$$

in which  $F$  is tsunami force and  $S$  is the lateral capacity of a superstructure.

$F$  and  $S$  were evaluated assuming that the lateral capacity of a superstructure in the transverse direction  $S$  can be evaluated by a friction force between a superstructure and a pier and that tsunami force  $F$  can be evaluated as a fluid force as [Kosa et al 2010]

$$F = \frac{1}{2} \rho_w C_d v^2 A \quad (2)$$

$$S = \mu W \quad (3)$$

in which  $\rho_w$  is mass density of fluid,  $C_d$  is the fluid water force coefficient,  $v$  is tsunami wave velocity,  $A$  is side area of a superstructure where tsunami force applies,  $\mu$  is the friction coefficient, and  $W$  is dead weight of a superstructure.  $C_d$  was determined according to the Japanese specification [JRA 2002] and  $v$  was assumed as 6.0 m/s based on estimated

tsunami particle velocity at Utatsu Bridge. The frictional coefficient  $\mu$  was assumed as 0.6 based on experiment [Shoji et al 2009].

Damage degree was defined Rank A; wash away of a superstructure, Rank B; some residual drift of a superstructure and Rank C; no movement of a superstructure.

Fig. 9 shows correlation of the parameter  $\beta$  with damage degree for 13 bridges. For comparison,  $\beta$  values evaluated for bridges during 2004 Sumatra Island earthquake are also shown here [Kosa et al 2010]. A correlation existed between the  $\beta$  value and damage degree for Indonesian bridges, with the mean  $\beta$  value of 0.8 for Rank A damage, 1.9 for Rank B damage, and 2.2 for Rank C damage. Consequently, there existed 2.5 times difference in  $\beta$  values between Ranks A and Rank C damage.

On the other hand, for 13 bridges, the mean  $\beta$  values was 0.8 for Rank A damage and 2.5 for Rank C damage during 2011 Great East Japan earthquake, though the latter had a relatively large scattering in the  $\beta$  value. The difference of  $\beta$  values between Ranks A and C damage was about 3.

## 6. CONCLUSIONS

Damage of road and railway bridges during 2011 Great East Japan earthquake was presented. Although more through collection of damage information as well as careful analyses is required, the following conclusions may be tentatively deduced based on the findings presented herein:

1) Ground motion induced damage of bridges which were built in accordance with the post-1990 design code was minor. Thus the effect of enhancing the shear and flexural capacity as well as ductility capacity, extensive implementation of elastomeric bearings and strengthening of unseating prevention devices were effective for mitigating damage during this earthquake. However, effectiveness of those measures against

much stronger near-field ground motions has to be carefully investigated since the ground motion induced by 2011 Great East Japan earthquake was smaller than anticipated target ground motions.

2) On the other hand, ground motion induced damage of bridges which were built in accordance with old code (approximately pre-1990) or which were not yet retrofitted was still extensive. This was in particular true for railway bridges. Appropriate seismic retrofit is required in the near future.

3) Tsunami induced damage was extensive to bridges along the Pacific coast. It seems that decks were uplifted and washed away upstream. Tsunami force effect has to be studied more so that it can be considered in design for bridges along the coast.

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