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IMPLICATIONS OF BRIDGE PERFORMANCE DURING GREAT EAST JAPAN EARTHQUAKE FOR U.S. SEISMIC DESIGN PRACTICE

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ABSTRACT: About 200 highway bridges and numerous rail bridges were damaged during the Great East Japan Earthquake. The causes of this damage were principally ground shaking (including ground failure and liquefaction), and tsunami inundation. Implications for US design practice include: (1) Japan's retrofit program and revised design specifications were very effective; (2) design provisions for elastomeric bearings may need revision; (3) design strategies for survival of tsunami inundation appear feasible; and (4) duration effects were inconclusive but sufficient to warrant further study.

Key Words: Great East Japan earthquake, bridge performance, design and retrofit programs, elastomeric bearings, tsunami inundation, duration, implications for United States

INTRODUCTION

About 200 highway bridges and numerous rail bridges were damaged during the Great East Japan Earthquake of March 11, 2011, including span unseating, foundation scour, ruptured bearings, column shear failures and approach fill settlements. The causes of this damage can be broadly classified in two categories: ground shaking including ground failure (liquefaction), and tsunami inundation. Of these, the tsunami was responsible for about one-half of the number of damaged bridges.

This damage is briefly reviewed in this paper followed by a set of observations regarding bridge performance during this extreme event and implications for US practice. Locations of bridges discussed in this paper are shown in Figure 1.

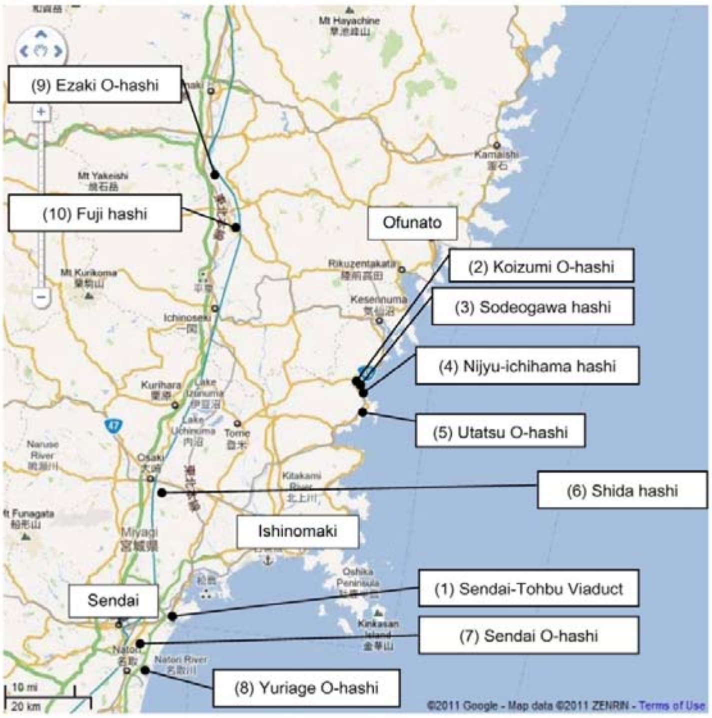

Fig. 1 Damaged bridge locations
(map: L. Marsh)

BRIDGE DAMAGE DUE TO GROUND SHAKING

In general the amount of damage due to ground shaking was remarkably light considering peak ground accelerations that in some locations that exceeded 1.0 g, (short-period spectral accelerations in excess of 5g), with durations in excess of 2 minutes.

The most likely explanation is that the design specifications for new bridges were significantly revised following widespread damage to bridges in the 1995 Hyogoken-Nanbu (Kobe) earthquake (JRA 1996), and that most, if not all, of the existing bridges on the national highway system had been seismically retrofitted over the last 10-15 years. Bridges that were damaged in this earthquake (by ground shaking) were generally older structures owned by city and local governments, where retrofitting had not yet begun, or was incomplete at the time of the earthquake, due to a lack of funding.

Table 1 summarizes the damage observed in six bridges due to ground shaking. It will be seen that typical damage includes failures of older steel bearings, anchor bolt pullout, and flexural/shear cracking at mid-height columns at, or near, rebar termination. In one bridge, transverse steel stoppers failed followed by rupture of adjacent elastomeric bearings with consequential distress to cross frames and web stiffeners of the plate girder superstructure. In another bridge, ground settlement and lateral flow due to liquefaction appear to be the reasons for the observed structural damage.

Table 1 Summary of typical damage due to ground shaking

Bridge	Type	Year Built	Spans	Length	Damage
Shida hashi	Steel girders (2), 2-column concrete piers	1957	9	NA	Failed steel bearings, anchor bolt pullout, abutment pedestal and back wall damage, settlement of superstructure over pier with bearing damage .Flexural cracking at mid-height in piers at/near rebar termination.
Sendai O-hashi	Steel girders, concrete wall piers	1965	9	310 m	Infill panel damage between original and new substructure units. Some delamination of fiber wrap retrofits of substructure units due to relative movement between units.
Fuji hashi	Steel plate girders (3), single column piers	1972	13	NA	Flexural/shear cracks in columns at/near rebar termination. Pin fracture in older-style steel bearings.
Yuriage O-hashi	Concrete box girder, concrete wall piers	1974	10	542 m	Steel bearing failure (roller bearings); insufficient capacity for longitudinal movement of pier caused, possibly, by liquefaction-induced lateral spread.
Ezaki hashi	Concrete box girder, concrete wall piers	1982	9	586 m	Flexural/shear cracks in piers at/near rebar termination. Inclined shear cracks in piers in weak direction.
Sendai-Tobu Viaduct	Steel plate girders (8) and steel box girders (3-5) on single- and 2-column steel columns	2000	NA	4.4 km	Failure of steel stoppers and elastomeric bearings at two locations. Buckled and/or severely distorted girder stiffeners, gusset plates, and cross frames. Some minor column yield at base.

Sendai-Tobu Viaduct

The damage to this 4.4 km long, multi-span viaduct in north Sendai was largely confined to a 10-span section between Piers 52 and 62. Built in 2000, this section of the viaduct was being widened at the time of the earthquake. New on- and off-ramps were under construction between Piers 54 and 56 to connect Route 10, carried by the viaduct, to Route 141 below. The superstructure comprises eight steel plate girders (I-girders) between Piers 50 and 52, three, four, or five steel box girders between Piers 52 and 56, and eight steel plate girders between Piers 56 and 63. Elastomeric bearings are used exclusively with external stoppers to restrain transverse movement at almost every pier. Piers 54, 55, 56, 58, 59 and 60 had recently been converted from single steel box columns to two-column steel box frames to accommodate new on- and off-ramps (Figure 2). The remaining piers (51, 52, 53 and 57) are single-column steel boxes (Figure 3).

The bridge suffered moderate-to-major damage during the earthquake but no span collapsed. This damage included the failure of 40 steel stoppers and 18 elastomeric bearings. Another 14 stoppers and 3 bearings were heavily damaged. In addition, girder stiffeners, gusset plates, and cross-frames were buckled or severely distorted.

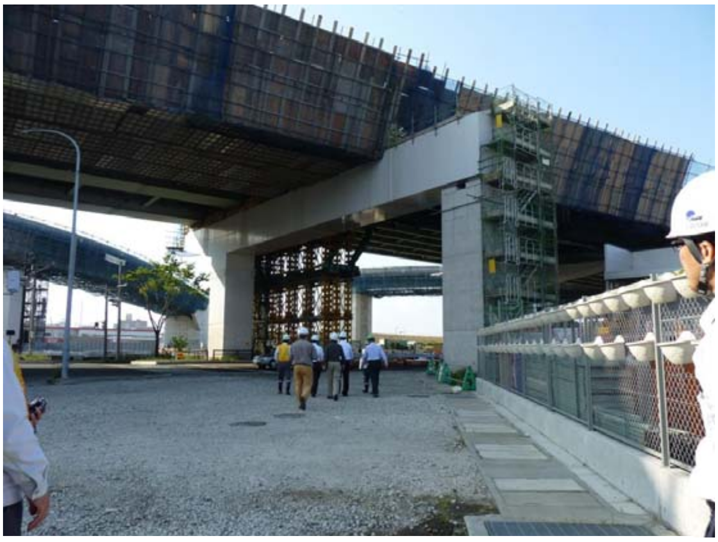

Fig. 2 Two-column frame, Pier 56, Sendai-Tobu Viaduct
(photo: E. Monzon)

The bearing damage was concentrated in a region of the viaduct where, as noted above, it was being widened and there was a corresponding change in lateral stiffness – from single-column hammerhead piers at Pier 57 to two-column frames at Piers 54, 55 and 56, for example. There was also a significant change in the in-plane stiffness of the superstructure in this section, from eight I-girders in Spans 52 and 57 to multiple single-cell box girders in Spans 53 to 56. This section of the viaduct was therefore relatively stiff (and particularly Spans 55 and 56) compared to sections to the north and south. It is therefore possible that when earthquake loads were applied, this difference in stiffness generated high lateral forces in the stoppers at the two interfaces (over Piers 52 and 56) leading to their failure and the transfer of load to the bearings.

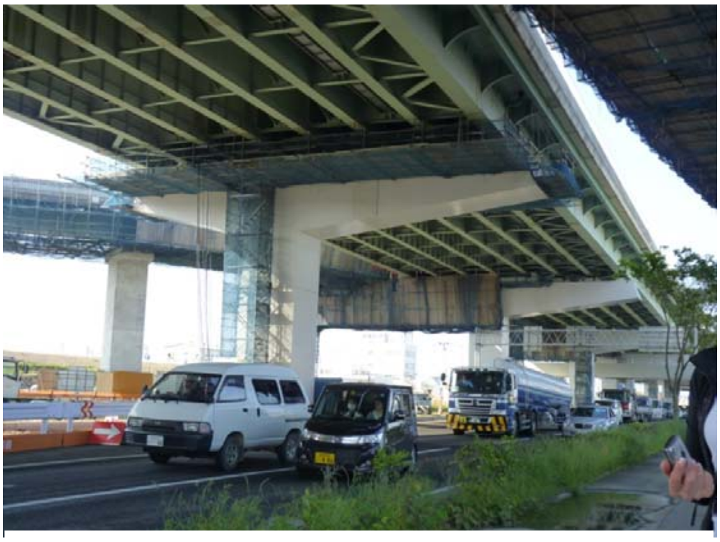

Fig. 3 Single-column pier, Pier 57, Sendai-Tobu Viaduct
(photo: E. Monzon)

Inspection of the damage to the bearings showed that many had ruptured completely through the elastomer, as if in direct tension. Others showed damage to the internal shims which had been severely distorted (Figure 4). Typical dimensions of the bearings at Pier 56 were 820 x 870 x 508 mm, with 8 x 33 mm layers of elastomer, 7 x 4.5 mm shims and 2 x 45 mm end plates. The masonry and sole plate connections were detailed to transfer both shear and axial forces (tension and compression) to the bearings.

It seems possible that the bearings failed due to the combination of two effects. First the high lateral forces in the steel stoppers at Piers 52 and 56 were probably not evenly distributed amongst the three effective stoppers. (Although there are six stoppers at each pier, only three are effective at any point in time.) This uneven distribution can arise in situations where the gaps between the stoppers and the sole plates of the bearings are not identical and one stopper engages before the others. Overloading of this stopper may then lead to its failure, followed by the transfer of load to the other stoppers which fail in turn. Once all the stoppers have failed the transfer of load to the bearings places them under very high shear strain.

The second effect is the generation of high tensile forces in the bearings at these same locations due to the difference in pier type. For example Pier 56 is a 2-column frame and Pier 57 is a single column hammerhead pier. Under lateral load the hammerhead rotates about a longitudinal axis twisting the superstructure about the same axis. But the pier cap in the 2-column frame at Pier 56 does not rotate in this manner and this frame resists the twisting of the superstructure. High tensile forces are developed in the bearings as a result. The simultaneous occurrence of high tension and high shear in the bearings could have led their failure.

The shim damage seen in Figure 4 most likely occurred when a ruptured bearing impacted a toppled stopper puncturing the cover rubber layers and distorting the edge of the shim plate.

It is noted that the above scenario (multiple stopper failure followed by multiple bearing failure) implies the occurrence of many cycles of large amplitude motion over an extended period of time, which is possible given the long duration of this particular earthquake.

Other damage to the superstructure included buckled cross-frame members, gusset plates and stiffeners, possibly due to the abrupt change in load path where the transverse member changes from a partial height diaphragm to a full depth cross-frame. But a more likely scenario is that this damage is due to the failure of the bearings below the girders leading to differential ‘settlement’ of the cross-frames and corresponding distortion and distress.

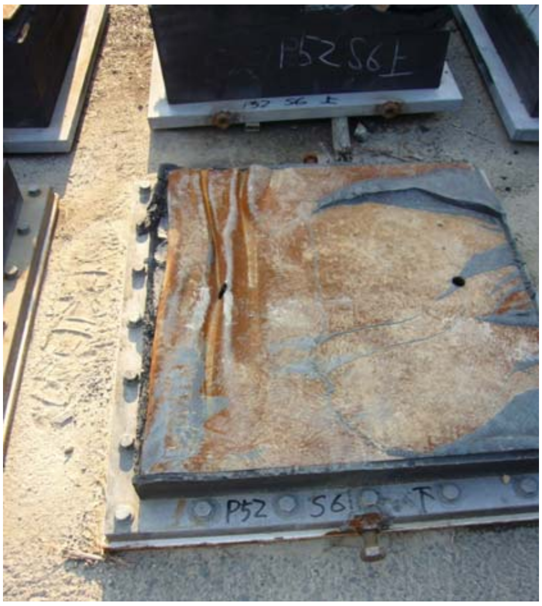

Fig. 4 Damaged elastomeric bearing from Pier 52
Sendai-Tobu Viaduct (photo: I. Buckle)

BRIDGE DAMAGE DUE TO TSUNAMI INUNDATION

Twelve bridges on Route 45 were seriously damaged by the tsunami, which had wave heights from Sendai to Hachinohe ranging from 6.2 to 11.8 m. Typical damage included loss of superstructure in non-integral bridges (bridges with bearing-supported girders), loss of approach fills and undermining of foundations due to scour. The performance of two of these bridges, and two nearby rail bridges, is described in this section.

Koizumi Highway and Rail Bridges

The Koizumi bridge spans the Tsuya River on Route 45 south of the city of Kesennuma. The bridge was constructed in 1975, has six 30.1-m spans (total length 182 m), and is 11.3-m wide. The superstructure comprises four steel plate girders supported by concrete piers on deep foundations. The bridge is without skew and only a slight vertical curve. The superstructure segments were continuous over three spans with expansion joints at the abutments and at the center pier (Pier 3). Piers 2 and 4 had fixed bearings, while Piers 1 and 5 had sliding bearings in the longitudinal direction.

The bridge had been seismically retrofitted using hydraulic dampers at the abutments. It is not known if similar restrainers or dampers had been installed at the expansion joint over the center pier.

All six spans were swept away during the tsunami (Figure 5). Wave heights of the order of 11.8 m were registered at Ofunato City just north of this site and the tsunami clearly overtopped this bridge taking all six spans upstream. Based on damage to the levee on the north bank of the river (Figure 6), some of these spans were swept along the top of the levee on the north bank, then over the levee altogether on the north side, and later back over the levee into the main channel where they came to rest about 400 m upstream from the bridge (Figure 7). Other spans took a different path and came to rest about 300 m upstream but on the south side of the levee on the south bank of the river. Four of the five piers are still standing, but the center pier (Pier 3) was overturned and believed to be under water in the river channel just upstream of the bridge (Figure 5).

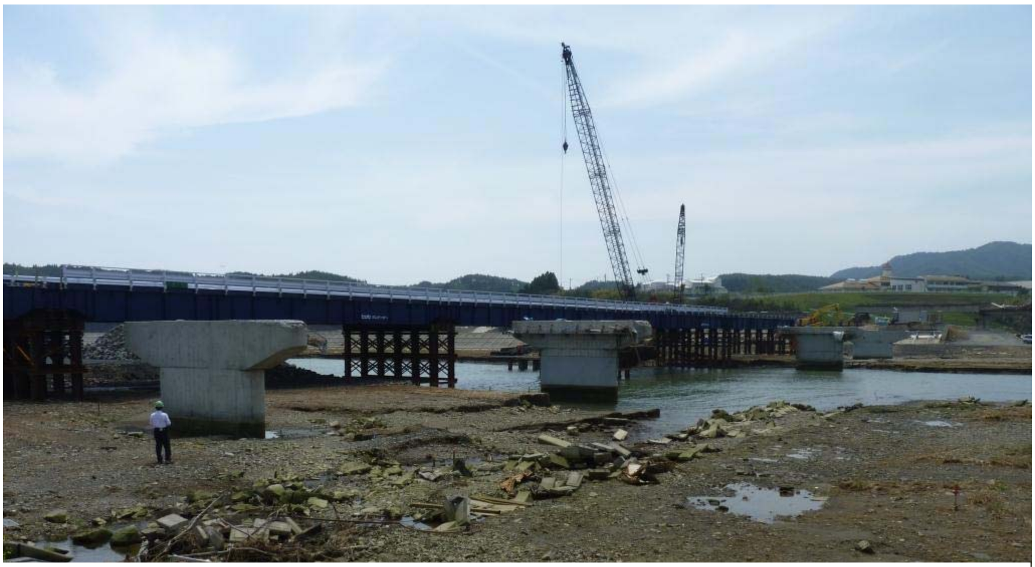

Fig.5 Remaining piers of Koizumi Bridge shown at low tide. Temporary bridge is under construction on seaward-side of bridge (photo: E. Monzon)

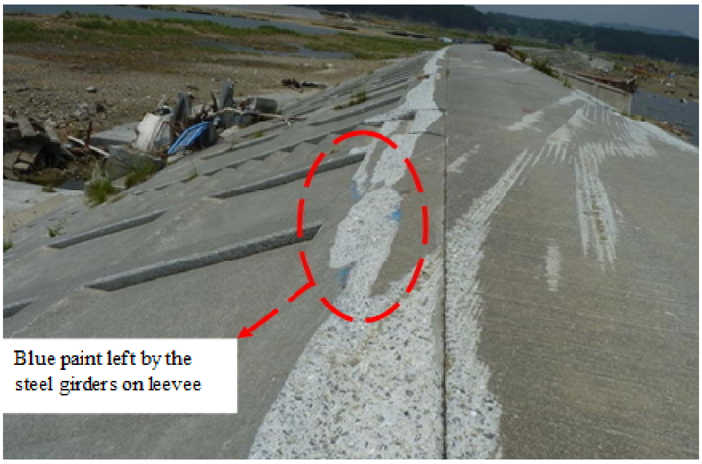

Fig. 6 Drag marks on top of levee caused by girders from Koizumi Bridge (photo: E. Monzon)

Fig. 7 Girders from Koizumi Bridge 400 m upstream in Tsuya River channel (photo: E. Monzon)

It is clear that the longitudinal dampers installed at the abutments and the transverse keys (stoppers) over the piers, offered little restraint to the lateral loads imposed by tsunami. Once these devices failed, the relatively light weight of the steel I-girders, together with the buoyancy effects of air trapped between the girders, enabled the superstructure to be easily lifted and carried significant distances upstream. The loss of Pier 3 was probably due to scour but this could not be confirmed. Despite the low tide at the time of the visit, this foundation was still underwater.

About 900 m upstream of the Koizumi Bridge, the JR East rail line to Kesennuma crosses the Tsuya River on a multispan, prestressed concrete girder viaduct. Five of these spans were washed out, but the piers survived (Figure 8). The in-coming tsunami apparently breached the levee behind the piers allowing flow oblique to the channel. The piers are tilted toward the breach, and the simple span, three-girder superstructures came to rest on the opposite side of the levee.

Of interest is the damage to the lower portions of the piers. The exposed reinforcement seen on the left side of each pier appears to have been pulled outward from the center of the column, rupturing the

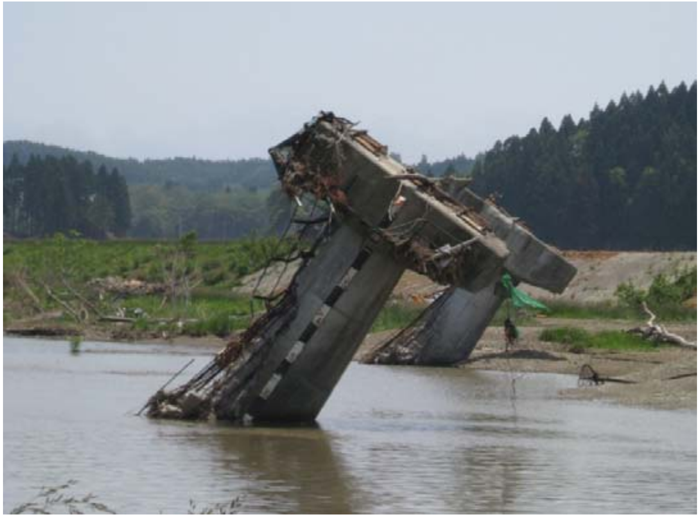

Fig. 8 Damaged piers of the JR Rail Viaduct crossing the Tsuya River (photo: S. Dashti)

relatively light transverse steel. This type of behavior is seen in the failure of beams that are unreinforced for shear, where a shear crack precipitates failure and tearing of the tensile reinforcement from the beam. In the case of the JR East piers, potential buoyancy of the superstructure due to trapped air and the hydrodynamic forces produced lateral loads on the piers along with eccentric vertical loading. The piers may have failed in shear above the foundation after plastically deforming under the combined lateral and vertical effects. Following the loss of shear capacity at the base, the tension reinforcement was torn from the piers.

In this postulated mode of failure the tilting of the pier is due to structural failure and not to scour and subsequent rotation of the foundation. Inspection of the columns and footings below the water line is required to confirm this behavior.

Nijyu-ichihama Highway and Rail Bridges

The Nijyu-ichihama highway bridge spans a small stream on Route 45, south of the Koizumi and Sodeo-gawa bridges. This bridge was built in 1971 and is a single-span prestressed concrete I-girder bridge supported on tall, cantilever abutments, which are in turn supported on steel pipe piles. The bridge has no skew, no curve and essentially no grade. The span is 16.64 m and the total width of the original structure is 8.7 m. End diaphragms engage each of the eleven I-girders comprising the deck and in turn, and were anchored to the abutment seats with tie-down rods in each bay. These same diaphragms acted as transverse shear keys restraining the lower flange of each girder from lateral movement.

The bridge had been widened on both sides at some time in the past using precast double-tee beams spanning between new abutments each founded on steel piles with heads at a much higher elevation than those of the original structure. The tsunami washed out the backfill behind both abutments and temporary approach spans, using steel I-girders, were placed to open the bridge to traffic. These spans are seen in Figure 9. Temporary steel towers to support these spans may also be seen in this figure.

Fig. 9 Loss of backfill on both approaches to single-span Nijyu-ichihama Bridge (photo: I. Buckle)

Apart from the loss of the seaward extension, this bridge has performed remarkably well from a structural point of view. It is essentially intact and the principal reason for closure was the loss of back fill due to erosion. Despite the buoyancy of trapped air, the superstructure was well anchored both vertically and laterally to the abutment seats and was not dislodged by the tsunami despite being overtopped. It is of course possible that the erosion of the abutment backfills and the creation of two alternative hydraulic channels took load off the bridge, but nevertheless the performance of this bridge is noteworthy.

About 100 meters upstream from the Nijyu-ichihama bridge is the JR East line to Kennesuma, which runs a distance of several hundred meters across the valley between tunnels at either end. This section of rail line was supported on a long fill embankment, two box culvert roadway underpasses, and a prestressed concrete, single span bridge over the river (Figure 10). The unprotected embankment appeared to be a granular material. As the wave overtopped the embankment, it displaced the tracks and significantly scoured and removed the upper 4 to 5 m of the fill. Apart from the loss of the approach fills, all the bridges in the valley appeared to be intact.

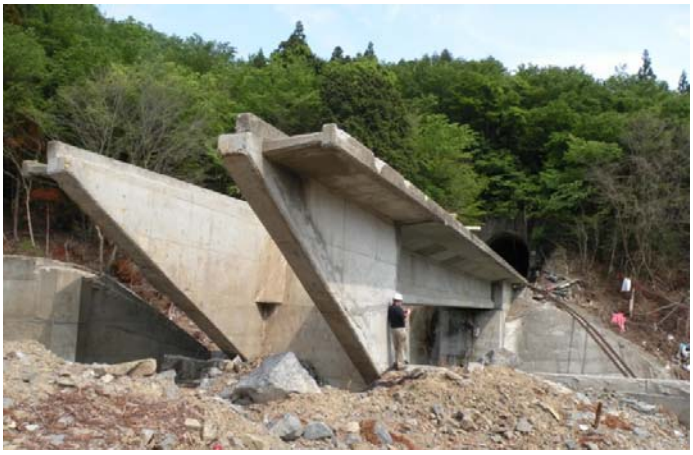

Fig. 10 Exposed wingwalls of JR Rail Bridge 100m upstream from Nijyu-ichihama Bridge due to loss of approach embankment (photo: D. Frost)

DESIGN IMPLICATIONS

The performance of the bridges described above has the following implications for the seismic design of new bridges (AASHTO 2007, AASHTO 2010), and the seismic retrofitting of existing bridges in the United States (Caltrans 2010, FHWA 2006):

1. Despite the magnitude and duration of this earthquake, bridge damage outside of the coastal zone was not heavy. This is believed to be due to the fact that capacity design principles were implemented in Japan for new bridges in the mid-1990s (JRA 1996), and an active retrofit program was undertaken for older bridges, following the Hyogo-ken Nanbu (Kobe) earthquake in 1995. Such performance endorses the design philosophy in the AASHTO specifications and in particular, the recent adoption of the displacement-based seismic design guidelines (AASHTO 2010) as an alternate to force-based provisions (AASHTO 2007). The success of various retrofit measures confirms the best practices adopted by both Caltrans and FHWA (Caltrans 2010, FHWA 2006).

2. Retrofitting is an effective means for minimizing earthquake damage in older bridges. Most of the observed structural damage due to ground shaking occurred in older bridges that had not yet been retrofitted, or only partly so. It is recommended that strong encouragement be given to owner agencies in the United States to accelerate their retrofit programs.

3. With the exception of several spans of the Sendai-Tobu Viaduct, elastomeric bearings performed well and considerably better than older-style, steel bearings. The reason for the poor performance of the Sendai-Tobu bearings needs to be determined quickly for it has widespread implication on the use of these devices as expansion and seismic isolation bearings in the United States. It is noted that the damage to the superstructure of this Viaduct reinforces the recent move in the U.S. to require superstructures be explicitly designed for seismic loads and the need for the lateral load path to be clearly identified and designed.

4. Damage to several older, un-retrofitted, bridge piers was concentrated in the reinforcement termination zone, and this vulnerability should be considered when prioritizing bridges for retrofitting in the United States.

5. Design methods to mitigate tsunami damage from inundation should be developed. Strategies to keep superstructures in place (such as using integral connections and venting trapped air to reduce buoyancy and equalize hydrostatic pressures on deck slabs) should be explored, along with armoring techniques to prevent undue scour of foundations and approach fills. In addition, the cost of deeper foundations should be weighed against the potential loss of a pier and the need for replacement of one or more spans.

6. Until analytical studies are complete it is not known to what extent the duration of this earthquake affected the observed damage, but it is expected to have been significant (e.g. in the Sendai-Tobu Viaduct). The effect of duration on structural response should be investigated and, if its importance is confirmed, methods should be developed for inclusion in design loadings and/or detailing.

CONCLUSIONS

The performance of the above sample of bridges leads to two major conclusions:

1. Capacity-based design is an appropriate philosophy for the design and retrofit of bridges subject to strong ground motions, and
2. Bridges can be designed to survive tsunami inundation with minimal damage. Survival is clearly dependent on superstructure type and bridges with integral connections appear to have the least vulnerability to loss-of-span. If approach fills are not severely eroded and the piers are of sufficient depth to be unaffected by scour, full functionality may be quickly restored.

Other conclusions relate to the vulnerability of older steel bridge bearings, columns with rebar termination at or near mid height, substructures on foundations in liquefiable soils, and superstructures with inadequate load paths for lateral loads. The effect of duration on response is inconclusive and requires further analysis.

It is noted that these conclusions are based on observations made and data recovered during reconnaissance visits in June and November 2011. They are therefore of a somewhat speculative nature due to the small number of bridges investigated and the absence of detailed field data such as foundation and soil details, bearing and tie-down details, superstructure weights, wave heights, and velocity profiles at each site. It follows that the above implications for U.S. practice and Conclusions, are likely to change as additional data becomes available and more detailed studies are completed.

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A joint EERI/FHWA/GEER reconnaissance team visited the affected area June 2-6, 2011, and investigated eleven bridges, some of which are reported in this paper. A follow-up visit was conducted by a subset of this team on November 9-12, 2011.

The team was hosted in Japan by Task Committee G of the UJNR Panel on Wind and Seismic Effects. Membership was as follows:

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