3 TRANSPORTATION SYSTEMS

EXECUTIVE SUMMARY

Although there was quite a bit of damage to the transportation system during the earthquake, there were no casualties and no structural collapses. There were quite a few landslides that may permanently close some roads, but fortunately, no one was injured. Most of the damage was the result of problem soils, particularly soils that had settled behind abutments, laterally spread along riverbanks, or slid along mountain passes. Also, several tunnels sustained damage that may require monitoring. There was damage to a few long-span bridges whose bearings, cross bracings, and other elements failed during the earthquake. These bridges were quickly inspected and reopened to traffic after it was determined that the damage didn't affect the load-carrying capacity of these bridges. Several long span bridges had temporary supports put under the girders to catch them in case of strong aftershocks. Similarly, there was damage to the ports, harbors and coastal structures due to quay walls being pushed out into the sea, but they were still usable for the recovery efforts.

As is well known, this area of Japan is covered by young alluvial soils that are extremely vulnerable to strong ground shaking. During earthquakes this soil caused damage to adjacent transportation structures because of liquefaction, settlement, and spreading. Even bridges supported on pile foundations had to be quickly repaired before being reopened to traffic due to settlement of the approaches. Also, many roads were cut into unstable slopes that slid downhill during the earthquake. Some slopes can be stabilized with retaining wall systems, but many of these coastal and mountain roads are on huge unstable slopes that will continue to slide and will be a problem that is only exacerbated by earthquakes. We observed few crib wall failures during this (or during previous earthquakes) suggesting that flexible retaining wall systems (that can distort without failing) may be a good choice in earthquake country.

3.1 Description of System

There is a well-developed and redundant transportation system (rail, road, ship, and air) around the area impacted by the earthquake. The East Japan Railway Company (JR East) has Shinkansen lines (dashed lines in Figure 3.1), regular JR lines (black lines) as well as freight branches operating in the region. There are also a number of other rail lines operating in the area (shown in gray). There is also bus service that competes with the trains across the mountains, along the coast, and in town although JR owns some of the longer bus lines. Because buses are more economical, the competition is sometimes fierce. Buses are cheaper and slower than trains. Private vehicles (or rental cars) are another alternative for travel. All of these industries are abundant in the area impacted by the earthquake (southwest Niigata Prefecture). There were no subways in the area of damage but taxicabs were abundant.

This kind of redundancy makes travel easier for commuters and travelers. Transport companies moving containers and other goods had more problems following the disaster. Containers can travel by train or truck as long as there are roads and rails available. They can travel by sea, as long as there is a port with equipment for unloading the containers and cargo. There is a small shipping port in Kashiwazaki (and one at the nearby nuclear power plant that receives shipments of plutonium). However, larger container ports are to the north in Niigata or to the south in Naoetsu. Local air travel is another sensible alternative for getting to or from places in the earthquake impacted prefecture of southwest Niigata. Although there are several nearby airports in Yoyama, Matsumoto, Niigata, Noto, and Komatsu, they still had to move people and goods into the area, once they landed.

The impacted area is composed of headlands to the north and south of Kashiwazaki, requiring long-span crossings. The city of Kashiwazaki is in a flat river valley filled with loose sediments blown in from the sea and deposited by the Ugawa and Sabaishi Rivers. The Japanese Alps are to the east, requiring steep grades and tunnels for both the railway and highway systems. All of these topographies caused problems during the earthquake.

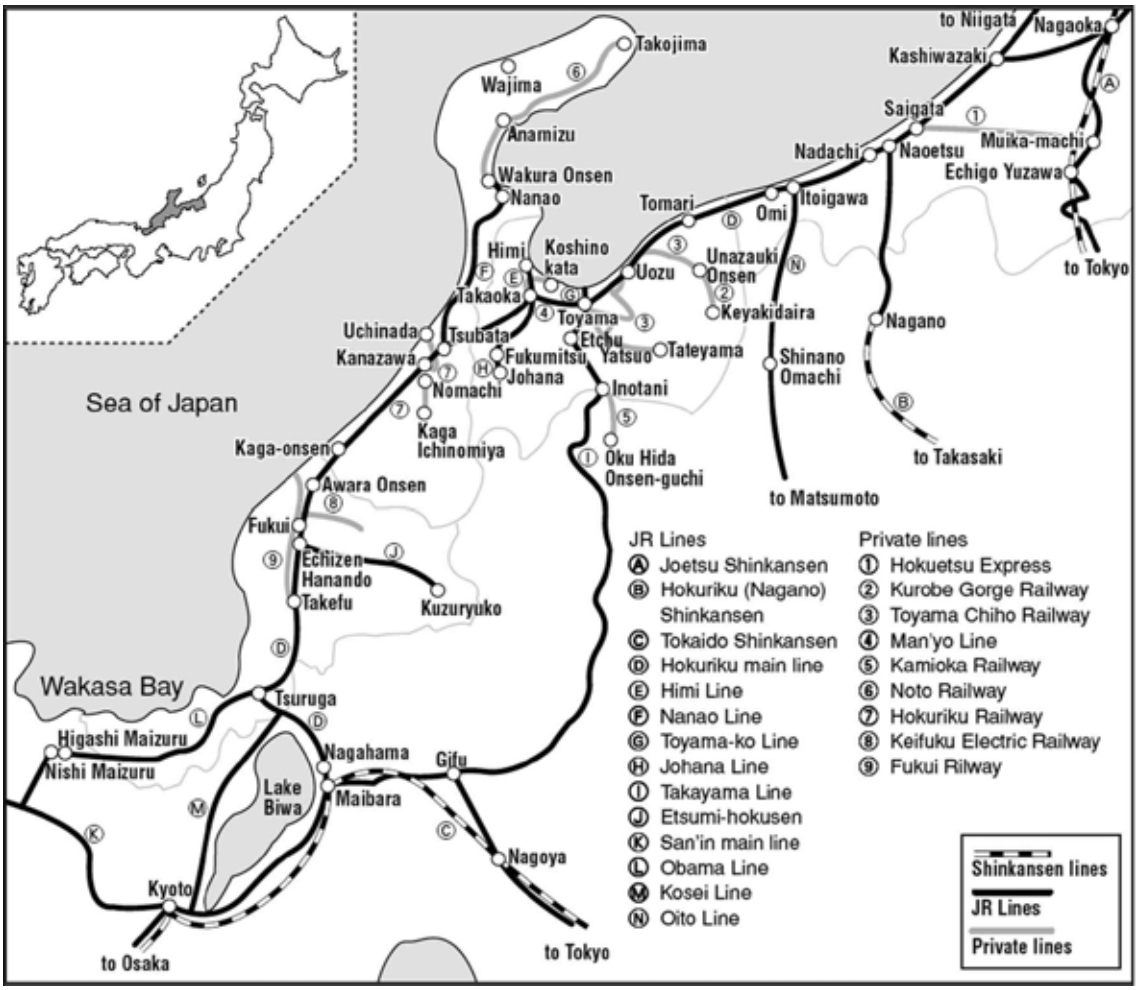

Fig. 3.1. Railways in the affected area

Similar to the rail system, there are private expressways (with rectangular symbols as shown in Figure 3.2), national highways (with triangular symbols), prefectural roads (with hexagonal symbols), and city streets (not shown) in the area.

Fig. 3.2 Roads in the affected area

3.2 Overview of System Performance

Transportation systems are designed to remain in service for smaller, more frequent earthquakes and to protect human life during larger, rare events. No one died due to transportation system failures during (or after) this earthquake, but the systems were put temporarily out of service. The Shin'etsu main rail line was closed due to landslides for several weeks. Owners of expressways, highways, and local roads struggled to remain open to deal with the disaster.

The Ministry of Land, Infrastructure, and Transport (MLIT) played the leadership role for the coordination of traffic in the earthquake damaged region. They provided support to local governments on damaged roads and possible detours (Unjoh, 2007). The Port of Kashiwazaki was seriously damaged but was still able to handle the earthquake response. There have been several earthquakes of about the same size in the same general area over the last few years. However, it was only the July 16th earthquake that caused extensive damage to Kashiwazaki City. Thus, this earthquake is thought to be the maximum event for this site, and consequently the performance of bridges and other transportation facilities performed well.

3.3 Performance of Transportation Facilities.

The following sections provide information collected on the performance of highway bridges, tunnels, retaining walls, railways, ports, and other transportation facilities during the earthquake. When GPS coordinates of the sites are available, they are provided in parenthesis with the name of the location.

3.3.1. Performance of Bridges

The GEER and the TCLEE teams investigated bridge damage on the Hokuriku Expressway, on National Highway 8, on bridges crossing the Sabaishi and the U Rivers, and on prefectural and city bridges. Most of the damage was the result of problem soils that caused lateral spreading or settlement to bridge approaches, making it impossible to use the bridge until repairs could be made. Damage to the bridges consisted of some combination of cracks in the abutments, vertical offsets at the approaches, and lateral offsets in the bearings (Table 3.1). On longer bridges, there was also some damage to structural elements. Nearly every bridge was in service at the time of the investigations.

Table 3.1: Bridges observed during reconnaissance.

Name	Latitude (deg)	Longitude (deg)	Length (m)	Liquefaction	Landslides	Lateral Spreading	Bearing Distortion	Approach Offset	Severe Bridge Damage	Moderate Bridge Damage	Minor Bridge Damage	No Bridge Damage
Hokuriku Expressway												
Hachisaki Bridges	N37.315°	E138.437°	5-span 185 m			x	x	x		x		
Omigawa Bridges	N37.343°	E138.482°	3-span 300 m				x				x	
Yoshiigawa Bridges	N37.399°	E138.625°	7-span 200 m					x			x	
-	N37.4225°	E138.6412°	130 m					x				
National Highway 8												
Toyota Bridges	N37.389°	E138.592°	3-span 160 m				x	x		x		
Route 522	N37.364°	E138.570°	2-span 10 m							x		
Ugawa River Bridge	N37.3605°	E138.549°	2-span 60 m				x					
Yoneyama Bridge	N37.346°	E138.488°	5-span 280 m		x						x	
Yonahime Bridge	N37.3168°	E138.4348°	1-span 120 m		x		x			x		
Sabaichi River Crossings												
Ansei Bridge	N37.3864	E138.5693	106 m			x		x				
Heisei Hashi	N37.3894°	E138.5742°	22 m	x		x	x	x			x	
Heisei O-Hashi	N37.3888°	E138.5767°	118 m	x		x	x	x			x	
Kaiun Bridge	N37.3918°	E138.5806°	111 m	x		x	x	x			x	
Railway Bridge	N37.3917°	E138.5899°	65 m									x
Nagoma Bridge	N37.3904°	E138.5908°	112 m	x		x	x	x		x		
Uehara Ni Bridge	N37.3900°	E138.5960°	65 m			x	x	x		x		
Uehara Ichi Bridge	N3895°	E138.5966°	100 m			x	x	x		x		
Tsurugi Bridge	37.3921°	E138.6008°	43 m			x		x		x		
Genji Bridge	N37.3845°	E138.6017°	95 m			x	x	x		x		
U River Crossings												
Rinko Yasaka	N37.3675°	E138.5433°	81 m			x	x	x		x		
Yasaka Bridge	N37.3658°	E138.5438°	62 m			x						
Ukawa Bridge	N37.3641°	E138.5462°	66 m			x						
Osu Bashi	N37.3635°	E138.5478°	66 m			x	x	x		x		
Railway Bridges	N37.3619°	E138.5493°	70 m – 2 span									x
Steel Bridge	N37.3547°	E138.5515°	100m – 4 span									x
Other Bridges												
Railway Bridge	N37.36896°	E138.56530°	20 m								x	
Rte 352 by NPP	N37.4248°	E138.6076°	20 m					x				
Highway 116 Bridge	N37.446°	E138.638°	20 m					x				
Highway 116 Bridge	N37.492°	E138.699°	20 m					x				

Hokuriku Expressway

The Hokuriku Expressway (Fig. 3.3) is owned by the Japanese government and operated by NEXCO (the East Nippon Expressway Company Limited). There was considerable damage to this highway during the earthquake. However, four hours after the earthquake, emergency repairs were completed and the emergency traffic could use the expressway, and 56 hours later, it was reopened to ordinary traffic. This helped with the transfer of people and goods to the earthquake affected area. There were at least 17 locations where damage was severe enough to temporarily close parts of the expressway. There was damage to bridges, tunnels, roads, and to other highway facilities. The damage extended from 338 KP to 416 KP, for a distance of 78 kilometers.

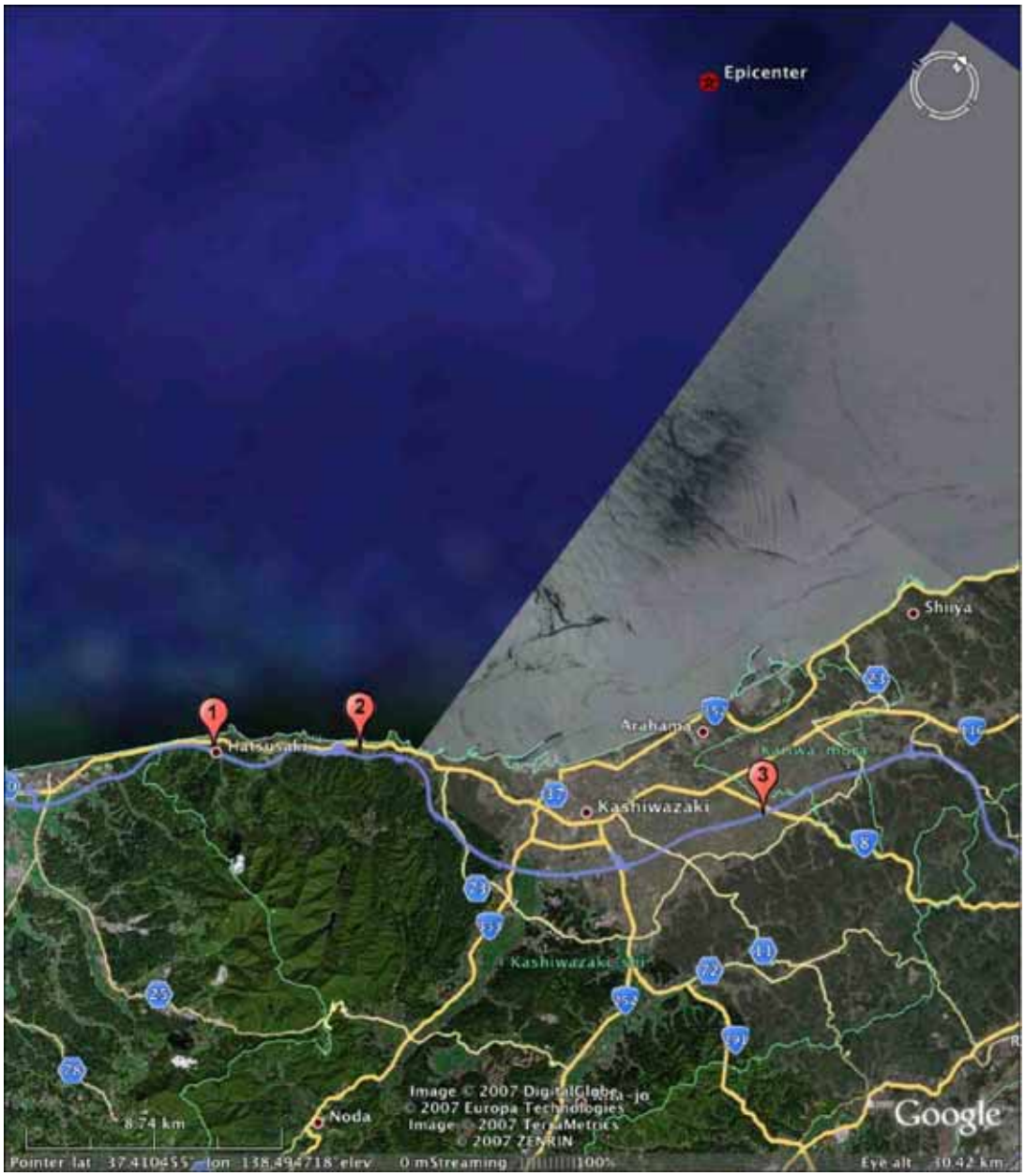

Fig. 3.3 Locations of Bridge Damage on the Hokuriku Expressway.

(1) Hachisaki Bridges (380.4 KP) N37.315°E138.437°

These are two parallel five-span concrete 'I' girder bridges on seat-type abutments and tall (30m+) pier walls (Fig. 3.4). The three northern spans are continuous over steel bearings with an expansion joint at the end of the third span. The piers are on piles except where there is rock near the surface. The bridges are just north of the Yoneyama tunnel, which was also damaged. Our team went under the bridge to study the bottom of the piers but the undergrowth was so thick we couldn't get close to them.

During the earthquake, the embankments moved downhill, exposing the wingwalls (Fig. 3.5). The girders are supported by fixed bearings at the abutment and it is thought that the shear force exceeded the shear strength of girder at the bearing connection. There was a spall at the bottom of the girders about 0.6m from the abutment backwall and on the other side of the bearing. Perhaps, as the abutment moved downward, large shear forces in the girders caused them to spall(Fig. 3.6). The damage occurred at the north end of the west bridge (small cracks were observed on the east bridge).

Fig. 3.4. Looking south at the Hachisaki Bridges.

When we arrived at the scene (on August 5th), both directions of traffic had been diverted to the east bridge, they had finished driving sheet piles around the abutment to stabilize the soil, and were casting a large pad at the west bridge so that they could jack up the southern three spans and repair the girders and bearings.

Fig. 3.5. Damage at north end of the west Hachisaki Bridge.

Fig. 3.6. Closer view of girders on the west Hachisaki Bridge.

(2) Omigawa Bridges (386.3 KP) N37.343° E138.482°

These are two haunched, three-span, segmentally constructed cast-in-place, concrete box girder bridges on very tall hollow, reinforced concrete piers (Fig. 3.7). Like several other bridges in the same area, the superstructures moved enough during the earthquake to break the fixed steel bearings (Fig. 3.8). Perhaps the weakest member, in this case is the

steel bearings that acted as fuses to protect the structure from more catastrophic damage. Jacking up these long, continuous superstructures is no easy task, and jacking them back transversely is also a lot of work, but it's also one of the most common types of bridge damage that occurs during earthquakes.

Fig. 3.7. Omigawa Bridges on the Horukiru Expressway.

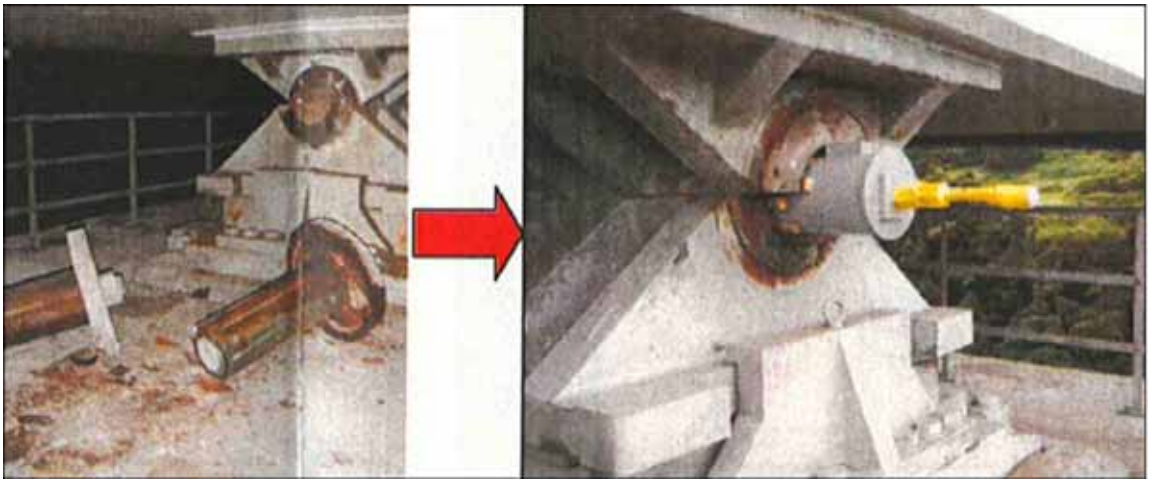

Fig. 3.8. Damage and Repair to Omigawa Bridge (photo courtesy of the Nippon Expressway Company.).

(3) Yoshiigawa Bridges (402.2KP) N37.399° E138.625°

These are two, long steel I girder bridges on single column bents and seat-type abutments. The bridges were being retrofit at the time of the earthquake (Fig. 3.9). Expressway bridges are built with an approach slab that is pinned to the abutment backwall, but this connection broke due to the ground shaking combined with the approach settlement (Fig. 3.10). The only other noticeable damage was the grout plate that the steel abutment

bearing was seated on had shattered as the steel girders banged against the steel longitudinal keys that were to limit movement (Fig 3.11).

Fig. 3.9. Looking under the Yoshiigawa Bridges (and at the piers being retrofitted).

Fig. 3.10. Damage to approach slab and embankment at Yoshiigawa Bridges.

Fig. 3.11. Bearing at south abutment of Yoshiigawa Bridges.

Sand was placed at the back of the approach slab and new asphalt was placed to create a gentle ramp back onto the bridge (Fig. 3.12). There was a lot more of this kind of approach damage on the Hokuriku Expressway and on many other roads and highways, especially where they crossed rivers.

Fig. 3.12 Repaired embankment and approach slab at Yoshiigawa Bridges.

There was a lot of approach settlement damage but the expressway was reopened (with some lane limitations to traffic) 56 hours after the earthquake, which allowed all vehicles to use the expressway.

National Highway 8.

National Highway 8 (Fig. 3.13) runs north along the coast until it reaches Kashiwazaki where it goes eastward through foothills to Nagoaka. The highway had damage along 60 km of its length during the earthquake. There were several landslides and slip-outs as well as damage to long-span bridges and river crossings. The landslides were at much further distance from the epicenter, suggesting that it takes much less ground shaking to disturb unstable rocks and soil.

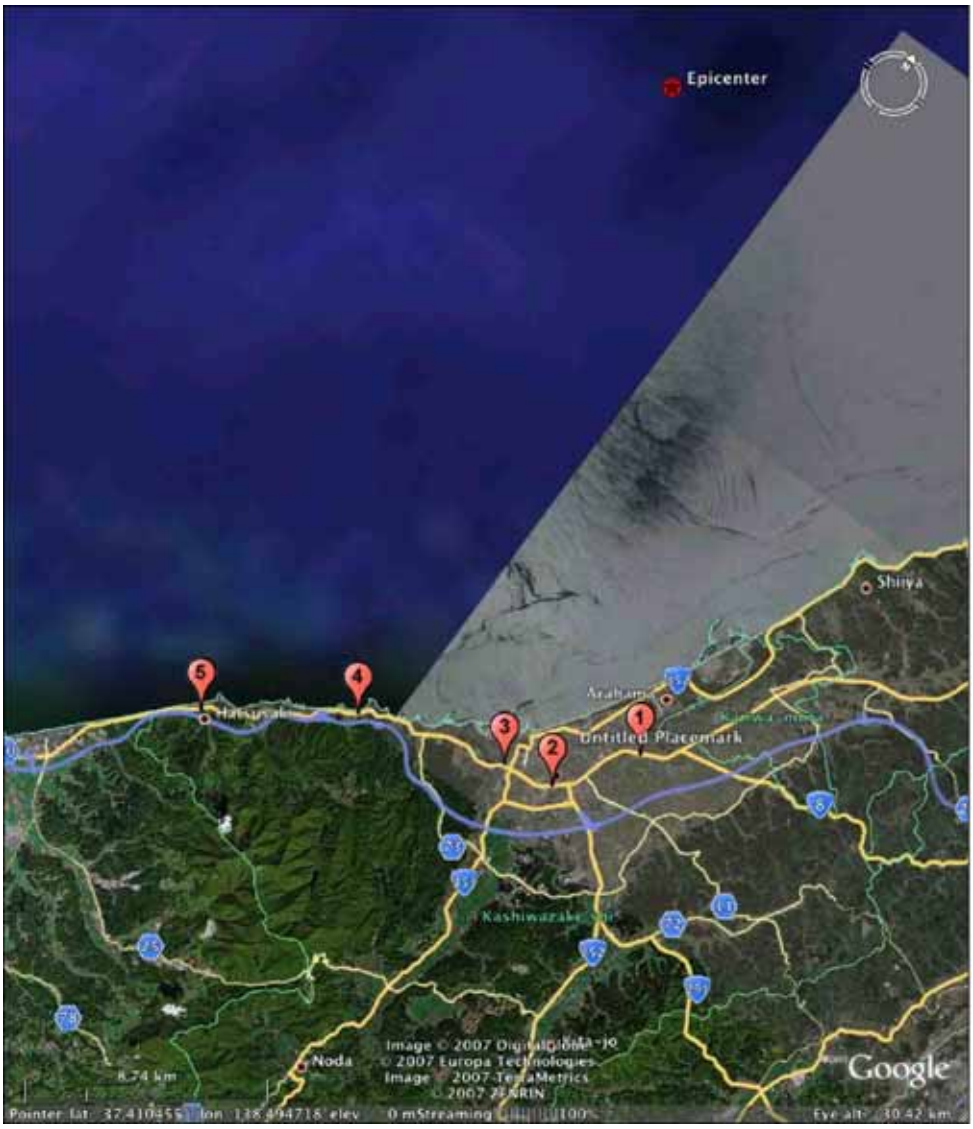

Fig. 3.13. Locations of Bridge Damage on National Highway 8.

(1) Toyota (Sabaishi River) Bridges (N37.389° E138.592°)

Two structures carry vehicles up and over the Sabaishi River on Highway 8. The river crossing has a 3 span continuous steel (highly skewed and curved) I girder superstructure on seat-type abutments and single column bents (Fig. 3.14). The deck is 160m long with a 12m width and is supported on tall elastomeric bearings with transverse shear keys and concrete backwalls to limit movement. The embankment behind abutment #1 (on the west end of the bridge) is supported by mechanically stabilized retaining walls that were undamaged by the earthquake. However, both bridge approaches settled about half a meter during the earthquake. A rigid box structure, that allowed vehicle access through the west bridge embankment, was now half a meter above the road surface, closing the highway (Fig. 3.15). The riverbanks and abutments moved towards the river and the superstructure moved to the northwest (Fig. 3.16). As a result of this movement the elastomeric bearings experienced some shear (Fig. 3.17). However, because the bearings take up to 70% shear strain for temperature change, 150% for a mild earthquake, and 250% for large earthquake they still have much more shear capacity.

Fig. 3.14. Three span steel girder Toyota Bridge over Sabaishi River.

Fig. 3.15. Settlement of west embankment (Courtesy G. Watanabe, Tokyo Institute of Technology).

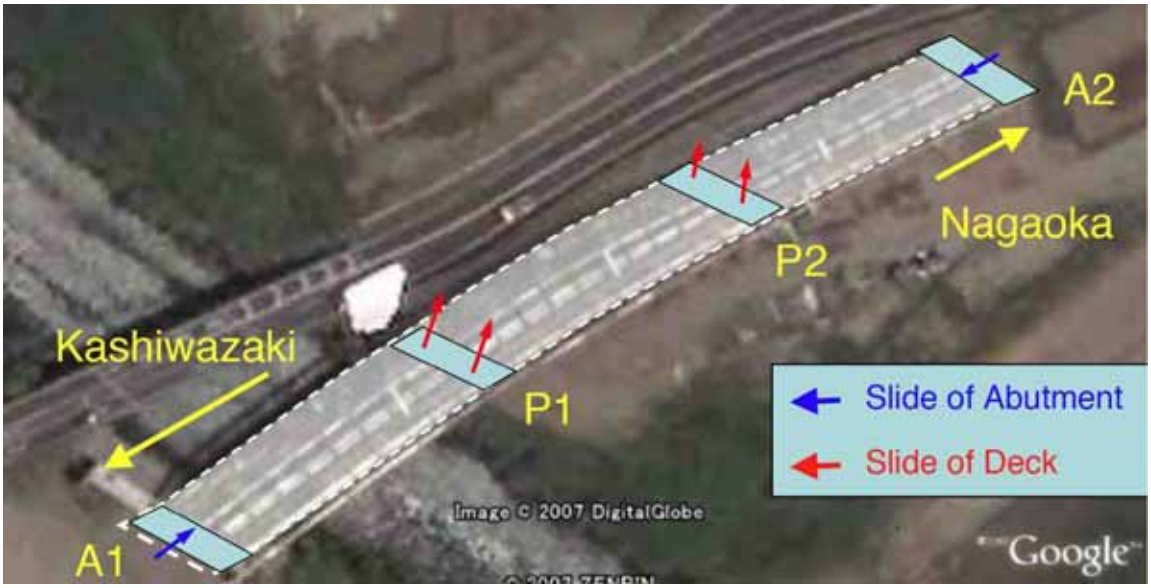

Fig. 3.16. Movement of Toyota Bridge (Courtesy G. Watanabe, Tokyo Institute of Technology).

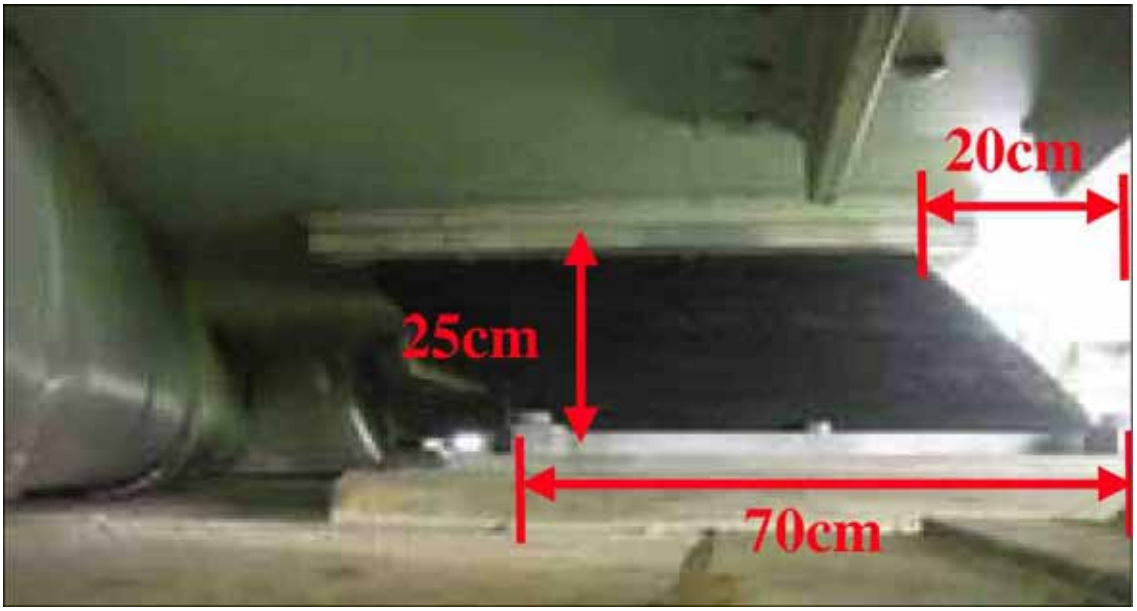

Fig. 3.17. Displacement of Bearing Bridge (Courtesy G. Watanabe, Tokyo Institute of Technology).

(2) Highway 8 (Railroad & Rte 522) Bridges N37.364° E138.570°

In downtown Kashiwazaki, on Highway 8 there are a couple of older bridges that are beginning to be maintenance problems. However, the only noticeable damage from the earthquake was some embankment damage behind the abutments on the Route 522 and the Japan Rail Bridges (Fig. 3.18 and 6.19).

Fig. 3.18. Damage and repairs where Highway 8 crosses Route 522 Bridge (Courtesy G. Watanabe, Tokyo Institute of Technology.

Fig. 3.19. Damage and repairs where railway crosses Highway 8 Bridge (Courtesy G. Watanabe, Tokyo Institute of Technology.

(3) Highway 8 over the Ugawa (N37.3605° E138.549°)

This is a two-span concrete I girder bridge on a pier wall and seat-type abutments (Fig. 3.20). Although bridges closer to the mouth of the Ugawa River sustained damage due to

lateral spreading, this bridge (about 1.2 km upstream) had just a little bit of embankment settlement that was quickly repaired (Fig. 3.21). Just north of the bridge were several manholes raised above another roadway's surface and a nearby stonecutter's shop had toppled statuary suggesting a PGA > 0.25g. and that liquefaction was a problem at this site. However, the embankment settlement was only about 10 cm and continued to decrease upstream, suggesting better soil and/or weaker accelerations.

Fig. 3.20. Highway 80 Bridge across the Ugawa.

Fig. 3.21. Slight (10 cm) settlement at bridge approaches.

(4) Yoneyama Bridge (N37.346° E138.488°)

This is a 280 meter long steel girder bridge (the longer spans are steel boxes and the shorter spans are steel stringers) supported on tall, four-legged steel towers, shorter two-legged piers, and seat-type abutments (see Fig. 3.22 and 3.23).

The bridge had been retrofit before the earthquake, which may have helped reduce the amount of damage that occurred. Lateral and vertical restrainers and stoppers were added at the superstructure (Fig. 3.24). The steel towers had been filled with concrete and stiffeners and doubler plates had been added to make the towers very stiff and strong. The bearing damage was very slight. It was just the failure of the stopper key element and deformation of the pin-cap, which did not affect the bearing function (Fig. 3.25). Other damage occurred as a result of the superstructure pounding against the abutment backwall.

Shikegi Unjoh (from Japan's Public Works Research Institute) said, 'The Yoneyama bridge had seismometers but they were of an older type and were out-of-order'. There was another new seismometer at ground level that recorded the free field motion during the July 16 main shock. Dr. Iwasaki of the Public Works Research Center (PWRC) wrote that it recorded a PGA of 0.65g. Dr. Iwasaki also mentioned that he wrote several papers about this bridge after the 1964 Niigata earthquake (Iwasaki, 1972) (Kuribayashi, 1967). One paper, "Response Analysis of Civil Engineering Structures Subjected to Earthquake Motions" is available at: http://www.fujipress.jp/JDR/open/DSSTR000100020012.pdf.

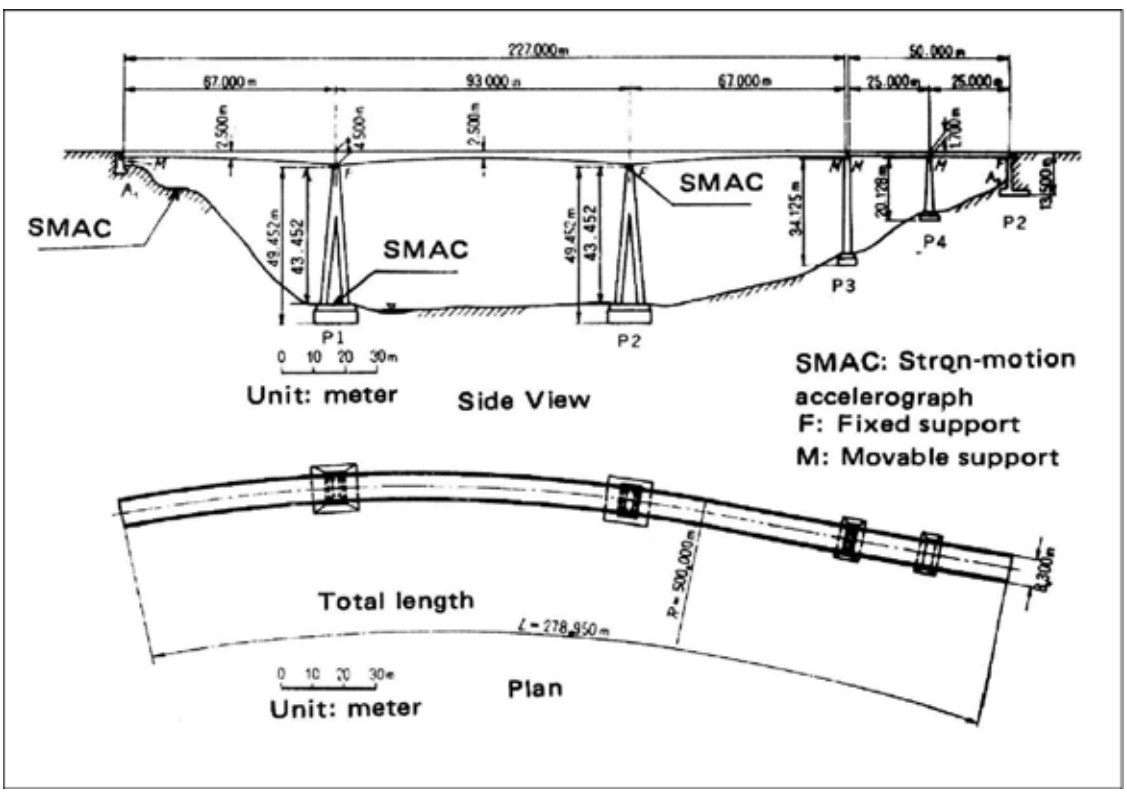

Fig. 3.22 Elevation and Plan Drawings for Yoneyama Bridge (Iwasaki, 1972).

Fig. 3.23. Yoneyama Bridge and landslide.

Fig. 3.24. Retrofitted bent cap at Yoneyama Bridge (Courtesy of S. Unjoh, PWRI).

Fig. 3.25. Deformation of Bearing Pin-Cap caused by uplift force (Courtesy of S. Unjoh, PWRI).

(5) Yonahime Bridge (N37.3168° E138.4348°)

This is a 120 m long through truss arch, crossing over a narrow canyon (Fig. 3.26) south of Kashiwazaki. We saw a landslide scarp on the uphill side of this bridge, and because it is supported by spread footings, it was probably a combination of ground movement and strong shaking that caused it to rack during the earthquake.

Fig. 3.26. Looking at the struts and cross-bracing of the Yonahime Bridge.

When we visited the bridge (on Aug. 4th) we saw many of its members had buckled (Fig. 3.27) and some of its connections were damaged (Fig. 3.28) along with its bearings (Fig. 3.29). Still, the bridge remained in service. Metal boxes were placed around the bearings to support the bridge in case a strong aftershock occurs. This is a very common bridge type. However, the struts and bracings need to be designed against lateral wind force and lateral earthquake force. It appears that the earthquake force during this earthquake exceeded the design force. A deck arch bridge, the Agewa Bridge immediately to the north was undamaged by the earthquake (Fig. 3.30). It might be possible to retrofit the Yonahime bridge for large earthquakes, possibly by supporting it on isolation bearings. Certainly, these fixed bearings, which we saw damaged on several of the longer span bridges, are a lot of trouble to replace.

Fig. 3.27. Buckled cross bracing on Yonahime Bridge.

Fig. 3.28. Torn cross bracing connection on Yonahime Bridge.

Fig. 3.29. Damage of set bolts and the addition of steel box supports in case of aftershocks

Fig. 3.30. Undamaged Agewa (deck arch) Bridge just north of damaged Yonahime Bridge.

National Highway 116.

There were about eight kilometers along National Highway 116 north of Kariwa Village and south of Funabashi (Fig. 3.31) with bridge damage during the earthquake. Highway 116 is built across a narrow river valley on one to two meters of fill. The river sediment and fill laterally spread, settled, and liquefied during the earthquake. Photos and reports of damage to Highway 116 were obtained from the Ministry of Land, Infrastructure, and Transportation (MLIT).

After the earthquake, southbound traffic on Highway 116 at Shinbo was backed up and there was no northbound traffic due to the settlement of the embankments of a river crossing (Damage Location 1) as shown in Figure 3.32. Similar damage due to soil settlement, liquefaction, and lateral spreading can be seen in Figure 3.33 to Figure 3.37 (taken by MLIT).

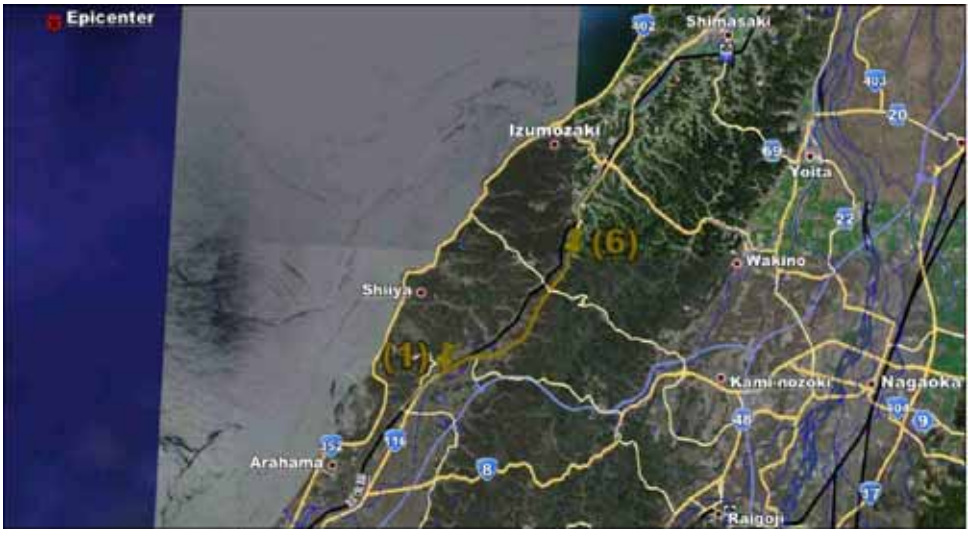

Fig. 3.31. Map showing limits of damage to Highway 116 during the earthquake.

Figs. 3.32 (1) Highway 116 (N37.446°E138.638°) – 0.6 m settlement - repaired by 4PM. (Courtesy of MLIT)

Figs. 3.33. (2) Settlement at box culvert on Highway 116. (Courtesy of MLIT)

Figs. 3.34. (3) Settlement at river crossing on Highway 116. (Courtesy of MLIT)

Fig. 3.35. (4) Settlement and cracking of pavement on Highway 116. (Courtesy of MLIT)

Fig. 3.36. (5) Highway 116 - Bridge approach settlement repaired by 8PM. (Courtesy of MLIT)

Figs. 3.37 (6) Highway 116 (N37.492°E138.699°) Large settlement at bridge embankment. (Courtesy of MLIT)

Sabaishi River Crossings

The bridges along the Sabaishi River consisted primarily of continuous steel I-girder superstructures resting on elastomeric bearings on reinforced concrete piers and abutments. Lateral spreading pushed the abutments toward the river, causing a permanent offset in the elastomeric bearings with the bottom of the bearing moving toward the river relative to the top of the bearing. Offsets in the bearings ranged from 7 cm to 17 cm, and in some cases the bearings had become unbonded from the steel plates in a small region near the edges of the connections. In some cases, some small transverse bearing displacements were evident, though the primary mode of deformation was longitudinal. Cracks up to 1 cm wide were observed in many of the concrete abutments at the 90° angle where the back wall connects to the horizontal platform on which the bearings are connected. Vertical offsets at the approaches of the damaged bridges ranged from about 5 cm to 30 cm, and asphalt or gravel backfill was often placed to permit cars to traverse the bridge. Some bridges that had settlement of the approach soils were functional without any repair required at the approach because the settlement occurred that the approach soil ramped up onto the bridge rather than forming a vertical offset. Utilities attached to the side of the bridge were often damaged near the abutments. Several of the bridges crossing the Sabaishi River in Kashiwazaki (Fig. 3.38) had settlement of their approaches due to lateral spreading of the river banks. Typically the abutments (on piles) would remain at their original height, but they were pushed against the superstructure by the lateral movement of the surrounding soil.

Fig. 3.38 Map of Sabaishi River Bridges

The bridge carrying National Highway 352 traffic across the river had little damage from the earthquake. However, the next two bridges (Heisei and Heisei O'Hashi) crossed onto an island (Fig. 3.39) composed of loose fill, which liquefied during the earthquake.

Fig. 3.39. An island in the meander of the Sabaishi River liquefied.

(1) Heisei Hashi (N37.3894° E138.5742°)

The smaller of the two bridges crossing the island, Heisei Hashi is on the north end. It is a single-span concrete box girder bridge on seat-type abutments.

During the earthquake, the south bridge embankment settled as the bank spread into the river. The wingwalls weren't attached to the abutment (which was on piles) and moved with the embankment (Fig. 3.40). The lateral spread was the result of liquefaction of the island (Fig. 3.41). Figure 3.42 shows about 400 cm settlement of the south embankment.

Fig. 3.40. Looking southeast at Heisei bridge abutment, settled embankment, and the island.

Fig. 3.41. Looking northwest at liquefaction of island and the Heisei Bridge.

Fig. 3.42. Looking north from island at Hashi bridge deck.

(2) Heisei O'Hashi (N37.3888° E138.5767°)

This is a continuous, two-span, concrete box girder bridge supported on seat abutments and a pier wall (Fig. 3.43). During the earthquake, the north bridge embankment settled about 40 cm (Fig. 3.44). There were also signs that the bridge underwent shaking and

pounding (Fig. 3.45). Although the abutments were on piles, the bearings were distorted, indicating that the abutments had been pushed toward the river (Fig. 3.46).

Fig. 3.43. Heisei O'Hashi

Fig. 3.44. Settlement of north bridge approach.

Fig. 3.45. The superstructure and bearings moved back and forth, cracking the abutment.

Fig. 3.46. Signs of movement of the abutments included distortion of bearings.

Note that unlike the north Heisei Bridge, there are apparently piles under the short wingwalls on the south Heisei Bridge. Figures 3.47 and 3.48 show the wingwalls at the same height as the abutment and they also show signs of pounding between them.

Fig. 3.47. Pounding between abutment and wingwall.

Fig. 3.48. Settlement of north bridge abutment.

(3) Kaiun Bridge (N37.3918° E138.5806°)

About 1/2 km upstream from the Heisei Bridges sits the four span Kaiun Bridge (Fig. 3.49), which had a 1 meter settlement of the south approach (Fig. 3.50).

Fig. 3.49. The four, steel 'I' girder span Kaiun Bridge on pier walls and seat abutments.

Fig. 3.50. About one meter settlement of south Kaiun bridge approach.

Like the other Sabaishi bridges, this settlement was caused by the riverbank spreading toward the river (Fig. 3.51). Although the expansion joint, the abutment, and the bearings should have been isolated from this movement by the pile supports, they were distorted as shown in Fig. 3.52 during the earthquake, perhaps due to the ground shaking.

Fig. 3.51 The soil around the south abutment moved towards the river during the earthquake.

Fig. 3.52. Distortion of expansion joint, bearings, and abutment on Kaiun Bridge.

(4) Nagomi Bridge (N37.3904° E138.5908°)

This is a three span steel girder bridge on pier walls and seat-type abutments (Fig. 3.53). There was evidence of fine grain soil being ejected at depth to the ground surface (along with settlement) at the saturated riverbanks (Fig. 3.54). There was damage to the abutment backwall (Fig. 3.55) and to the bridge approaches (Fig. 3.56).

Fig. 3.53. Nagomi Bridge (Courtesy of Scott Brandenberg).

Fig. 3.54. Soil settled around pier in riverbank (Courtesy of Scott Brandenberg).

Fig. 3.55. Distorted bearing and cracked abutment backwall (Courtesy of Scott Brandenberg).

Fig, 3,56. Repairs to bridge approach on Nagomi Bridge (Courtesy of Scott Brandenberg).

(5) Genji Bridge (N37.3845° E138.6017°)

This is the last Sabaishi River bridge that had a big settlement at the approach (Fig. 3.57).

Fig. 3.57 Genji Bridge (Courtesy of Scott Brandenberg).

U River (Ugawa) Crossings

Unlike the Sabaishi River (which had lateral spreading for about 5 km upstream), the banks of the Ugawa (a river south of the Sabaishi in Kashiwazaki) laterally spread for only one km from the mouth of the river (Fig. 3.58). This may be due to the more meandering path of the Sabaishi which resulted in more sediment being dropped along the banks. Also, the Sabaishi travels along a large sand dune with poorly compacted soil. Another difference between these two rivers is that the Ugawa has at least 8 meter high levees built up along the banks and much greater flood protection (pumps and gates). Perhaps it is this tendency of the Ugawa to flood that periodically cleans its banks of loose sediment. At any rate, the tall dikes meant that when the banks spread, they carried more material, and caused damage to the slope paving and flood control facilities.

Fig. 3.58 Map of U River Bridges.

(1) Rinko Yasaka Bridge (N37.3675° E138.5433°)

This is a two span concrete I girder bridge on seat abutments and a single column bent with a hammerhead cap at the mouth of the Ugawa. This location had the most lateral spreading on the Ugawa. The entire levee was dragged toward the river (Fig. 3.59) and the abutment was damaged and separated from the wingwall (Fig. 3.60).

Fig. 3.59. Rinko Yasaka Bridge at the mouth of the Ugawa.

Fig. 3.60. Abutment and wingwall damage from the earthquake.

The concrete I girders broke the abutment backwall (Fig. 3.61), the bridge approaches were damaged but quickly repaired (Fig. 3.62), and it appeared that repair work was going on at the center pier (Fig. 3.63).

Fig. 3.61. Backwall-Diaphragm damaged by earthquake.

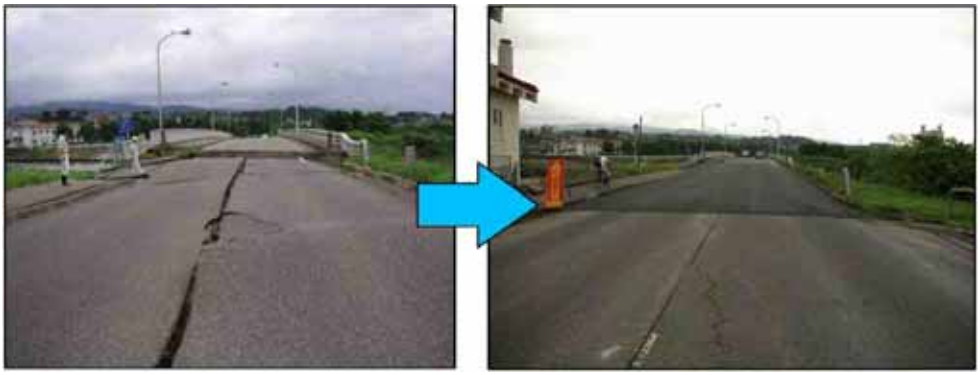

Fig. 3.62. Before and after views of approach and bridge deck.

Fig. 3.63. Pier supporting Rinko Yasaka Bridge.

(2) Yasaka Bridge (N37.3658° E138.5438°)

Like most of the bridges over the Ugawa, this is a two span, simply-supported river crossing, but with steel girders, and supported on a pier wall and seat-type abutments (Fig. 3.64). It is about 200 meters from the more heavily damaged Rinko Yasaka Bridge, but the soil was much firmer at this site, there was less than 100 cm of settlement at the approaches, and no damage to the abutments or wingwalls (Fig. 3.65).

Fig. 3.64. The Yasaka Bridge.

Fig. 3.65. Approach and deck of Yasaka Bridge.

(3) Ukawa Bridge (N37.3641° E138.5462°)

Continuing to walk upstream another 300 meters, we came to the third bridge crossing the Ugawa. The Ukawa bridge is a two (simple) span 'I' girder bridge supported by a pier wall and seat-type abutments (Fig. 3.66). All of these bridges are about 60 meters long. As seen in Figure 3.67 the only damage was some settlement of the sidewalk on the south end of the bridge.

Fig. 3.66. The Ukawa Bridge.

Fig. 3.67. Approach to Ukawa Bridge.

(4) Osu Bashi (N37.3635° E138.5478°)

The fourth crossing, a narrow (6 m wide) two-span concrete box girder bridge on a single-column pier with tall and short abutments on either side (Fig. 3.68) had more damage. Although the south levee moved very little, the embankment between the levee and the bridge settled a meter (Fig. 3.69) and there was damage to the abutment including a leaking waterline (Fig. 3.70). Figure 3.71 provides a closer view of the settled embankment.

Fig. 3.68. Looking east (upstream) at the Osu Bashi.

Fig. 3.69. Looking north at the south approach to Osu Bashi.

Fig. 3.70. The superstructure is supported on a single bearing (note waterline).

Fig. 3.71. Closer view of settlement at south abutment.

(5) Railway Bridges (N37.3619° E138.5493°)

These are two through girder bridges on steel rocker bearings, seat-type abutments, and a pier wall 200 meters upstream from Osu Bashi (Figures 3.72 and 3.73). They are stronger than the road bridges we had previously studied. When we arrived on August 6th, rail crews were re-attaching the rails and there was new ballast on the approaches. However, they went off to lunch before we could ask them if they were repairing earthquake damage. The rails were rusty, suggesting they hadn't been used in awhile, but its possible that the railway was closed due to damage down south at Omigawa Station. The bridge was built in 1997.

Fig. 3.72. Looking under the Railway Bridges.

Fig. 3.73. Looking at rails on the west bridge where the rail crews had left their tools.

(6) Four-span steel bridge (N37.3547° E138.5515°)

This was the last bridge across the Ugawa where we saw a slight sign of approach settlement. It's a four-simple span steel 'I' girder bridge that crosses over the river, the levees, and a river road. We walked past a lot of flood control facilities along the river including the flood gate seen in Figure 3.74. This bridge has a long embankment on the south side. Figure 3.75 shows a very slight settlement at the north approach.

Fig. 3.74 View looking south across the Ugawa River at this steel bridge.

Fig. 3.75. Looking at the north approach.

3.3.2. Performance of Tunnels

When the Geo-Engineering Earthquake Reconnaissance (GEER) team performed their investigation (from July 20th to July 24th), they were unaware of the damage to highway tunnels. It was only after careful inspection of the tunnels by the owners that the extent of the damage was recognized. When TCLEE performed their investigation (August 1st to August 5th), there was more information available, including information on the performance of tunnels.

In Japan, tunnels are more often used than in the US as a way for highways (and railways) to go through mountains rather than over mountain passes. There were hundreds of tunnels in the area impacted by the earthquake, and although most of these tunnels were undamaged, there was undoubtedly more tunnel damage than we had time to investigate.

Bored tunnels are thought to perform fairly well during earthquakes, as long as the material that supports them is uniform along the length of the tunnel. The examples of damage in this section (Fig. 3.76) show how local conditions can have an impact on tunnels and they also show that many of these problems were of a long-standing nature that was exacerbated by the earthquake.

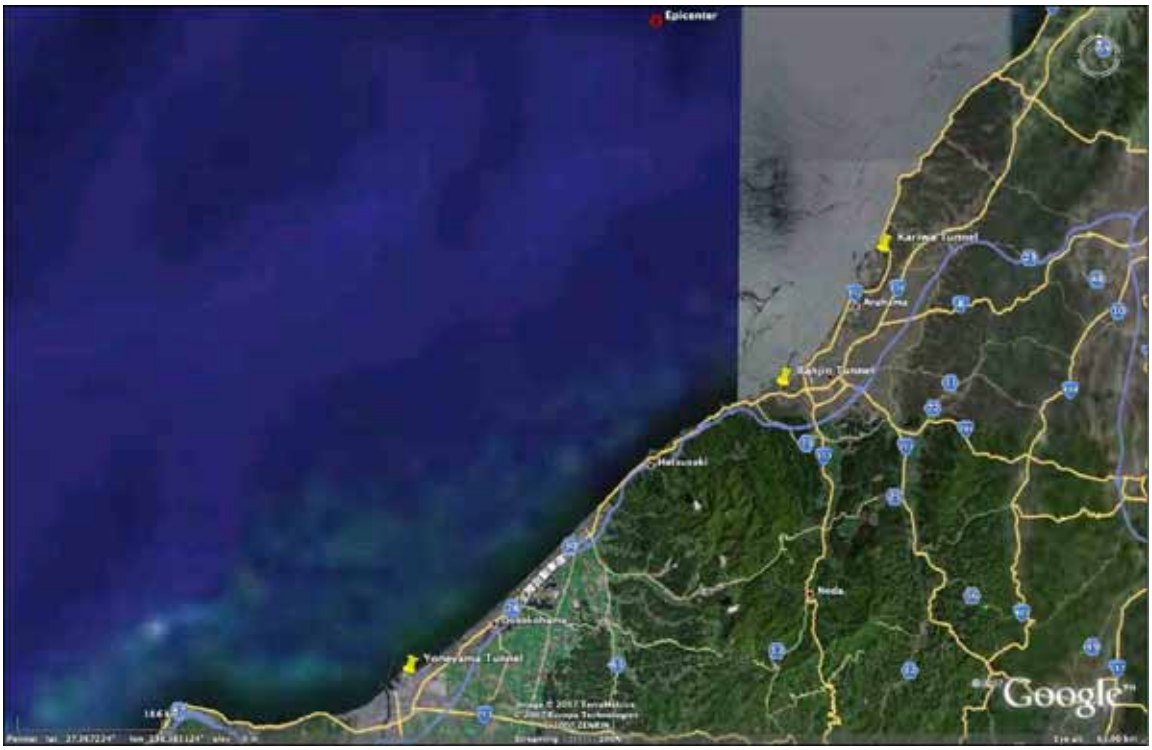

Fig. 3.76 Map showing locations of tunnel damage.

(1) Yoneyama Tunnel (381.8 KP) N37.190° E138.261°

These are two parallel, 1600 meter long tunnels just south of the damaged Hasaki Bridges. The tunnels are 7m in height (from top of roadway to tunnel invert) and 10m wide. The northbound tunnel had 15-20 longitudinal cracks at mid-height of the lining of the northbound tube (Figure 3.77). The cracks on each side of the tunnel were offset by 20 meters, possibly indicating that some geological feature crossed the tunnel at a sharp

angle. The south bound tube was even more heavily damaged, but had already been repaired and opened for traffic. Also, the roadway was pushed upward or one side of the tunnel was pushed downward (Figure 3.78).

Fig. 3.77. Offset cracks in Yoneyama Tunnel.

Fig. 3.78. Differential settlement between tunnel and roadway.

The Yoneyama tunnel goes through the 366 meter high Shiroyama (Shiro mountain) located 30 km SSE of the epicenter and 12 km SE of Kashiwazaki city (see Fig 3.79).

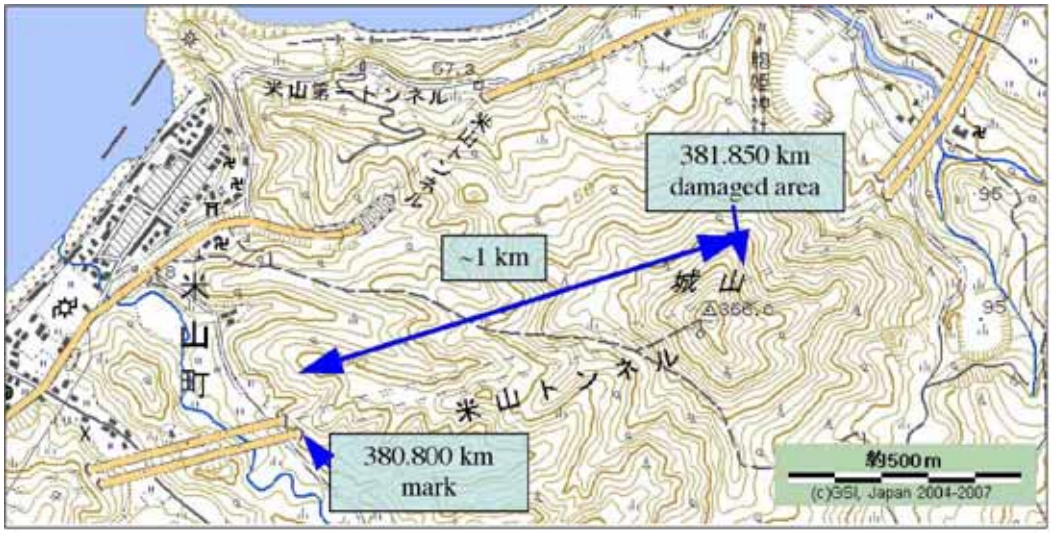

Fig. 3.79. Annotated map showing location of the Yoneyama expressway tunnel (Map by Geographical Survey Institute, Japan.)

In the north bound tube at the 381.870 km mark, approximately 1km from the south entrance, the sound isolation plates were not yet put back in place so we could take the photo shown in Figure 3.77. We observed that horizontal cracks on the left and right side were off-set approximately 23 meters in the longitudinal direction (see Fig. 3.80). From this observation, it is inferred that that some geologic feature may be crossing the tunnel with a 45 degree angle to the longitudinal axis of the tunnel. NEXCO personnel have similar suspicions (Oka, Personal communication). At the time of this writing (August 20, 2007) we are waiting for further information regarding the geological section from the time of construction, which would be helpful to understand the reason for the damage and also possibly give hints how to avoid such damage in the future when constructing tunnels under similar conditions. As also can be seen in Figure 3.77 there are many older repaired cracks that indicate some ongoing deformation in this part of the tunnel.

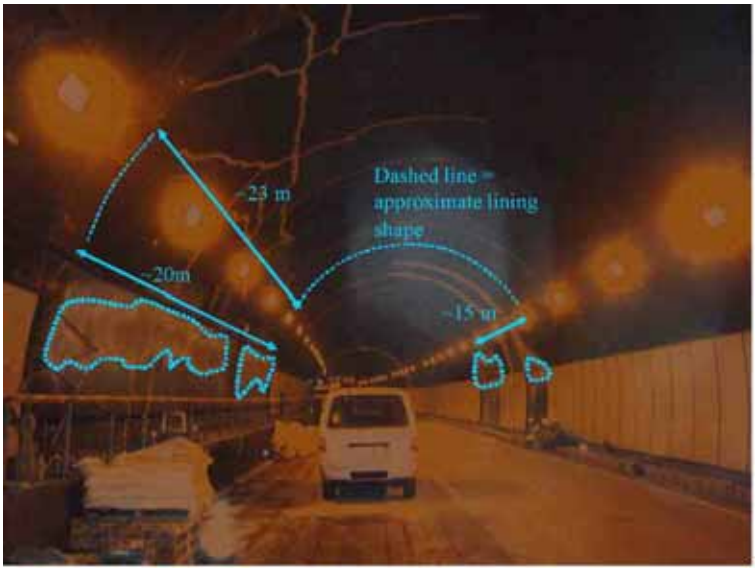

Fig. 3.80 Photo showing horizontal cracks inside Yoneyama expressway tunnel (see text for details.)

The east side of the slab as seen in Figure 3.81 was uplifted approximately 15 cm on the east side north bound tube of the Yoneyama tunnel and the curb stone was pushed forward indicating a compression of the bottom part of the tunnel. Unfortunately we did not measure the total width of the tunnel at the road elevation so we do not know exactly how much the tunnel was compressed, however from the inclination of the curb as seen in Figure 3.81, we estimate it to be more than 5 cm. For reference a cross section of the tunnel as constructed is provided in Figure 3.82.

Fig. 3.81 Slab was uplift approximately 15 cm on the east side north bound tube of the Yoneyama tunnel. The curb stone was pushed forward indicating a compression of the bottom part of the tunnel.

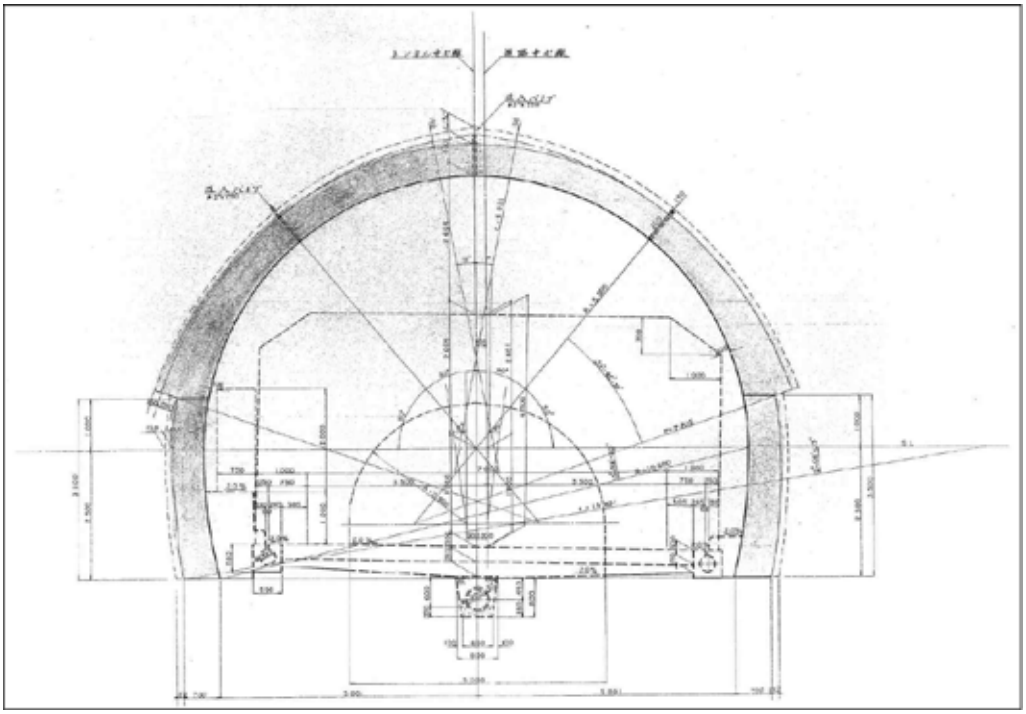

Fig. 3.82 As-built tunnel cross section.

(2) Banjin Tunnel (N37.3598° E138.5318°)

The 308 m long, 10.3 m wide and 6.4 m tall Banjin tunnel is located to the about 1.5 km west to the Kashiwazaki city. Its portal thickness is 1.5 meters as seen in Figure 3.83 and Figure 3.84, which show the tunnel's north and south entrances, respectively. On the south side of the hill is directly below two sets of railway tracks. Soil movement towards the ocean side (down hill) was observed above nearby the concrete covered slope face at the northern entrance (see Figure 3.86). Soil movement induced ground cracks, which were covered with blue plastic sheets in Figure 3.86 a) and indicated by red lines in Figure 3.86b). Cracks beneath sheets are likely much larger than the ones seen in Figure 3.86b). Cracks in concrete covered slope face were also observed. This concrete covered slope face cuts the tunnel at an angle as shown in Figure 3.87. There is large change in rigidity of the material surrounding the tunnel at the location of concrete covered slope face. Since the material changes from soil to concrete and then to air, stress concentrations are likely to have caused the tunnel to crack. The cracks observed in the crown of the tunnel lining are indicated as thicker dashed line in Figure 3.88, whereas the thinner dashed line shows the lining segment joint. Repair was ongoing as we visited the tunnel.

The tunnel suffered cracking at its northwest portal Figure 3.88 due to movement of surrounding concrete covered slope face, which cuts the tunnel at an angle (see Figure 3.87). Furthermore, sound insolating plates on the lining came off due to the shaking and/or tunnel lining deformation.

As seen in Figure 3.89 the tunnel crosses through an alluvial deposit in the south (white in the geological map) and through an older sand dune in the north. Even though the geological map is of a 1:100,000 scale it gives a rough idea of the geological variation along the tunnel. The tunnel had also suffered cracking before the earthquake; older repaired cracks were found in the middle one third of the tunnel. These old cracks possibly indicate some change in geological structure and mechanical properties of the surrounding ground. Since more cracks were found on the west side than on the east side of the tunnel, and the western side cracks were located further south, it is likely that there is South-West North-East trending change in geology structure, which crosses the tunnel at an angle. We also observed that the west side was moister than the eastern side (see Figures 3.90 and 3.91) which seem plausible since the phreatic line is likely lower towards the river (blue line in Figure 3.83) on the east side.

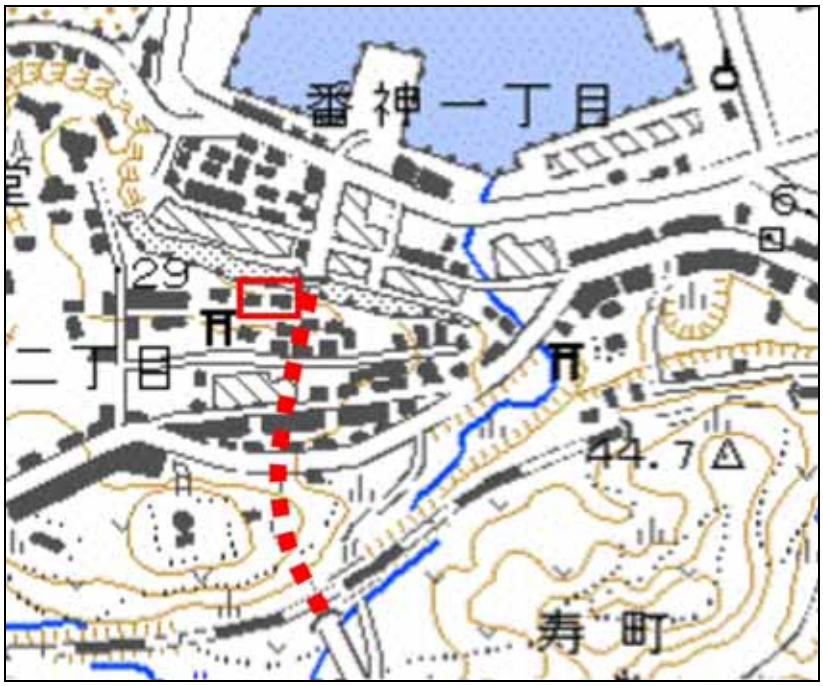

Fig. 3.83 Banjin tunnel location. Red rectangle shows location of photo Fig. 3.86

Fig. 3.84. Entrance to Banjin Tunnel by the Port of Kashiwazaki.

Fig. 3.85. Southeast Tunnel Entrance.

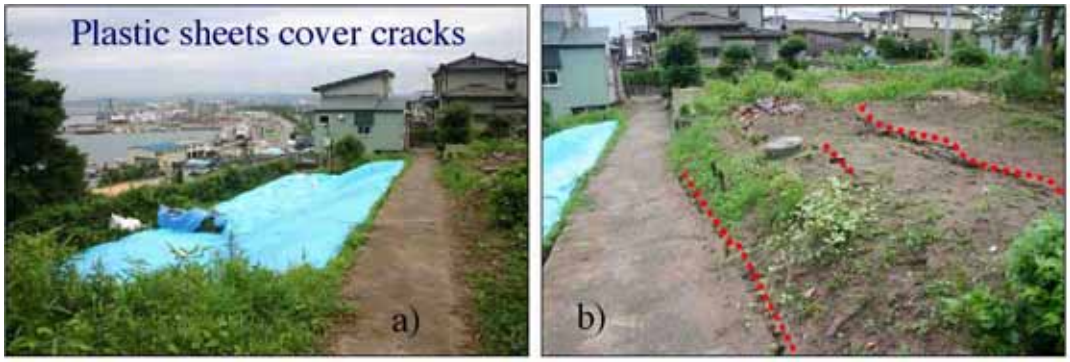

Fig.3.86. Soil movement induced ground cracks, which were covered with sheets in a) and indicated by red lines in b). Cracks beneath sheet are likely much larger than the ones seen in b). (Photo courtesy of Satoshi Murakami at Ibaraki University, Japan)

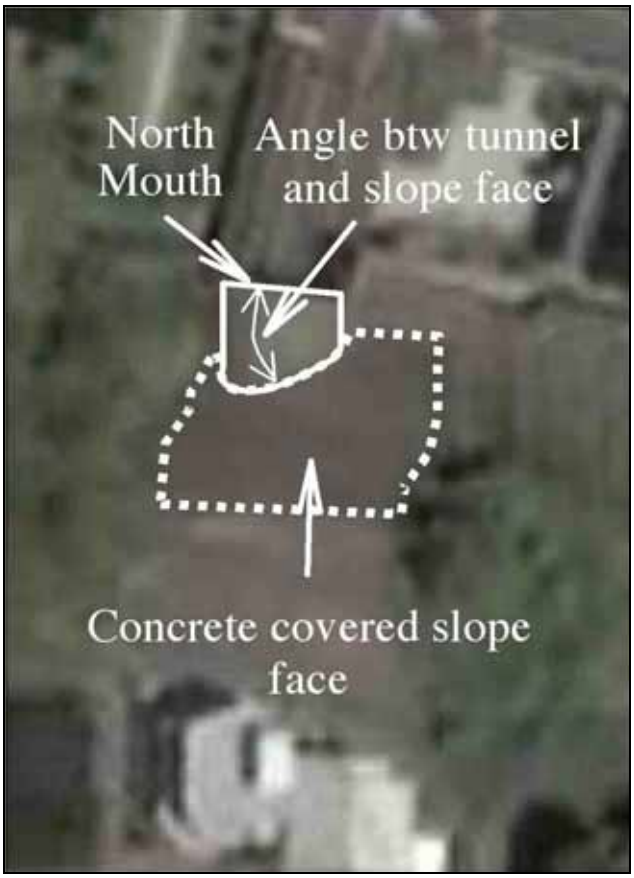

Fig. 3.87. The concrete covered slope face cuts the tunnel at an angle and this is also seen in the cracks in side the tunnel Fig. 3.88

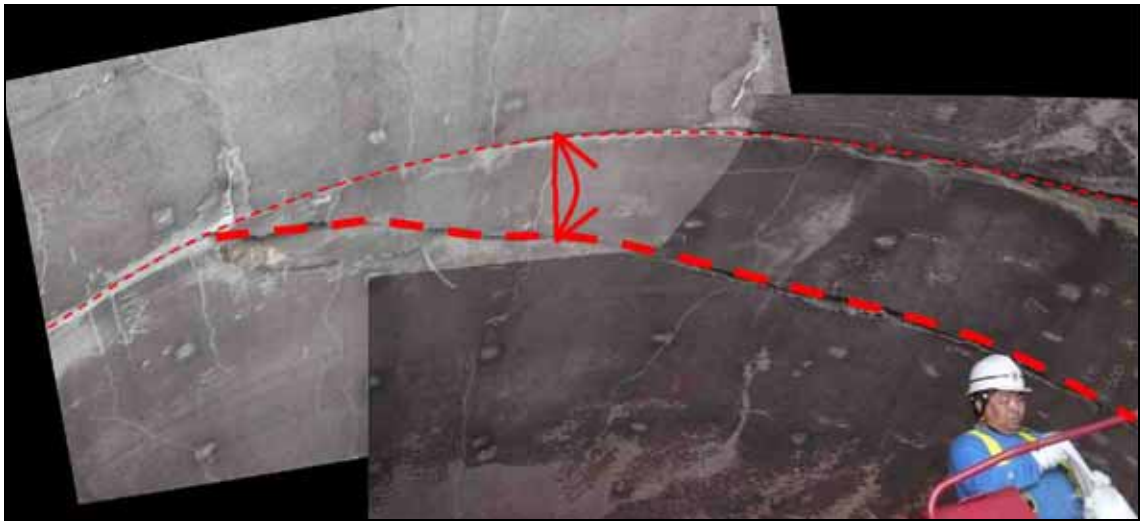

Fig. 3.88. . Photo tunnel lining crown showing segment joint as thinner dashed ling and the thicker line indicates cracks due to that the concrete covered slope face out side the tunnel crossed the tunnel at angle. Repair was ongoing as we visited the tunnel

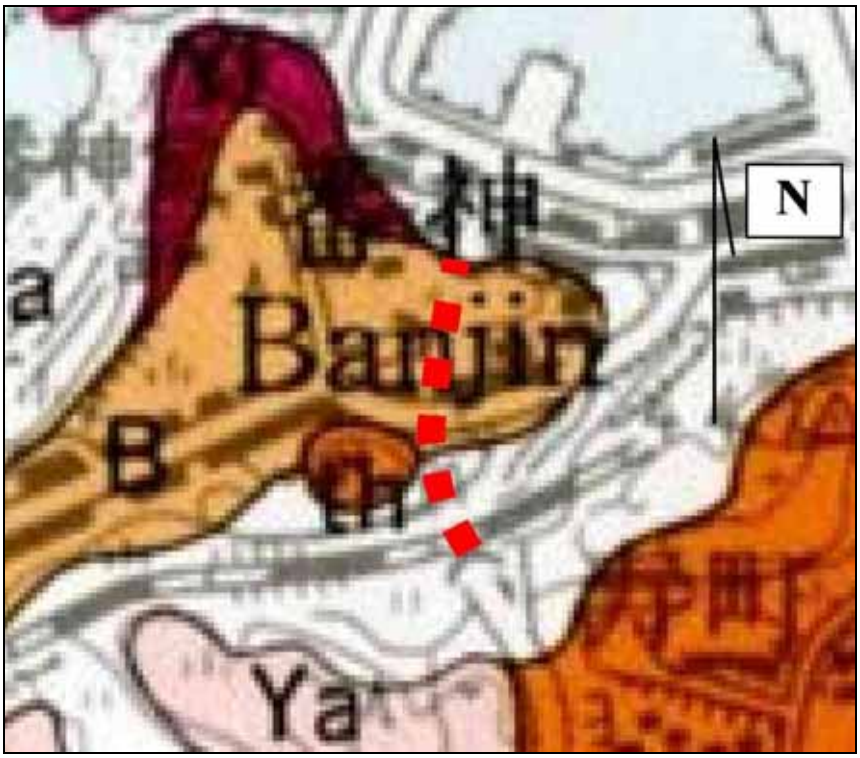

Fig. 3.88 Banjin tunnel geology. (Map by Geological Survey of Japan)

Fig. 3.90 Old moister cracks on west side of Banjin tunnel.

Fig. 3.91 Old dryer cracks on east side of Banjin tunnel

(3) Kariwa Tunnel (N37.4367° E138.6045°)

The Kariwa Tunnel goes through a 80 meter high sand dune just to the north of the nuclear power plant (Fig. 3.92). It was 530 meter long and suffered extensive cracking. Some 100 meters from northern entrance a large amount of leaking was observed (Fig. 3.93). Looking at an aerial photo of the tunnel one can see that leaf trees are more common in the area above the water leakage than in other parts along the tunnel where pine trees are more common. This was confirmed by a second independent visit to the

areas above tunnel. It is possible that the peculiar curve of the tunnel stops water behind it and forces the ground water to run along outside the tunnel, which is inclined at a grade of 1.4% towards north.

Fig. 3.92. South Entrance to Kariwa Tunnel.

Fig. 3.93. Water leaking inside the Kariwa Tunnel.

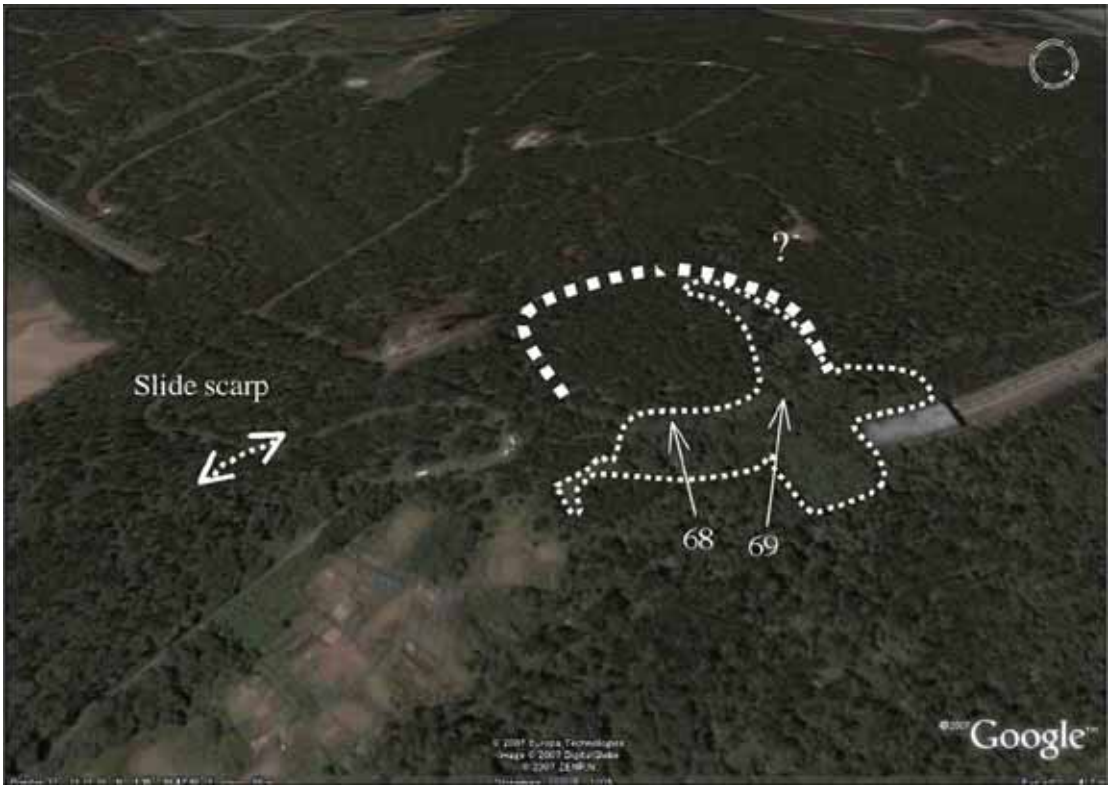

Fig. 3.94 Kariwa tunnel as seen from the North-East with Google Earth. The TEPCO nuclear power plant is located in the South-West. See further description in text.

There is a fair amount of cracks in the lining in the northern part and approximately 120m from its northern end there is a water leakage with more water on the south-west side of the tunnel, i.e. towards the mountain side.

Before the earthquake, cracks at several locations were “stapled” together, with help of two bolts at each side of a 50 cm long, 5 cm wide angle-beam (see Fig. 3.95). Furthermore, in the northern part of the tunnel, the lining’s upper half had been covered white FRP (Fiber Reinforced Plastic) sheets prior to the earthquake. At the time of our visit August 6, and 12, several of these sheets had torn and fallen off, during the earthquake and/or before.

With the help of infrared imaging we were able to detect some features hidden behind FRP sheets, invisible to the naked eye (see Fig. 3.95). It is likely the lines seen in the infrared image are cracks previously repaired with less porous material, and therefore showing up as having higher temperature. The ground water permeates much easier the surrounding concrete that appears as cooler.

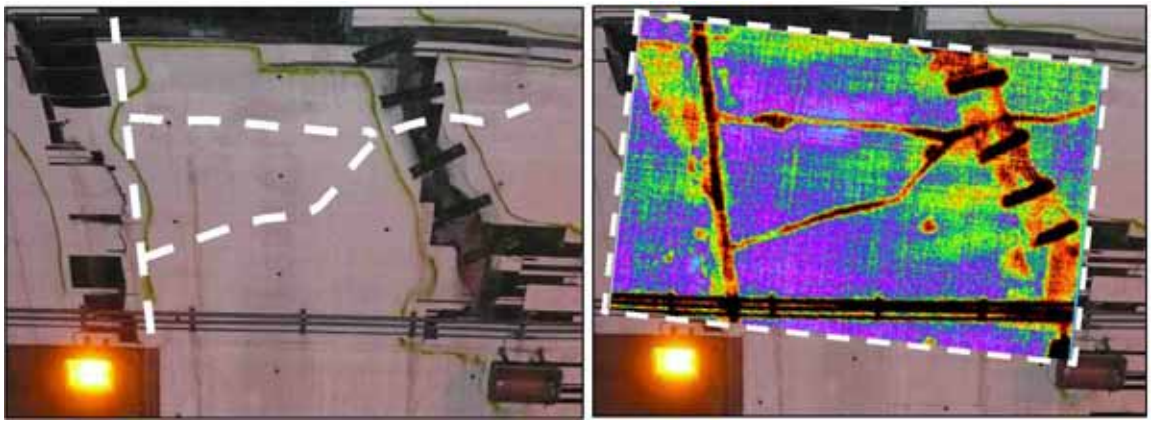

Fig. 3.95. Old cracks "stapled" together on the right side of the picture. Feature hidden behind FRP sheets detected by infrared camera.

Observation outside and on the ground above the tunnel shows that there are ground displacements and related cracks in many locations. Specifically, at the location just above the section where most leakage was observed, it seems that broadleaf plants are dominating, the brighter green area inside dotted line in Figure 3.95, over the pine trees, which are observed in many other locations along the tunnel, and all over the sand dune. In this area, at point 68 in Figure 3.94, some 15 cm downward movement and ground cracks were observed (see left photo in Fig. 3.97). Walking on the ground surface above, along the tunnel, towards the north end, smaller cracks were observed. A fence, restricting the entrance to the nuclear power plant area, runs along this section of the tunnel and due to soil movement, gaps between fence footings and soil were observed at several locations (see Fig. 3.96).

Fig. 3.96. Footing hanging in the air since soil moved down slope toward NE.

Fig. 3.97. Ground cracks approximately above water leakage location in Kariwa tunnel. Left: 15 cm downward movement of soil to the left of the person in the photo (point 68 in Fig.3.93) Right: Crack extends further up the slope beyond Nuclear power plant fence. (point 69 in Fig. 3.93)

The fence also buckled.

At the north end of the tunnel there is 60 meter long stiff retaining structure (see Figure 3.98) with two thick concrete walls along each side of the road connected with large dimension beams spanning the road plus the width of the side walks. The beam height is approximately 2 meters, and the thickness is 1.5 meters.

Fig. 3.98. North end retaining structure with rigid beams connecting right side retaining wall with left side wall.

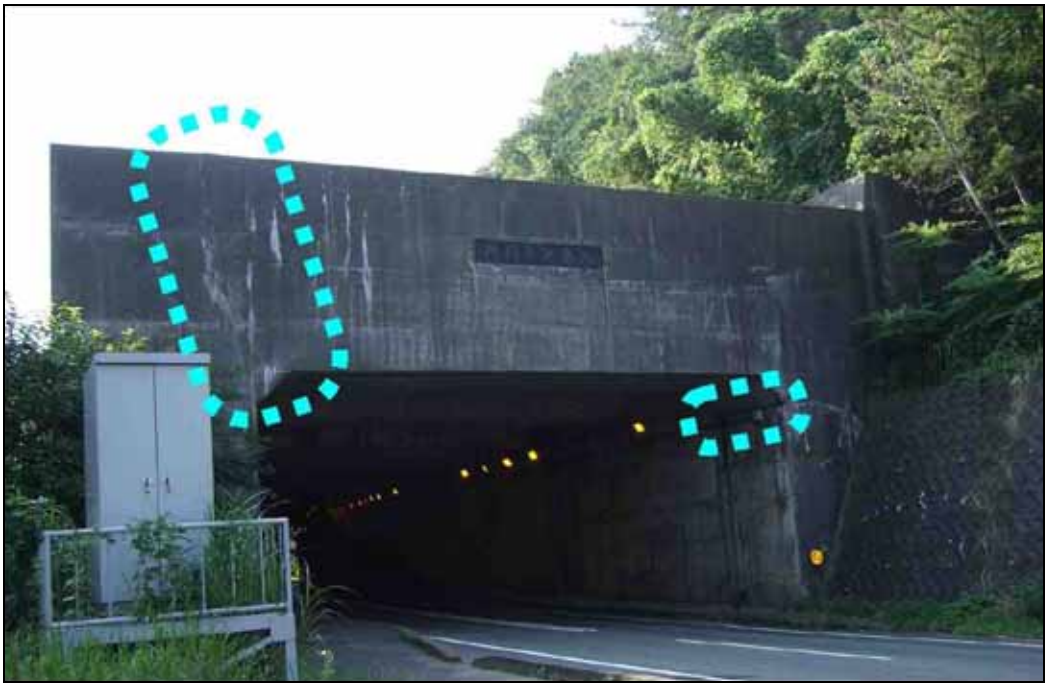

Fig. 3.99. North end retaining structure with crack on the left side and 1cm separation on the right side

Close to the southern 1/3 of the tunnel, an approximately 50 meter long slide scarp was observed with 30 to 50 cm displacement in the south part and 5 to 20 cm in the north part (see Fig. 3.100).

Fig. 3.100. Slide scarp observed at right white circle in Fig. 3.93.

A hypothesis is that ground water from the high sand dune is seeping down towards, and being dammed up behind the tunnel. We tried to confirm this also with temperature measurements on the lining close to the location of the maximum leakage, conjecturing that the ground water would cool down the south west side as compared to the north-east side. Half of a degree difference was observed, however this should be investigated further.

3.3.3. Performance of Retaining Walls

Most retaining walls perform very well during earthquakes. In fact, many highway owners, including Caltrans don't design retaining walls for earthquakes. They either assume that the design for service loads is robust enough to handle earthquakes or that retaining wall damage does not usually constitute a life-safety issue. Moreover, the available methods (usually based on Monobe and Okabe), cannot be used for PGA>0.4g. Therefore, it is of vital importance during earthquakes to document the performance of retaining walls to determine if the current practice is justified.

We saw many examples of retaining walls that performed well during this earthquake. For instance, the mechanically stabilized earth (MSE) retaining walls supporting the embankment of the Toyota Bridge in Figure 3.15 continued to support the embankment after it had settled almost a meter during the earthquake. MSE walls include a lattice of metal strips, plastic grids, or other materials embedded into the soil and attached to facing material like masonry panels.

We only saw retaining wall damage in situations where the wall was never expected to support the quantity of material that slid downhill during the earthquake. The following examples show this situation clearly.

(1) Landslide along Highway 8 (N37.333° E138.466°)

At this location the roadway had been cut into a steep slope that was supported by gabions about 50 m below the road at the toe of the slope (Figure 3.101). During the earthquake, a failure surface formed under the roadway, and the slope slid downhill several meters pushing the gabion wall over.

Fig. 3.101. Slip-out on Highway 8 (Courtesy of GEER).

The road may have been partially on cut and partially on fill, because the house and part of the roadway did not slide downhill (Figure 3.102). Note that the retaining wall (on the right side of the photos) did a good job of supporting the road during the earthquake.

Fig. 3.102 Slip-out on Highway 8 (Courtesy of GEER).

(2) Small slide on Highway 8 (N37.3168° E138.4348°)

As shown in Figures 3.103 and 3.104, this slide occurred where a short concrete retaining wall topped with fencing at the toe of the slope. This wall was not adequate and a rockslide 100 m wide and about 10 m high temporarily closed Highway 8. By the next day, the slide had been removed from the roadway but sandbags reduced the traveled way to a single lane.

Fig. 3.103. Photo of slide on Highway 8 taken on July 16, 2007. (Courtesy of MLIT)

Fig. 3.104. Photo of slide on Highway 8 taken on July 17, 2007. (Courtesy of MLIT)

(3) Near Kasashima (N37.3355° E138.4616°)

Near the village of Kasashima, a local road was built on a 10 m high seawall. Above the road is a 40 meter bluff composed of weathered sediment over rock. During the earthquake, some of the sediment slid onto the road and sea (Fig. 3.105). The seawall was damaged a little further up the coast due to lateral spreading (Fig. 3.106)

Fig. 3.105. Another view of slide showing surrounding area.

Fig. 3.106. Seawall between Kasashima and Omigawa was damaged due to lateral spreading.

(4) National Highway 352 near Nuclear Power Plant (N37.48455° E138.62435°)

This steep headland was stabilized with retaining and crib walls above and below the roadway. The sloping ground above these walls had been shotcreted and covered with a wire fabric that was nailed into the slope. During the earthquake there were many shallow slides above and below the road (Figures 3.107 and 3.108). Many of these slides had initiated above the shotcrete and dragged the wire fabric onto the roadway.

Fig. 3.107. Damage to Highway 352 from slides above and below the roadway.

Fig. 3.108. Landslides above and below Highway 352.

3.3.4 Performance of Roadways

Roadways were the most vulnerable element of the transportation system during the Kashiwazaki earthquake. Some roadways were in cut sections, which seem more vulnerable to having landslides cover them, while roadways on fill were more vulnerable to having slip-outs or of sliding downhill. The following examples show roadways damaged due to lateral spreading, landslides, and due to the failure of adjacent structures such as buildings and utility lines.

(1) Landslide on Highway 8 (N37.414° E138.713°)

In the foothills east of Kashiwazaki, a landslide pushed Highway 8 into the Kurokawa River (Figure 3.109). The scarp was 60 m above the road and the slide damaged about 80 m of roadway. This section of Highway 8 was reopened to traffic about a week after the earthquake. The ministry cleared away the slide, cut the toe of the slope deeper into the mountainside, and paved a new section of roadway (Figure 3.110). When we arrived at the site (on August 3rd), they were providing new drainage under the roadway (Figure 3.111), laying out a fabric on the slope to help retain it, and reseeding the damaged mountainside, (Figure 3.112). A waterline must have been broken by the landslide because we saw a new pipe on the bridge crossing over the Kurokawa River.

Fig. 3.109. View of Highway 8 being pushed into Kurokawa River by slide.

Fig. 3.110. Work to stabilize the soil and repave the highway (photo taken August 4, 2007).

Fig. 3.111. Old roadway is now lying along the Kurokawa River.

Fig. 3.112. Grading and restraining slope above Highway 8 (photo taken on August 4, 2007).

(2) Road to Incinerator Plant (N37.3923° E138.581°)

This area along the Sabaishi River was composed of very loose and saturated alluvium and consequently suffered a great deal of damage during the earthquake (Fig. 3.113). Ground shaking was strong enough to break the chimney of the nearby incinerator plant (Fig. 3.114). The road to the plant laterally spread toward the Sabaishi River where there were several sand boils (Fig. 3.115). Gas and water lines under the road were exposed, when the road slid into the river and had to be capped (Fig. 3.116).

Fig. 3.113. Incinerator Plant Road slid about 10 meters downward.

Fig. 3.114. Slide scarp with incinerator plant in the background.

Fig. 3.115. Sand boils along Sabaishi River.

Fig. 3.116. Capping utility lines after they were exposed by laterally spreading roadway.

(3) Slip-out of roadway on Highway 8 (N37.3565° E138.5217°)

There was a small slide of a roadway embankment that slid into an adjacent field (Figures 3.117 and 3.118). In the photos there is light pole and a power pole that rotated as the weak soil pushed against them. New fill was placed, new asphalt was laid, the old roadway was removed, and a tarp covered the existing embankment to prevent erosion.

Fig. 3.117. Photo of landslide taken on July 23, 2007.

Fig. 3.118. Photo of landslide taken on August 3, 2007.

(4) Area around Hatsusaki (N37.3220° E138.4391°)

This isolated area along the coast was devastated by landslides, which we noticed when we happened to look down from the bridges on Highway 8 we were studying. Note the many transportation facilities (as well as other lifelines) squeezed between the headlands. There are roads along the shore and up in the headlands, as well a railway, bridges, tunnels, and other lifelines (Figures 3.119 to 3.121). All of these facilities are exceptionally vulnerable to ground shaking and landslides.

Fig. 3.119. Looking Northwest at Yoneyama-Cho and landslides.

Fig. 3.120. Landslide blocked roads and turned over gravity wall (tunnel was okay).

Fig. 3.121. More landslide along the coast.

Some of the roads blocked by landslides may never be reopened We were struck by the power of these falling rocks to damage roads (Figures 3.122 to 3.124).
Communities and transportation facilities along bluffs and headlands are at higher risk from earthquakes. However, the redundancy of transportation systems in this area (streets, roads, highways, railways) made the post-earthquake recovery a little easier.

Fig. 3.122. Slides may have permanently destroyed this prefectural road.

Fig. 3.123. Roads along shore and up in the headlands were both destroyed by landslides.

Fig. 3.123. Coastal road was severely damaged by falling rocks.

(5) Damage on Prefectural Route 279 (N37.4733° E138.6213°)

Prefectural Route 279 is an east-west road, which terminates at National Highway 352 on Japan's west coast. As Route 279 approaches the Village of Shiya, it is on a slight cut in

hilly terrain that laterally spread during the earthquake. Note in Figure 3.125, the road spread to the south, and pushed against a signpost, almost tipping it over.

Fig. 3.125. Damage to Route 279 near national highway 352.

(6) National Highway 252 (N37.3544° E138.5984°)

This is an example of a road that was closed due to damage of the buildings surrounding the highway. This type of co-location damage is very common during earthquakes. In the village of Kami Tajiri a house fell onto the highway, closing a lane until it could be removed (Figure 3.126).

Fig. 3.126. Highway 252 had one lane closed due to co-location damage.

(7) National Highway 291 (N37.3418° E138.6504°)

In the mountains east of Kashiwazaki, there were some large landslides as well as smaller slip-outs like the one shown in Figure 3.127.

Fig. 3.127. Damage and repair to highway 291.

(8) Damage due to utility line failure.

There was other road damage in the area. In particular, many roads were closed because utility pipes had lifted up (due to buoyancy) or the road had settled (due to liquefaction) so that the manholes blocked the roadway (Figures 3.128 and 3.129).

Fig. 3.128 Manholes blocking roads.

Fig. 3.129. In these photos, the utility line and the manhole have damaged the road.

3.3.5 Performance of Railways

Although we didn't investigate railway damage in great detail, we did visit a few locations of damage and we also read reports that described where tracks had buckled or the embankment had settled. We couldn't find any railway bridges that were damaged. Railway bridges are tough, and are thought to suffer less damage during earthquakes. Still, we did see several railway bridges that were being worked on (Fig. 3.72 to Fig 3.73) and some of that work could have been due to the earthquake. The report below, written the day after the earthquake by the Niigata Prefecture Government, does a good job of summarizing the most serious railway damage. However, their perspective is service within Niigata Prefecture. From a national perspective, the situation was worse, since the Shin'etsu line is also used for travel between Osaka and Hokkaido. Japan Rail put out daily reports alerting the public of routes still out of service (Table 3.2).

July 17, 2007 2:00PM 18th Report - Road and Transportation Information

Transportation Policy Division, Niigata Prefecture

1. Joetsu Shinkansen (Bullet Train) Service

- Service has been resumed as normal.

2. JR Local Trains

■ JR EAST - Notification of Suspension of Services

- Shin'etsu Main Line (Saigata-Miyauchi stations)
  - Service is completely suspended.
  - A part of the railroad track has been covered with soil due to a large-scale landslide that has occurred between Kasashima and Oumikawa.
  - Restoration is likely to take a long period of time.
  - Alternative transportation service covering the same route is not available due to the condition of the area roads.
- Echigo Line (Yoshida – Kashiwazaki stations)
  - Service is completely suspended.
  - The railroad track has been damaged in some areas. (Inspection will likely take some more time to complete.)
  - Restoration work will start on July 17th2007.
  - There is no scheduled date for the resumption of service.
  - Alternative transportation service covering the same route is not available due to the condition of the area roads.
  - One train car has derailed at Kashiwazaki Station. Restoration work starts on July 17th, 2007.

※ Other railway services will be operated as usual starting from the first departure. (Delays are expected in some lines.)

■ JR WEST - Notification of Suspension of Services

- Hokuriku Main Line – Service is in operation as usual.
  For the following express and rapid trains bound for Niigata are suspended up to July 23rd due to the suspended service between Saigata and Miyauchi.
  Express: Hokuetsu, Nihonkai, Hokuriku, Twilight Express
  Rapid: Kitaguni, Noto
- Oito Line – Service is in operation as usual.

■Freight Train

- Shin'etsu Main Line
  - A freight train that had stopped at Yoneyama Tunnel #1 reportedly did not derail. (The train was headed for Sapporo from Umeda.)
  - There is no scheduled date for restoration between Yoneyama and Kasashima.

3. Hokuetsu Express

The Hokuetsu Express will operate as usual including Express Hakutaka.
Trains are delayed for 10 - 40 minutes in some areas.

Table 3-2 Japan Rail List of Suspended Train Routes on July 18, 2007

Train Route	Stations
Hokuetsu	Kanazawa to Niigata
Nihonkai	Osaka to Aomori
Twilight express	Osaka to Sapporo
Kitaguni	Osaka to Niigata
Hokuriku	Ueno(Tokyo) to Kanazawa
Noto	Ueno(Tokyo) to Kanazawa

Figure 3.130 shows the four locations where railway damage occurred, which will be discussed in this chapter. However, these are just examples of the types of damage that were more widespread during the earthquake.

Fig. 3.130. Locations of railway damage discussed in the section.

In addition to these damage locations, Figure 3.131 shows the Kashiwazaki JR station where a train car was knocked off the tracks and leaning on its side during the earthquake.

We spoke to the station manager and he said there was no damage to the station and no injuries to passengers.

Fig. 3.131. Train Car that was knocked over during earthquake.

(1) Omigawa Station (N37.34490° E138.48435°)

A large landslide at Omigawa Station damaged the tracks and other railway facilities, just south of the Yoneyama Bridge on Highway 8 that we previously examined (Fig. 3.132).

Fig. 3.132 Landslide across JR Shin'etsu tracks south of Yoneyama Bridge.

It took about a month to remove the damage and repair the tracks and station. Figure 3.133 shows how the station looked when the Geotechnical Earthquake Engineering Reconnaissance (GEER) team investigated the site on July 21st, five days after the earthquake.

Fig. 3.133 Photo of Omigawa Station taken on July 21st (Courtesy of the GEER).

When the TCLEE team arrived on August 4th the landslide had been removed and a crew was repairing the track (Figure 3.134). The station was put back into operation on August 11th. The entire JR Shin'etsu Line was put back into full operation on September 13th, 2007.

Fig. 3.134. Photo of Omigawa Station taken on August 4th.

It may seem bad luck that the biggest slide occurred just beyond the slide protection barrier as seen in Figure 3.135, but it was also good luck that it didn't take out the houses on the top of the bluff or Highway 8. Actually, these headland slides are fairly shallow, at least near the top of the headlands, since the eroded material collects at the bottom.

Fig. 3.135 Omigawa landslide caused damage below rather than at top of headland.

(2) Example of Track Buckling in Kashiwazaki (N37.3704° E138.5648°)

Tracks buckled at locations where the railway embankment moved laterally during the earthquake (Figure 3.136). During the time of the GEER investigation, rail crews were quickly repairing this minor damage. Note the combination of concrete and timber sleepers on the track and the adjacent damaged house.

Fig. 3.136. Location in Kashiwazaki where track buckled due to lateral movement (Courtesy of GEER)

(3) Arahama Station (N37.4056° E138.6020°)

Arahama is an example of another train station that was damaged during the earthquake, this time due to liquefaction. Not only did the tracks buckle, liquefaction induced settlement and lateral spreading damaged the platform, the power lines, signaling equipment, etc (Figure 3.137).

Fig. 3.137. Liquefaction induced damage at Arahama Station (Courtesy of GEER).

(4) Settlement of tracks near Do-hai (N37.3932° E138.5952°)

This location provides an example of settlement of the track due to loss of bearing capacity of the embankment due to liquefaction. The embankment was about two-meters high but settled considerably during the earthquake (Figure 3.138). The track is about 100 meters from a small lake (Figure 3.139).

Fig. 3.138. Settlement of tracks due to liquefaction of embankment (Courtesy of GEER).

Fig. 3.139. Location of settlement was about 100 meters from a small lake (Courtesy of GEER).

3.3.6 Performance of Ports, Harbor and Coastal Structures

The coastal portion of Niigata prefecture affected by the Kashiwazaki earthquake includes numerous small harbors, one moderate sized port, and extensive coastal structures for shore protection. Kashiwazaki Port, the largest facility in the meizoseismal region, includes facilities for commercial fishing and breakbulk handling, but no large scale conveyance systems or containerized cargo handing berths. The several nearby small harbors have been developed for public marinas and smaller commercial fishing operations. The port and harbor facilities were near the south end of the earthquake rupture plane, which ran beneath and parallel to an approximately 30km long stretch of the coast (Kayen et al. 2007), and the extent of the coastal infrastructure impacted by the event was widespread. The coastal location of the Kashiwazaki earthquake provided the reconnaissance team with the opportunity to assess the performance of the Kashiwazaki Port, several small harbors, and an array of coastal structures (breakwaters, jetties, seawalls) to ground motions of moderate intensity along this 30 km portion of the coast that is roughly coincident with the extent of the rupture plane.

The field investigation established regional trends in the seismic performance of waterfront structures and the occurrence of ground failures at ports, harbors, and other coastal settings. The inspection of coastal facilities was initiated from the southwest, at the small village of Kasashima, progressing northeast through the city of Kashiwazaki, and terminating near the village of Izumozaki. The sites are geospatially located via Google Earth and thumbnail photographs are provided for each location at http://walrus.wr.usgs.gov/infobank/n/nii07jp/html/n-ii-07-jp.sites.kmz. An attempt to correlate the observed seismic performance of selected structures to ground motion levels was made, however there are very few strong motion instruments located at the coastal facilities inspected. Approximate estimates were therefore made on the basis of local recordings on different soil conditions, or on established attenuation relationships adjusted for the recorded data (Kayen et al, 2007). Pertinent geotechnical, structural, and construction data for the waterfront structures was not available at the time of the reconnaissance. Efforts are currently being made to update the database however there are several sites where pertinent aspects of the waterfront development, for example the type and size of the quay walls or foundation types, have not been verified.

The southwestern limit of the coastal investigation was located at the village of Kasashima, approximately 9 km from downtown Kashiwazaki (City Hall). The harbor has been primarily developed for small fishing boats. The harbor exhibited negligible to very minor evidence of ground deformation and seismic effects to the quay walls, seawalls, and breakwaters). Very small, localized ground deformations were observed in native beach deposits and fill areas, and it appears that the ground motions in this area were just strong enough to initiate incipient lateral deformation of the ground and waterfront earth retention structures (Figure 3.139). The ground motions in this area were likely in the range of 0.10g to 0.15g, with a bracketed duration (PGA > 0.05g) of approximately 10 seconds.

This site is useful for investigating the regional extent of liquefaction-induced ground failure at ports located southwest of the meisoseismal region. Maximum cumulative

lateral spread displacements of 7 cm to 12 cm were observed in beach deposits with associated lateral deformation of short quay walls, which appeared to be concrete caissons, of 1 cm to 2 cm. No movement of larger block walls or breakwaters was observed. Settlement of paved aprons of roughly 10 cm to 12 cm was noted behind the short quay wall in this area (Figures 3.140 and 3.141).

The reconnaissance team did not inspect coastal facilities and beaches located southwest of Kasashima harbor. It is possible that minor evidence of liquefaction occurred to the southwest, however in the absence of significantly different site effects, the impact on coastal infrastructure was likely minimal. The investigation team did not receive reports of significant damage to coastal facilities southwest of Kasashima, despite the occurrence of several moderate to large landslides in cut slopes for highways and rail routes in this area.

Fig. 3.139. Minor ground failure in fill deposits along the waterfront at Kasashima (N37.33702° E138.46680°). The limited extent of the ground cracking indicated incipient lateral movement

Fig. 3.140. Kasashima Harbor – Ground settlement and minor lateral deformation adjacent to quay walls (N37.33702° E138.46680°).

Fig. 3.141. Kasashima Harbor – Settlement of fill behind quay walls (N37.33702° E138.46680°).

From Kasashima the reconnaissance progressed northeastward toward the city of Kashiwazaki. Much of the coastline in this area was inaccessible at the time of the investigation due to the closure of access roads and existence of steep cliffs leading to small beaches. The beach at Yoneyama, located adjacent to the Oumigawa Train Station impacted by the large landslide and roughly 7 km WSW from city hall, was observed during the LIDAR survey of the slide (Kayen et al, 2007). The beach is composed of cobbles and therefore not vulnerable to liquefaction. With the exception of the damage caused by the landslide of the coastal slope to railroad infrastructure, no significant damage to sea walls was observed along this beach.

Kujiranami Beach is a rather broad, sandy beach located approximately 4 km WSW from city hall. The landward portion of the beach is lined with light commercial facilities and tourist shops. No thorough investigation of these facilities was made, although based on cursory observations made from the roadway it appeared that almost all structures performed well in the area. No evidence of widespread liquefaction induced ground failures was observed along this portion of the coastline.

The Kashiwazaki Marina, located roughly 3.6 km WSW from city hall, was inspected and categorized as a site of minor damage. Evidence of liquefaction in fill soil included deposits of sand that had been vented up through pavement (Figure 3.142) and minor ground failures. Ground deformation was observed in the form of minor lateral movement (generally 2 cm to 7 cm) of short quay walls and maximum ground settlements adjacent to quay walls of roughly 7 cm to 15 cm (Figures 3.143 and 3.144). Small sand boils were observed in isolated portions of the boat storage yard. The alignment of the quay walls was generally very good indicating at most fairly uniform, minor lateral movement. Estimates of maximum lateral wall movement are 6 cm to 10 cm in this area. Minor ground cracking and lateral deformation was observed around an operations building (Figure 3.145) and along the perimeter of the marina property. Paved and unpaved parking areas located in filled areas adjacent to the facility exhibited only minor cracking with cumulative displacements on the order of 3 cm to 5 cm.

Minor ground failures were observed at the Onodachi Seaside Park adjacent to the Kashwazaki Marina. The park includes a multi-story observation structure that is located on fill. The foundation type is not currently known. The sloping, grass-covered fill at this site exhibited minor cracking indicating incipient ground failure. Little settlement was observed.

Fig. 3.142. Kashiwazaki Marina – Evidence of minor liquefaction on the boat storage yard (N37.36093° E138.52027°).

Fig. 3.143. Kashiwazaki Marina – Evidence of minor lateral movement of quay walls and associated settlement of the paved apron (N37.36093° E138.52027°).

Fig. 3.144. Kashiwazaki Marina – Evidence of lateral movement and settlement behind quay walls (N37.36093° E138.52027°).

Fig. 3.145. Kashiwazaki Marina – Minor settlement adjacent to a waterfront operations building. No structural damage was observed however repairs to the appurtenant utilities were being made during the site visit (N37.36093° E138.52027°).

The primary focus of the coastal reconnaissance was the Port of Kashiwazaki, the largest marine complex in the region. The Port of Kashiwazaki is relatively large in lateral extent covering approximately 1.25 km (2.2 km of waterfront berths). For the sake of this investigation the port has been subdivided into the following sections (Figure 3.146);

1. Western Inner and Outer Harbor.
2. Fishing and Light Breakbulk Handling Facilities.
3. Steel Scrap Handling Facility.
4. Central Quay.
5. Eastern Berth.

The reconnaissance team is unaware of strong motion data obtained within the port. On the basis of data from local strong motion stations located on alluvial soils (Soil Class D) the peak horizontal accelerations in the vicinity of the port may have exceeded 0.3g. The bracketed durations (PGA > 0.1g) of the motions in the region were generally between 5 and 10 seconds.

Fig. 3.146. The Port of Kashiwazaki (modified from Google Earth, 2007).

Western Inner and Outer Harbor

The westernmost portion of the port includes; (a) an older portion of the berth that appears to be relatively unused, (b) an asphalt heating and pumping facility, (c) a new facility that includes sightseeing boat mooring and a single-story office building, (d) the outer quay wall, and (e) the major seawall and breakwater that extends from the beach into the harbor a total length of approximately 2.4 km. The landward portion of the berth appeared to include both steel sheetpile sections and a wharf deck supported by vertical concrete piles (Figure 3.147). No lateral movement of this portion of the wharf was noted or damage to the piles was noted, although only the waterfront row of piles was visible during the investigation. Immediately adjacent to the pile-supported wharf is an asphalt storage and transport facility. The Take Naka asphalt facility (b) exhibited very little damage (Figures 3.148). The facility was in operation at the time of the reconnaissance and the inspection focused on the performance of the large, insulted asphalt holding tank. The tank is secured at its base by a bolt and flange detail. Many of the flanges were bent however none of the bolts appeared to have failed (Figures 3.149 and 3.150). No "elephant's foot" bulking of the tank was noted, although the exterior of the tank is covered with a layer of insulation prohibiting a more thorough inspection. The elevation of the asphalt in the tank at the time of the earthquake was not determined. The asphalt distribution pipe, which is slightly elevated above the ground, did not exhibit signs of distress despite extensive ground failure along the alignment adjacent to the seawall (Figure 3.151) or areas near the sightseeing facility, which is addressed next.

Fig. 3.147. Kashiwazaki Port – Western Inner Harbor. Older, undamaged pile-supported wharf located in the western portion of the port facility (N37.36713° E138.53210°). The asphalt storage tank is visible at the right.

Fig. 3.148. Kashiwazaki Port – Western Inner Harbor. Asphalt storage and transport facility located in the western portion of the port (N37.36713° E138.53210°).

Fig. 3.149. Kashiwazaki Port – Western Inner Harbor. Damaged anchorage of the asphalt storage tank (N37.36713° E138.53210°).

Fig. 3.150. Kashiwazaki Port – Western Inner Harbor. Anchorage system for the asphalt storage tank (N37.36713° E138.53210°).

Fig. 3.151. Kashiwazaki Port – Western Inner Harbor. Seawall and exposed section of the elevated asphalt pipe (N37.36713° E138.53210°). The pavement cracking is due to liquefaction and lateral deformation.

Within the Western Inner Harbor area, a new coastal sightseeing facility was completed shortly before the earthquake. The new development included a single-story structure with observation decks on the roof, associated pedestrian areas, and a mooring area for the cruise boats. This area was subjected to the effects of widespread liquefaction, lateral spreading, and settlement. On the basis of the exterior inspection it appeared that the structure performed very well, but did exhibit minor distress along two pedestrian-accessible roof areas on either side of the main structure. The distress is likely due to adjacent lateral spreading and differential settlement between the ground and structure.

The paved areas and walkways surrounding the building were extensively damaged (Figures 3.152 and 3.153). Settlements exceeded 30 cm in many areas and a pronounced graben formed between the building and the waterfront quay wall. The quay wall also moved laterally and tilted a minor amount (Figure 3.154). Structural plans and details of the quay wall were not available at the time of the survey. This area was extensively investigated by the team, including collection of high resolution ground-based LiDAR topographic data of the ground and structures (Figure 3.155).

Fig. 3.152. Kashiwazaki Port – Coastal Tour Facility. Lateral ground deformation due to liquefaction and quay wall movement (N37.36713° E138.53210°).

Fig. 3.153. Kashiwazaki Port – Coastal Tour Facility. Sand boils showing evidence of liquefaction behind the Tour Facility building. (N37.36713° E138.53210°).

Fig. 3.154. Kashiwazaki Port – Coastal Tour Facility. Liquefaction–induced damage to the wharf with minor displacement of the quay wall (N37.36713° E138.53210°).

Fig. 3.155. Overview of the Kashiwazaki Port – Coastal Tour Facility and Western Inner Harbor during terrestrial lidar survey data collection.

The western outer harbor area was subjected to large lateral ground deformation and settlement due to liquefaction (Figures 3.156 and 3.157). The seawall/breakwater was inspected landward from the beach adjacent to the asphalt handling facility to the outer quay area. The breakwater exhibited negligible to minor displacement adjacent to the beach. It also appeared to have performed well along the outer harbor section of the port near the coastal tour facility despite the occurrence of liquefaction in adjacent fill (Figure 3.158). The waterfront side of the seawall is armored with concrete tetrapods. The tetrapods appeared to be unreinforced and numerous units showed damage due to rocking during the earthquake (Figure 3.159). This type of damage to tetrapods and other wave armor units was noted throughout the Kashiwazaki region.

Fig. 3.156. Kashiwazaki Port – Western Outer Harbor. Evidence of lateral ground deformation adjacent to the outer quay wall Inset photo shows close-up of the magnitude of deformations in dashed area (N37.36713° E138.53210°).

Fig. 3.157. Kashiwazaki Port – Western Outer Harbor. Lateral spreading in the inner port area. 0.7 m of displacement was measured over the 79 meter long track-line (yellow tape in photo) for a total lateral strain of 0.9% (N37.36713° E138.53210°). View looking southwest towards the coastal tour facility and asphalt tank.

Fig. 3.158. Kashiwazaki Port – Western Outer Harbor. Evidence of ground failure adjacent to the seawall at the outer quay wall area (N37.36713° E138.53210°). View looking southwest towards the coastal tour facility and asphalt tank.

Fig. 3.159. Kashiwazaki Port – Western Outer Harbor. The damage to tetrapod wave armor units appeared to have occurred during the earthquake (N37.36713° E138.53210°).

Fisherman's Wharves and Light Breakbulk Handling Facilities

Along the inner harbor immediately to the east of the previously described facilities is a series of shallow water wharves that are being used primarily for small fishing operations and various light, breakbulk handling. The quaywalls in this portion of the port appear to include both steel sheetpile walls and concrete caissons. No structural plans or as-built drawings were available to the team at the time of the investigation.

At the time of the field investigation, the terminal that extends into the harbor (Figure 3.160) was being used along one berth as a staging area for a dredging barge and on the opposite side of the terminal as a fishing and/or light cargo handling area. This berth exhibited extensive liquefaction-induced damage. Lateral movement of the quay wall (anchored sheetpile wall) along the western face was approximately one meter with settlements of roughly 0.5 m to 0.6 m (Figures 3.160 and 3.161). The quay wall tilted roughly 5°. The lateral deformations along the north and eastern faces were less pronounced (roughly 0.3 m), although the settlement of the backfill was equally severe (0.6 m) (Figures 3.162 and 3.163). It appears that the quay wall along the eastern portion of the terminal had been constructed using concrete caissons. It is believed that a portion of the difference in the observed lateral quay wall movement is due to the use of different structures during construction of the terminal. The heights of the sheetpile walls and caissons above the mudline were not discerned during the reconnaissance.

Fig. 3.160. Kashiwazaki Port – Fishing and Light Breakbulk Facilities. West facing portion of the berth showing displacement of the sheet pile wall and ground settlement due to liquefaction of sandy backfill (N37.36532° E138.53213°).

Fig. 3.161. Kashiwazaki Port – Fishing and Light Breakbulk Facilities. West facing portion of the berth showing displacement of the sheetpile wall and ground settlement due to liquefaction of sandy backfill (N37.36532° E138.53213°).

Fig. 3.162. Kashiwazaki Port – Fishing and Light Breakbulk Facilities. North facing portion of the berth showing lateral displacement of the quay wall and ground settlement due to liquefaction of sandy backfill (N37.36532° E138.53213°).

Fig. 3.163. Kashiwazaki Port – Fishing and Light Breakbulk Facilities. Pronounced ground settlement adjacent to the quay wall (N37.36532° E138.53213°).

The waterfront areas along the fishing facilities exhibited minor ground and quay wall deformations (Figures 3.164 and 3.165). All wharf side buildings were open for operation, however no visual inspection of the building interiors was made by the team. The condition of the structures and their foundations was not determined at the time of the investigation.

Fig. 3.164. Kashiwazaki Port – Fishing and Light Breakbulk Facilities. Minor settlement of paved aprons adjacent to the quay wall (N37.36532° E138.53213°). Photo by Y. Tanaka.

Fig. 3.165. Kashiwazaki Port – Fishing and Light Breakbulk Facilities. Minor settlement of paved aprons adjacent to the quay wall (N37.36532° E138.53213°). (Photo by Y. Tanaka).

Steel Scrap Handling Facility

Moving eastward along the port complex the next major industrial facility is a steel scrap handling and transport wharf. Sand boils, ground cracks and liquefaction-induced settlement were pervasive throughout this facility, particularly around the waterfront perimeter (Figure 3.166). Waterfront settlements up to approximately 35 cm were noted and lateral sheetpile wall movements varied between 2 cm and 30 cm depending on the orientation of the quay (Figures 3.167 and 3.168). Pronounced pavement disruption and ground cracks in the landward section of the facility indicated lateral deformation of the anchor system for the sheetpile walls. Lateral movement and rotation of near-shore concrete blocks with bollards, and damage to wave armor units were also observed. Sporadic evidence of liquefaction was observed along the eastern extent of the berth where very minor to negligible quay wall movement was observed (Figure 3.169).

Fig. 3.166. Kashiwazaki Port – Steel Scrap Transport Facility. Pervasive settlement in the paved backland areas was observed (N37.36668° E138.53565°).

Fig. 3.167. Kashiwazaki Port – Steel Scrap Transport Facility. Northern portion of the berth showing evidence of ground failure due to liquefaction of sand fill and damage to concrete aprons (N37.36668° E138.53565°).

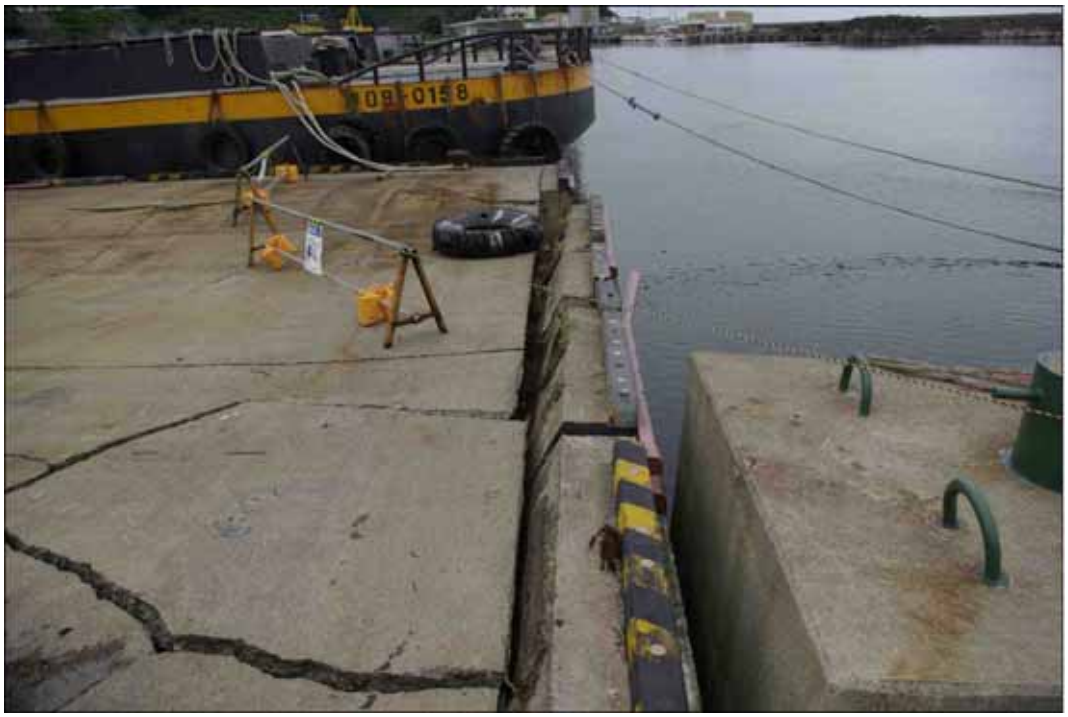

Fig. 3.168. Kashiwazaki Port – Steel Scrap Transport Facility. Northern portion of the berth showing quay wall displacements, and settlement and cracking of pavement (N37.36668° E138.53565°).

Fig. 3.169. Kashiwazaki Port – Steel Scrap Transport Facility. Eastern portion of the berth showing evidence of liquefaction and yet small quay wall displacement (N37.36668° E138.53565°).

Central Quay

This short length of quay wall is located between the Steel Scrap Handling Facility and the terminal that was being used at the time of the reconnaissance by the Japan Coast Guard, as described in the following subsection. The water is relatively shallow in this portion of the port although the exact depth of water and height of quay wall was not determined during the investigation. The quay wall was laterally displaced roughly 30 cm to 45 cm (Figure 3.170).

Fig. 3.170. Kashiwazaki Port – Central quay. North facing, landward quay wall exhibiting lateral movement and rotation of the quay wall (N37.36593° E138.53692°).

Eastern Berth (Japan Coast Guard/Japan Self Defense Forces Staging Area)

The largest terminal area at the Port of Kashiwazaki was being used by the Japanese Coast Guard and Japanese Self Defense Forces at the time of the investigation. The bayward, north facing berth of this terminal is approximately 400 m in length. The terminal served as a staging area for military assistance personnel and stockpile area for construction materials being used in the recovery and rebuilding efforts. A large pile of steel scrap from operations conducted prior to the earthquake was located immediately adjacent to the western quay wall. This western potion of the terminal was being utilized by the Coast Guard. Only very minor liquefaction effects, lateral displacement, and settlement were observed in this area. Small sand boils were noted and lateral displacements of the quay wall were very small (≤ 5 cm) (Figures 3.171 and 3.172). Apron settlements of roughly 2 cm to 7 cm were observed. Maximum settlements adjacent to bollards were roughly 7 cm. As in other areas of the port, no structural data has been obtained for the quay walls. The quays walls appeared to be sheet pile sections as evinced by waterfront geometry and the deformation patterns in pavement adjacent to the wall and possible anchor systems. Obtaining design drawings and plans for the waterfront structures will be very useful for comparing the observed performance with that anticipated for these structures under the seismic loading that occurred in the region. This task remains a high priority.

Fig. 3.171. Kashiwazaki Port – Eastern Berth. West facing quay wall exhibiting minor lateral movement and rotation of the quay wall (N37.36680° E138.53825°).

Fig. 3.172. Kashiwazaki Port – Eastern Berth. Minor ground cracks located parallel to the western quay wall, possibly associated with lateral deformation of an anchor wall (N37.36680° E138.53825°).

The west end of the north facing quay was also utilized by the Japan Coast Guard. This portion of the quay exhibited negligible lateral displacement (≤ 4 cm) and minor settlement (Figures 3.173 and 3.174). Overall, its seismic performance of this portion of the terminal was very good. The middle to eastern end of the quay was being used to berth a ship of the Japan Self Defense Forces at the time of the survey. The effects of liquefaction were similarly minor with localized apron cracking and minor lateral displacement along much of the berth (Figure 3.175).

The magnitude and extent of liquefaction-related ground failure and quay wall displacement increased along the berth to the east (i.e. closer to the eastern boundary of the berth near the mouth of the U River) (Figure 3.176). Within roughly 100 m of the U River the effects of liquefaction became pronounced. Significant damage was observed to the quay wall with lateral displacement and failure of the upper portion of the wall (Figures 3.177 and 3.178).

Fig. 3.173. Kashiwazaki Port – Eastern Berth. Evidence of very minor lateral movement of quay walls and minor settlement located along the northern face of the terminal (N37.36680° E138.53825°).

Fig. 3.174. Kashiwazaki Port – Eastern Berth. Evidence of minor settlement adjacent to a pile-supported (?) bollard anchorage located along the northern face of the terminal (N37.36680° E138.53825°).

Fig. 3.175. Kashiwazaki Port – Eastern Berth. Evidence of very minor lateral movement of quay walls and minor settlement located along the northern face of the terminal (approximately N37.36680° E138.53825°).

Fig. 3.176. Kashiwazaki Port – Eastern Berth. Evidence of more extensive lateral quay wall displacement and ground settlement located along the eastern end of the terminal (approximately N37.36680° E138.53825°).

Fig. 3.177. Kashiwazaki Port – Eastern Berth. Significant liquefaction-related damage to the quay walls located at the eastern edge of the terminal adjacent to the mouth of the U River (approximately N37.36680° E138.53825°).

Fig. 3.178. Kashiwazaki Port – Eastern Berth. Significant liquefaction-related damage to the quay walls located adjacent to the U River (approximately N37.36680° E138.53825°).

Taiheiyo Cement Tanks – This facility is located in the backland portion of the Eastern Berth terminal. The issue of business interruption at the port was considered by the reconnaissance team. In light of the regional need for concrete associated with restoration efforts following the earthquake it seems that the operation of this facility would be important. Inspection of the tank foundations revealed minor ground settlement of 5 cm to 12 cm adjacent to the tank foundations (Figures 3.179 and 3.180). Liquefaction effects were negligible in this area. The tanks and appurtenant piping appeared to be in good shape with negligible damage, however the facility was not in operation at the time of the investigation.

Fig. 3.179. Kashiwazaki Port (Taiheiyo Cement Facility) – Evidence of minor settlement of sandy fill adjacent to one of the large diameter tanks (approximately N37.36665° E138.53983°).

Fig. 3.180. Kashiwazaki Port (Taiheiyo Cement Facility) – Evidence of minor settlement of sandy fill adjacent to one of the large diameter tanks (approximately N37.36665° E138.53983°).

The eastern boundary of the Port of Kashiwazaki is located along the mouth of the U River. None of the breakwater structures located offshore of the port complex (west bank), or the levee and breakwater structures along the east bank near the river mouth were investigated during this reconnaissance. The Minatomachi Seaside Park, bounded to the west by the U River, was entirely occupied by Self Defense Forces and being used for temporary housing and staging areas for relief efforts. The area is generally open parkland and a very cursory walk-through inspection was made. Evidence of liquefaction was prevalent throughout the park. Ground cracks, sand boils, and damages to paved surfaces were widespread. Numerous sand boils and extensive ground cracks associated with lateral spreading were observed on the beach. No structural inspections were performed to assess the performance of building foundations in this area.

The oceanfront from the Minatomachi Park to the mouth of the Sabaishi River, located roughly 2.5 km to the northeast, consists of two public beaches followed by a long reach of wave armor units backed by concrete seawalls. A long portion of the seawall extending southwest from the Sabaishi River was inspected by the reconnaissance team.

The mouth of the Sabaishi River is protected along its southwest bank by a wall constructed of sheet piles with concrete filling (Figure 3.181). Negligible lateral movement of the riverfront wall was noted and roughly 5 cm to 7 cm of settlement was noted adjacent to the sheet pile wall. Evidence of approximately 30 cm of lateral spreading in the sand deposits behind the sheet pile shore protection was also observed. In addition, minor lateral deformation and damage to the concrete slope armor panels was observed across the river from this site. The extent of lateral spreading at this location was considerably less than what would be anticipated for a waterfront site where shaking

in the underlying firm soils was likely in excess of 0.30g, indicating that the sheetpile and concrete wall restrained the adjacent soil mass from moving.

Fig. 3.181. Mouth of the Sabaishi River showing the channelization and erosion control measures (approximately N37.38868° E138.56508°). The Ansei Bridge is visible in the background.

Along the waterfront to the southwest of the Sabaishi River and transverse to the riverfront sheet pile wall in Figure 3.182 is a seawall that extends southwest roughly 2.0 km towards the Minatomachi Seaside Park. This seawall was inspected for over a 1 km length. Observations were made from the mouth of the Sabaishi River southwest to an armored rockfill breakwater located roughly 1.2 km to the southwest. The initial portion of the seawall is located along a narrow beach with wave armor units. This portion of the seawall exhibited lateral deformation, rotation, and structural damage along the counterfort ends of several of the individual wall sections (Figure 3.183).

Fig. 3.182. Eastern end of the seawall located near the mouth of the Sabaishi River (approximately N37.38868° E138.56413°). Lateral spreading behind the wave armor units, as well as ground settlement and minor sea wall damage was widespread.

Fig. 3.183. Lateral displacement of the seawall located near the mouth of the Sabaishi River (approximately N37.38868° E138.56413°).

Approximately 120m to the southwest there is a seaward step in the seawall. This point was the location of a seawall failure that included a collapsed section of wall, significant settlement and cracking of the backfill, and settlement of the concrete panels located behind the seawall (Figure 3.184). The backfill adjacent to the seawall is composed of clean, rounded cobbles (diameter > 15 cm) (Figure 7.185). The lateral wall movement in this area was roughly 0.6 m.

The use of cobble backfill was observed over much of the 1km length of the seawall that was inspected and this fill was roughly 3m to 4m wide in most areas. The fill placed adjacent to the cobbles is predominantly sand with areas of sandy clay and clayey sand. For most of the reach between the Sabaishi River and Minatomachi Park a large embankment fill has been placed. The placement of the long embankment imposes static shear stresses in the foundation adjacent to the seawall. It is therefore likely that the additional seismic loading and associated seismic shear stresses were sufficient to induce widespread failures in the embankment and foundation thereby affecting the seawall (Figure 3.186).

Fig. 3.184. Partial collapse of a portion of the seawall (approximately N37.38868° E138.56413°).

Fig. 3.185. Close-up of the collapsed portion of the seawall showing the clean cobble backfill (N37.38868° E138.56413°).

Fig. 3.186. Extensive ground failures in proximity to the seawall located near the mouth of the Sabaishi River (approximately N37.38868° E138.56413°).

As previously noted the area located behind the seawall has been developed with a benched, earth embankment composed of sandy clay and clayey sand that extends almost the entire distance to the beaches near the Minatomachi Seaside Park. The seawall exhibited an increased seaward tilt as the investigation moved to the southwest (Figure 3.187). The cumulative lateral deformation was crudely estimated to be approximately 1.5 m to 2.0 m. The actual lateral movement was very difficult to estimate in the field due to the cumulative impacts of lateral spreading, levee slumping, and foundation failure. The embankment exhibited pervasive longitudinal cracks along the crest and side slopes, and a less well-developed set of transverse cracks. The longitudinal cracks were noted on both sides of the embankment however the deformations in the seaward direction were much more pronounced (Figure 3.188). The embankment is discontinuous and is bisected by several unpaved access roads. At these sections it is clear that the embankment experienced significant differential settlement, transverse cracking, and longitudinal cracking extending through the entire earth structure (Figure 3.189). The radial pattern of longitudinal cracks observed at the base of one of the embankment sections clearly demonstrated the extent of foundation failure and differential settlement of the embankment. No liquefaction features (e.g., sand boils, vented sand) were observed in the embankment fill or in tension cracks. Several large voids (> 1 m^3) were exposed near the toes of failed portions of the embankment.

Fig. 3.187. Evidence of seawall rotation, ground settlement, and lateral spreading at the toe of the benched embankment.

Fig. 3.188. Representative ground cracking and lateral displacement of the clayey earth embankment adjacent to the seawall.

Fig. 3.189. Extensive ground deformations adjacent to the embankment and seawall.

The lateral ground deformation associated with these embankment and foundation failures impacted the seawall and contributed to the development of larger displacements (Figure 3.190). At one location near the southwestern end of this area ground settlement and lateral movement of the seawall was accompanied by the failure of the top of the wall

(Figure 3.191). The inspection of the seawall was terminated at the location of an armored groin structure (possibly an outfall structure). At the time of the investigation a track-hoe was being utilized to move rockfill between the seawall and the concrete armor blocks that overlie the rockfill offshore (Figure 3.192).

Fig. 3.190. Ground deformations adjacent to the embankment and seawall.

Fig. 3.191. Lateral displacement, rotation, and structural damage to the seawall.

Fig. 3.192. Repair of an armored, rockfill breakwater (possibly an outfall structure).

No oceanfront inspection was made by the reconnaissance team along a roughly 7.5 km portion of the coast extending northeast from the mouth of the Sabaishi River. This area includes the Kariwa nuclear power plant, which was not accessible to the team. The next coastal observations were made at site located roughly 1.8 to 2.0 km northeast of the nuclear power plant facility. Minor displacement of the seawall and landward embankments, very likely due to liquefaction of beach sands at the toe of the walls, was observed (Figure 3.193). Displacements of 21 cm vertical, and 55 cm horizontal were measured between displaced seawall sections and adjacent wall sections that performed well. The total length of the failed seawall was 40 m (Figure 3.194).

Approximately 3.5 km northeast (direct line) of this site liquefaction of beach sand led to lateral spreading of the road embankment and failure (rotation and translation) of the concrete seawall. Deformations were on the order of 25 cm vertical, and up to 64 cm horizontal, with approximately 9° of rotation (Figures 3.195 and 3.196). As with the previous site, lateral spreading led to significant damage to the adjacent road surface rendering parts of the road unusable. This site marked the northeastern-most extent of the coastal investigation.

Fig. 3.193. Evidence of minor liquefaction and lateral spreading in sandy beach deposits adjacent to the seawall (N37.45478°, E138.60927°).

Fig. 3.194. Rotation and lateral displacement of the concrete seawall (N37.45478°, E138.60927°).

Fig. 3.195. Liquefaction-induced damage to the seawall and adjacent roadway (N37.48497°, E138.62645°).

Fig. 3.196. Lateral spreading associated with liquefaction of beach sand (N37.48497°, E138.62645°).

3.4. Major Observations and Recommendations

Almost all of the damage to transportation systems and facilities was a result of problem soils. Recorded ground shaking was fairly high and caused landslides, liquefaction, and lateral spreading, but very little damage to highway and railway structures. The 2004 Earthquake near Nagaoka also caused shaking around Kashiwazaki city, but without causing much damage.

The July 16, 2007 earthquake caused about $100 million US dollars in damage to the transportation systems. However, Japan's robust construction industry was able to make most repairs within a few hours of the earthquake. There is only one section to take one week for reopen on Route 8 (the slope failure at the road to Nagaoka). Other locations were completed within one day. On the expressway, it took four hours for emergency vehicle access and 56 hours to reopen it to the public. The longest closures to roads was due to large landslides.

Because of the considerably expensive soil remediation techniques, it is not clear whether this is a cost-effective technique along highways and bridges with problem soils. It requires the remediation of a large area, it's expensive, and is not always effective. Perhaps a better idea based on boring data, is to strengthen the foundation structurally. Although most owners of railways and highways are concerned to prevent loss of life on their facilities for large earthquakes, it may be cost-efficient to spend some money to improve functionality following earthquakes, especially in highly seismic regions like Niigata Prefecture.

Still, transportation structures, in general performed well for this earthquake, which may be close to the maximum earthquake at this site. There were no casualties or collapses and repairs were made very quickly.

US engineers, especially those using HAZUS as a tool to estimate bridge damage should study this report carefully. We routinely saw lateral spreading greater than 50 cm with almost no bridge damage, while HAZUS would typify this as major bridge damage. Also, engineers will sometimes propose spending millions of dollars to retrofit bridges for lateral spreading that is usually not justified, at least from a life-safety perspective. The damage to the tunnels discussed in the chapter should be more thoroughly studied. Bored tunnels are not considered to be vulnerable to ground shaking during earthquakes. Why these tunnels were damaged needs to be studied to prevent catastrophic failures during future earthquakes.

It is the authors' hope that the examples shown in this chapter will be carefully studied by researchers, and used to help bridge owners make better decisions about liquefaction hazards.

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