Learning from Earthquakes

Bridge Performance in the Mw 9.0 Tohoku, Japan, Earthquake of March 11, 2011

A joint EERI/Federal Highway Administration (FHWA)/Geotechnical Extreme Events Reconnaissance (GEER) Association reconnaissance team visited the affected area from June 2 to June 6, 2011. Members included Ian Buckle, University of Nevada-Reno; Shideh Dashti, University of Colorado-Boulder; David Frost, Georgia Institute of Technology; Lee Marsh, Berger/ABAM Engineers; Eric Monzon, University of Nevada-Reno; W. Phillip Yen, FHWA, (co-chair). The team was hosted in Japan by Task Committee G of the U.S.-Japan Cooperative Program in Natural Resources (UJNR) Panel on Wind and Seismic Effects. Japanese members included Taku Hanai, Public Works Research Institute; Jun-ichi Hoshikuma, Public Works Research Institute; Kazuhiko Kawashima, Tokyo Institute of Technology; Teturou Kuwabara (co-chair), Public Works Research Institute; Hideaki Nishida, Public Works Research Institute; Keiichi Tamura, Public Works Research Institute; and Shigeki Unjoh, National Institute for Land and Infrastructure Management.

The publication of this report is supported by EERI under NSF Award #CMMI-1142058; by the FHWA under Contract DTFH61-07-C-00031 to the University of Nevada-Reno; and by GEER under NSF Award #CMMI-1138203 to the University of California, Davis.

Introduction

About 200 highway bridges and numerous rail bridges were damaged during the March 11th earthquake by effects ranging from span unseating, ruptured bearings, and column shear failures, to foundation scour and approach fill settlements. The causes can be broadly classified in two categories: ground shaking, including ground failure (liquefaction); and tsunami inundation. The tsunami was responsible for the damage in about one-half of the bridges.

The reconnaissance team investigated 11 bridges, of which two had extensive bearing failures, two had column failures, two had combined bearing and column failures, and four suffered tsunami-related damage (unseated spans, scour, loss of approach fill). The locations and names of ten of the bridges visited are shown in Figure 1. The 11th bridge was the Arakawa Wangan Bridge across the Arakawa River in Tokyo. The performance of five of the bridges is summarized in this report. A detailed description of all 11 bridges is given in a reconnaissance report to be published by the Federal Highway Administration in the near future.

Damage Due to Ground Shaking

In general, the amount of damage due to ground shaking was remarkably light considering peak ground accelerations in some locations that exceeded 1.0 g with short-period spectral accelerations in excess of 5 g. The most likely explanation for this light damage was that the ground motions at the affected bridges were relatively short-period, with most peak ground accelerations occurring in the 0.2-0.5 Hz range. This range of frequencies is generally considered to be the most damaging to structures due to its potential to induce resonance in structures with similar natural frequencies.

Figure 1. Bridges Investigated by EERI/FHWA/GEER Reconnaissance Team (map: L. Marsh).
of the moderate damage is that most, if not all, of the bridges in the national highway system had been seismically retrofitted over the last 10-15 years (in response to the widespread damage to bridges in the 1995 Kobe earthquake). Bridges damaged by ground shaking in this earthquake were generally older structures owned by city and local governments, where retrofit programs have not been as active due to a lack of funding.

With one exception, new bridges performed very well regardless of ownership, most probably due to the adoption of conservative capacity design principles in the JRA Design Specifications in the 1990s. The one exception was the elastomeric bearings failure in a section of the Sendai-Tobu Viaduct described below.

**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 bridge suffered moderate-to-major damage but no span collapsed. This damage included the failure of 40 steel stoppers and 18 elastomeric bearings. Another 14 stoppers and three bearings were heavily damaged. In addition, girder stiffeners, gusset plates, and cross-frames were buckled or severely distorted. Locations of the failed and damaged stoppers and bearings, due to both the March 11 main shock and the April 7 aftershock, are given in Table 1, as are span lengths and types between Piers 51 and 58. 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 were 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 the new on- and off-ramps (Figure 2). The remaining piers (51, 52, 53 and 57) are single-column steel boxes (Figure 3).

The pattern of the bearing damage in Table 1 is particularly interesting. It is concentrated in regions of the viaduct where there is a significant 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 is 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 is therefore very stiff (particularly Spans 55 and 56), while sections to the north and south are comparatively flexible. When earthquake loads are applied, the difference in displacements at the two interfaces (Piers 52 and 56) generates high lateral forces in the stoppers at these two piers leading to their failure and the transfer of load to the bearings.

Inspection of the damage to the bearings showed that many had ruptured completely through the elastomer, as if in direct tension. In other bearings, the internal shims had been severely distorted (Figure 4). Typical dimensions of the bear-

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**Table 1. Span Details and Distribution of Failed Steel Stoppers and Elastomeric Bearings in the Sendai-Tobu Viaduct**

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**Notes**

1. There are two lines of stoppers and bearings on Piers 52, 56, and 58.
2. Numbers in parentheses are numbers of stoppers and bearings damaged but not ruptured.

37 (14) 18 (1) 3 (2)
ings at Pier 56 are 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) into the bearings.

It seems possible that the bearings failed due to the combination of two effects: simultaneous high shear and high tension in the bearings. First, the high lateral forces in the steel stoppers at Piers 52 and 56 were probably not evenly distributed among the three effective stoppers. (Although there are six stoppers at each pier, only three are effective in any one direction at any time.) This uneven distribution arose because the gaps between the stoppers and the sole plates of the bearings are not exactly the same at each location, and one stopper will generally engage before the others. Overloading of this stopper very likely led to its failure, followed by the transfer of load to the other stoppers, which failed in turn. Once all the stoppers failed, the transfer of load to the bearings placed 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 two-column frame and Pier 57 is a single-column hammer-head pier. Under lateral load, the hammerhead rotated about a longitudinal axis, twisting the superstructure about the same axis. But the pier cap in the two-column frame at Pier 56 did not rotate in this manner, and this frame resisted the twisting of the superstructure. High tensile forces developed in the bearings as a result.

The shim damage seen in Figure 4 most likely resulted when a ruptured bearing collided with a toppled stopper, puncturing the cover rubber layers and distorting the edge of the shim plate. On the other hand, the expected failure mode of an elastomeric bearing is rupture within a rubber layer and not delamination at the shim plate. The clean surface of this plate in Figure 4 implies inadequate bond between the elastomer and shim during manufacture, thus reducing the bearing’s capacity for combined tension and shear.

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 more likely due to the failure of the bearings below the girders, leading to differential “settlement” of the cross-frames and corresponding distortion and distress.

Yuriage Bridge. The Yuriage Bridge carries Route 10 over the Natori River near the Sendai airport. There was tsunami run-up in the area, but the wave passed under the bridge without damaging it. The bridge was built in 1974 and comprises ten spans with an overall length of 542 m. The three main spans, located between Piers 2 and 5, are cast-in-place concrete box-girders, and the center of these three spans comprises two balanced cantilevers meeting at mid-span (Figure 5). The approach spans are prestressed concrete I-girders, simply supported on steel bearings at the pier caps,
founded on wall piers with caisson-type foundations. Pier 1 was damaged during the 1978 Miyagi-Oki earthquake and was repaired with a concrete jacket. No widening or other seismic retrofits have been made since that time.

There were complete bearing failures at Piers 2 and 5, at the transition between the approach spans and main spans. These failures are attributed to permanent pier movement, possibly caused by liquefaction. Evidence of extensive liquefaction was found under the approach spans and 30 cm of ground settlement was observed adjacent to one of the piers under the south approach. Since Piers 2 and 5 are close to the river, lateral spreading may have caused these piers to move toward the center of the river channel. Figure 6 shows this movement for Pier 2 to be of the order of 6 cm, which clearly exceeded the capacity of the roller bearing at this location. In addition, damage to the transverse stopper is evident, indicating the simultaneous significant shaking transverse to the bridge.

**Shida Bridge and Levee.** The Shida Bridge is a nine-span steel plate-girder bridge carrying Route 32 over the Naruse River east of Osaka. The bridge was built in 1957 and comprises a two-girder steel superstructure supported on concrete two-column piers. The foundation type is unknown. The bridge is straight, has no skew, and only a slight vertical curvature. Typical piers are shown in Figure 7. The superstructure is articulated in every other span with drop-in spans and in-span seats that form inflection points rendering the structure indeterminate (Figure 8). Such a drop-in span is visible in the center span of Figure 7, where rust stains from the deck joint have discolored the two in-span hinge areas. Also visible are seat extensions installed on the piers throughout the bridge. No other seismic retrofitting was seen. The drop-in spans have transverse guides that serve to restrain the
movements of the spans and may help prevent dislodgment of the spans under seismic loading. These guides are positioned at the side of the drop-in bearings, the soffit of the plate girders, and the center of the transverse bracing near the bottom of the girders.

The superstructure rests on steel bearings at each pier, and each continuous span (that is, alternate spans to the drop-in spans) has one fixed and one movable (sliding) bearing in the longitudinal direction arranged as shown in Figure 8.

Damage to the Shida Bridge included a dropped fixed bearing, a fixed bearing with sheared anchor bolts, abutment backwall failure due to soil pressure, yielded fixed bearing anchor bolts and cracked pier walls. None of the dropped-in spans became unseated and there was no apparent damage to the steel cross frames.

There was approximately 15 cm of settlement in the backfill behind the abutment, and pressure behind the abutment backwall was sufficiently large to crack and yield this wall, in all likelihood as the abutment fill settled and moved towards the river channel during the earthquake. The abutment is founded on a levee that parallels the river channel; movements in this levee, just upstream from the bridge, are described later in this section.

One of the fixed bearings at Pier 3 failed by dropping off its lower seat, as seen in Figure 9. Also seen in this figure is a gap between the top of the bearing and the bottom of the girder, indicating that the bolts of the other fixed bearing at Pier 3 have been sheared off and the girder at this location has lifted clear of the bearing. This suggests that the bearing that dropped on the other end of the pier must be supporting the gravity load at the pier, because no redundant load path exists. The bearing and girder have dropped far enough that the upper bearing has become wedged against the lower portion of the bearing, thus providing vertical support. It is likely that the upper bolts of the left-hand bearing sheared before the right-hand bearing dropped from its pedestal. Because there is no vertical load path through the bearing pin to sustain tensile forces on the bolts, shear is the probable mode of failure. Following loss of shear capacity on the left side, all shear for this frame would have to be resisted by the right-hand bearing. This increase in shear demand likely caused the right-hand bearing to un-seat. Failure of this bearing probably caused the noticeable dip in the elevation of the bridge roadway and handrail (estimated at 10 cm) seen in Figure 10. It also possible that some of this deformation is due to foundation settlement from widespread liquefaction in the area.

Pier 3 is one of the infill-wall piers. The infill probably increased the stiffness at Pier 3 in the longitudinal direction which increased the forces in the fixed bearing and resulted in the dropped bearing. Close inspection of the anchor bolts at the top of the left side of Pier 3 indicated that those bolts were elongated or hammered outward more than any of the bolts on the other fixed piers of the bridge. This led to the conclusion that the forces were higher at Pier 3 due to the infill wall.

Following the unseating of the right-hand bearing at Pier 3, the superstructure moved towards Abutment 1, as indicated by the final location of the right-hand bearing at Pier 3. This was confirmed by the observed movement in the expansion bearing at Abutment 1, as indicated in Figure 11. The cracks in the masonry pedestal beneath the girder indicate that the shear keys were engaged as the superstructure moved in this direction.
At two other fixed piers, 1 and 6, the anchor bolts between the bearing and the top of the pier were either elongated or pulled out of their embedment by the longitudinal inertial action of the superstructure. The longitudinal response of the bridge was probably limited by the yielding or slippage of these bolts, and this action provided partial force limitation; however, it was not enough to prevent damage to the piers below the bearings. At both Piers 1 and 6, there was moderate cracking and potential yielding. Figure 12 shows examples of such damage (in fact, both piers were being repaired at the time of the reconnaissance visit, as is evident by the scaffolding and enclosures present).

Due to the rust present in the exposed reinforcement of Pier 6, it is likely that there had been delamination of the reinforcement for some time before the March 2011 earthquake. Since it was built in 1957, the bridge has been through other significant earthquakes. Repairs to other piers, in particular the infill wall ones, was evident, but it is not known whether the infills were repairs to earthquake damage or were added at the time of original construction.

While investigating the performance of the Shida Bridge, we observed that the crest of an 8-m high levee along the Naruse River just upstream of the bridge had settled 1.0-1.5 m. This settlement was accompanied by a lateral toe movement of up to 4-5 m downstream and significant slumping on the downstream face. Protective sheeting covering the failed portion of the levee is seen in Figure 13. Concern that the levee might completely fail during the upcoming rainy season led to a temporary repair consisting of a 240-m long, 3.8-m high, 4.5-m wide, double-sided sheet pile cofferdam on top of a 3.8-m-high berm.

**Damage Due to Tsunami Inundation**

Twelve bridges on Route 45 Sendai to Hachinohe were seriously damaged by tsunami waves with heights ranging from 6.2 to 11.8 m. Damage to two of these bridges and nearby rail bridges is described in this section.
Koizumi Highway and Rail Bridges. The Koizumi bridge spans the Tsuya River on Route 45 just south of the city of Kesen-numa. Constructed in 1975, it has six 30.1-m spans (total length 182 m), and is 11.3 m wide. The superstructure comprised four steel plate girders supported by concrete piers on deep foundations. The bridge is without skew and had 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 whether similar restrainers or dampers had been installed at the expansion joint over the center pier. No other retrofitting, such as support length extensions or substructure strengthening, was evident.

All six spans were swept away during the tsunami (Figure 14). Wave heights on the order of 11.8 m were registered at Ofunato City just north of the bridge, and the tsunami clearly overtopped the bridge, taking all six spans upstream. Damage to the levee on the north bank of the river (Figure 15) indicated that some of the spans were lifted off their piers and 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 16). Other spans took different paths 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 from the bridge (Figure 14).

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, made it possible for the superstructure to be lifted and carried significant distances upstream. The loss of Pier 3 was probably due to scour, but that could not be confirmed since the foundation was still underwater despite the low tide at the time of our visit.

About 900 m upstream from 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 17). The incoming 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 relatively light transverse steel. Behavior like this 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 plastic deformation 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, but inspection of the columns and footings below the water line is required to confirm this hypothesis.

**Nijyu-ichihama Highway and Rail Bridges.** The Nijyu-ichihama Highway Bridge spans a small stream on Route 45 south of Kesennuma and 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 supported in turn 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, 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 was widened on both sides at some time in the past, with precast double-tee beams spanning between new abutments, each founded on steel piles with heads at a higher elevation than those of the original structure. The tsunami washed out the backfill behind both abutments; temporary approach spans, using steel I-girders, were placed to open the road to traffic (see Figure 18). Temporary steel towers to support these spans may also be seen in this figure.

Apart from the loss of the seaward extension, this bridge performed remarkably well from a structural point of view. It is essentially intact and the principal reason for closure was the loss of backfill due to erosion. Despite the buoyancy of trapped air, the superstructure was

**Figure 17.** Damaged piers of the JR Rail Viaduct crossing the Tsuya River *(photo: S. Dashti).*

**Figure 18.** Loss of backfill on both approaches to single-span Nijyu-ichihama Bridge *(photo: I. Buckle).*
well anchored both vertically and laterally to the abutment seats and was not dislodged by tsunami waves despite being overtopped. It is of course possible that the erosion of the abutment backfills and the opening up of two alternative hydraulic channels took load off the bridge, but nevertheless the performance of the bridge is noteworthy.

About 100 m 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 19). The unprotected embankment fill appeared to be a granular material. As the wave overtopped the embankment, it displaced the tracks (Figure 20) and significantly scoured and removed the upper 4-5 m of the fill. Apart from the loss of the approach fills, all the bridges in the valley appeared to be intact.

Research Hypotheses

The reconnaissance team took four hypotheses into its bridge survey, and the evidence gained supports three of the four:

- that coastal bridges in tsunami-prone areas should be designed or retrofitted to give resistance to wave inundation,
- that the duration of strong ground motion shaking should be included in the design and retrofit of highway and rail structures,
- that retrofitting reinforced concrete columns should be done with due regard to load path, through the substructure, and
- that skewed bridges are very susceptible to unseating if not adequately restrained at their abutments.

There is clear evidence that bridges can be designed to withstand tsunami inundation. This can be seen, for example, by comparing the performance of the Koizumi Bridge with that of the Nijyu-ichihama Bridge. The superstructure of the Koizumi Bridge was supported on bearings and the spans were swept away, whereas the superstructure of the Nijyu-ichihama Bridge was integrally tied to the abutments, and the span survived being overstapped. Other examples illustrate the superior performance of bridges with integral connections, such as the Nijyu-ichihama rail bridges.

With respect to the first hypothesis, we think that coastal bridges should be designed for tsunami effects, and that it can be done without undue cost. In addition to integral connections, venting trapped air, protecting fills against erosion, designing foundation caps to minimize scour effects, and providing deeper foundations to allow scour without collapse are all strategies that provide a first line of defense at a reasonable cost.

The degree to which the duration of this earthquake exacerbated the damage cannot be determined convincingly without analysis (which has yet to be done), but circumstantial evidence suggests that 18 elastomeric bearings in the Sendai Tobu Viaduct ruptured completely because there was time for the stoppers to fail and then the bearings. A shorter earthquake (of the same frequency content and peak ground acceleration) might have failed only the stoppers and none of the bearings. It is agreed that duration deserves further study, but the extent to which it can be (should be) included in design, is not yet clear.

The load path hypothesis appears to have been prompted by reported damage to one of the Shinkansen viaducts. Unfortunately, the reconnaissance team was not able to visit this structure, but did observe the importance of load path in other
structures such as the Shida Bridge, in which (as noted above) the lack of redundancy in the load path from the two-girder superstructure to the bearings led to a domino set of bearing failures until it reached a set of fixed bearings on a nonductile, infill pier. The accumulated longitudinal inertia load from the superstructure to this one pier damaged the pier, but there was no collapse because, as the pier deflected, the nearby abutment was engaged. Some of the inertia load was then diverted away from the pier, but it damaged the abutment pedestals in the process. This is one of many illustrations of the importance of understanding the load path from the superstructure to the substructure, through a set of vulnerable steel bearings, to nonductile piers. The failure of the elastomeric bearings in the Sendai-Tobu Viaduct also exemplifies the need to understand the load path, but in a different kind of structure. As noted above, many factors contributed to the failure of the bearings, but the root cause is thought to be the construction of on- and off-ramps for a new interchange. Changing single-column piers into two-column frames stiffened this section of the viaduct, modifying the original load path and concentrating load at the interface between the modified and unmodified sections. High lateral shear exceeded the capacity of the steel stoppers, which failed and transferred load to the bearings, which, in turn, failed under combined tension and shear. None of the bridges investigated was skewed, and the team did not learn of any skewed bridges damaged by ground shaking. Thus, the last hypothesis could not be tested in this particular earthquake.

Preliminary Conclusions

The following conclusions are based on observations made and data recovered during the reconnaissance; however, they are of a speculative nature due to the small number of bridges investigated and the absence of field data at each site, such as foundation and soil details, bearing and tie-down details, superstructure weights, wave heights, and velocity profiles. These conclusions are therefore likely to change as additional data become available and further studies are completed.

1. Despite the magnitude of this earthquake, bridge damage outside of the coastal zone was not heavy. This is believed to be due in part to the distance from the epicentral region, and in part to the conservative form of capacity design implemented in Japan for new bridges in the 1990s. Furthermore, an active retrofit program was undertaken for older bridges following the Kobe earthquake in 1995, especially on the national highway system.

2. Aftershocks that follow large-magnitude earthquakes can themselves be large and damaging, and the damage to some bridges was aggravated in subsequent aftershocks.

3. Retrofitting is an effective way to minimize earthquake damage in older bridges. Most of the observed structural damage was to older bridges that had not been retrofitted, or only partly so. It is recommended that owner agencies be strongly encouraged to accelerate their retrofit programs.

4. With the exception of several spans in the Sendai area, elastomeric bearings performed well and considerably better than older-style, steel bearings. The poor performance of the Sendai-Tobu bearings needs to be fully analyzed quickly because of the growing worldwide use of them as movement and isolation bearings.

5. Damage to several older, unretrofitted, bridge piers was concentrated in the reinforcement termination zone, and this vulnerability should be considered when prioritizing bridges for retrofitting. Bridges damaged in this manner are susceptible to additional damage during aftershocks that will lead to longer repair times, more restrictive load limits, or even closure during repair.

6. 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 to replace one or more spans.

7. 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. The effect of duration on structural response should be investigated.

Acknowledgments

We are grateful for the collegiality of the Japan team members, the free exchange of technical data, the excellent field logistics, and the many useful discussions about bridge performance.

References