Damage Investigation of Matsurube Bridge during the 2008 Iwate-Miyagi-Nairiku Earthquake

by

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ABSTRACT

The Matsurube Bridge on National Highway Route #342 was collapsed during the 2008 Iwate-Miyagi-Nairiku Earthquake. The slope, which supported the abutment and pier foundations, displaced about 10m to the bridge center because of the landslide caused by the strong ground motion. The bridge was moved with the landslide, and one pier completely failed and overturned, and finally the bridge superstructure fell down to the ground. This paper presents the results of damage investigation of the Matsurube Bridge, and the damage mechanism and the lessons learned from this particular type of damage is discussed.

KEYWORDS: Collapse, Iwate-Miyagi-Nariku Earthquake, Landslide, Matsurube Bridge

1. INTRODUCTION

The Iwate-Miyagi-Nairiku Earthquake with magnitude of 7.2 occurred on June 14, 2008 at the border between Iwate and Miyagi prefectures, Tohoku region. There were serious damage including about 500 casualties and damage to about 2,500 houses. During the earthquake, large-scale landslides occurred in mountain areas and caused river closures which resulted in the development of natural dams, and road closures. Although bridge structures in the area were not affected so much by the strong ground motion, the Matsurube Bridge on national highway route #342 was collapsed during the earthquake. The bridge was managed by the Iwate prefectural government. MLIT dispatched the emergency investigation team including the experts of bridge structures to the site of the Matsurube Bridge just after the earthquake in order to technically support the Iwate prefectural government. The team reported their investigation result to the Iwate prefectural government and the media for the early recovery of the national highway route #342.

The superstructure of the Matsurube Bridge was a 3-span continuous steel girder with total length of 94.9m. The superstructure was supported by two piers in the middle spans and two abutments at both ends. The strong earthquake shaking caused landslide at the slope where the abutment and pier were located, and the abutment and pier moved by about 10m to the opposite side. One pier was completely failed and overturned and finally the superstructure fell down to the ground as a jackknife. This particular type of damage has not been found in the past earthquakes.

This paper presents the results of damage investigation of the collapsed Matsurube Bridge, and the damage mechanism and the lessons learned from this particular type of damage is discussed.

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2. DAMAEG INVESTIGATION OF MATSURUBE BRIDGE

2.1 Iwate-Miyagi-Nairiku Earthquake [1]

The Iwate-Miyagi Nairiku Earthquake occurred at around 8:43AM on June 16, 2008. The magnitude of the earthquake was 7.2 and the depth of epicenter was 8km. Japan Meteorological Intensity (JMA Intensity) was the 6 Upper in the epicentral area. The high intensities were observed in the wide area in Iwate and Miyagi prefectures.

Damage extended to the 5 prefectures in the Tohoku region, in particular heavy damage was reported in the Iwate and Miyagi prefectures. Casualties of 23 (dead: 13, missing: 10), injured: 451, damage to houses: 2557, were reported. The total damage was evaluated as 152 billion Yen. Since the earthquake occurred in the mountain area, large landslides and slope failures occurred and caused river closures and road closures.

The National Research Institute for Earth Science and Disaster Prevention (NIED) reported the strong motion observation data. The strongest ground motion was observed at the IWTH25 station which was located by just a distance of 1.3 km from the collapsed bridge site. The maximum PGA was 1,143gal in NS, 1,433gal in EW, 3,866gal in UD, and 4,022gal as a combined value of these 3 components. It should be noted here that very strong UD acceleration with a very short period, which has not yet observed in the past earthquakes, was recorded. **Fig.1** shows the response acceleration spectrum of the IWTH25 record. It is found that the ground motion was very strong in the short natural period range less than 0.3s, but the shaking intensity decreases in longer natural period range than 0.3s. That means that the earthquake seriously affected on the short natural period structures but not so much on the longer natural period structures. For example, typical bridge structures with general spans and general height of substructures have the natural period of 0.5 to 1.0s, therefore, it was estimated that the shaking was not so strong effect on such bridge structures.

2.2 MLIT TEC-FORCE

MLIT has established TEC-FORCE (Technical Emergency Control Force) in April 2008 [2]. Objectives of the TEC-FORCE are to provide technical emergency supports to the local governments where the large scale disasters occur or where the risk of disasters is high. The TEC-FORCE operates in order to make the local governments' countermeasures quicker and more effective. The TEC-FORCE teams are established at MLIT Headquarters Office, NILIM, Geographical Survey Institute (GSI), MLIT Regional Bureau and JMA, and consist of several units including advance unit, field support unit, information communication unit, high tech leading unit, damage investigation unit, emergency operation unit, transport support unit, geographic information unit, meteorological and terrestrial information unit. When a large scale natural disaster occurs, MLIT dispatches the TEC-FORCE and they work to support the local governments through the damage investigation, prevention of damage expansion, and early recovery works.

During the Iwate-Miyagi-Nairiku Earthquake, total number of days and engineers are about 1,400 in the first month after the earthquake. As for this bridge collapse, 5 engineers including bridge experts were dispatched as TEC-FORCE team just after the earthquake to investigate what happened at this bridge [3].

2.3 Bridge Design Condition

The Matsurube Bridge was constructed in 1978 and managed by the Iwate prefectural government. It was 3-span continuous steel girder bridge with length of 94.9m (27m+40m+27m) as shown in **Fig.2**. The width of deck is about 10m. Two piers in the middle spans were RC wall type piers with height of 25m, and both abutments were an inverted T-shaped RC wall type. Ground condition is Type I (Stiff) according to the JRA highway bridge design specifications and therefore all foundations of piers and abutments were a spread type. Seismic coefficient employed in the original design was 0.15 with allowable stress design method. The superstructure consisted of 4 steel plate girders with concrete slab deck. The fixed steel bearing was provided at the A1 abutment and others on two piers and A2 abutment were movable steel bearings in the longitudinal direction. In the transverse direction, all bearings had side stoppers to constraint the transverse movement.

2.4 Damage to Matsurube Bridge

Photo 1 shows an aerial photo of the collapsed bridge. The photo was taken and provided by the Pasco Corporation and the Kokusai Kogyo Co., Ltd. It is understood that P2 pier was collapsed and a part of superstructure between A2 and P1 fell down to the ground. Deck end at A1 abutment also fell down from the seat of the A1 abutment. Looking at the mountain area at the back of A1 abutment, some large cracks and slope deformation were recognized. It was estimated that the landslide in the part of mountain slope occurred and that the slope moved to the bridge and affected on the collapse.

The detailed damage situation of the bridge members is shown in the followings.

1) Superstructure (**Photo 2, 3**)

As wrote in the above, the superstructure between A2 abutment and P1 pier fell down to the ground as a jackknife. Deck end also fell down from the seat of the A1 abutment. At P1 pier, the steel girders were broken and overturned laterally because of the falling down of the superstructure between A2 abutment and P1 pier.

2) A1 Abutment (**Photo 4, 5**)

Road surface at the backward of A1 abutment failed heavily and large cracks and slips of the soils were developed. Significant cracks from parapet wall to the abutment wall were also found at the side of abutment.

3) P1 Pier (**Photo 3, 6**)

Small cracks were observed at the mid-height section of the column, they were possibly at the section of the concrete placement joint. But the columns itself was not damaged remarkably. The pier was inclined so that the pier top is close to A1 abutment and the bottom to the direction of A2 abutment.

4) P2 Pier (**Photo 7**)

P2 pier failed in the particular failure mode. The column was broken and separated into 3 parts. Two parts fell down just around the bottom of original column. Top part of the column just fell down in the A2 abutment side and stood just along the bottom part column. The middle part of column fell down and overturned in the opposite A1 abutment side. The bottom part still remained at the original position.

5) A2 Abutment (**Photo 8, 9**)

The particular failure mode was also found at the A2 abutment. Parapet wall was failed and pushed into the backfill soil. The push-into distance was about 4m. The evidence to show the impact between the deck end and the parapet wall was recognized at the surface of the parapet wall and the end section of the steel girders.

6) Measurement of Geometry of the Bridge

The distance between the locations of substructures was measured by using simple method at the site. Although it is necessary to measure more accurate manner, **Fig.3** shows the measured values. The distance between A1 abutment and A2 abutment shortened to about 85m. Since the original distance was about 95m, so the distance shortened by about 10m. The distance between A1 abutment and adjacent P1 pier was about 26m. Original distance was 27m, so some shortening was found. On the other hand, the distance between P1 pier and A2 abutment was 59m. Since original distance was about 68m, therefore, about 9m shortened. The distance between A1 abutment and P1 pier was not so much change but the distance change between P1 pier and A2 abutment was about 10m. That means that the A1 abutment and P1 Pier moved toward P2 pier and A2 abutment.

2.5 Estimation of Damage Mechanism

Based on the detailed damage investigation and the measurement of the distance between substructures, the damage mechanism was estimated. The more detailed measurement and investigation including the boring of soils was needed but the failure was estimated to be developed in the following mechanism. **Fig.4** shows the estimated sequences of the collapse of the bridge.

1) The failure and slide of the mountain slope was developed by the earthquake shaking and the abutment and pier which were supported by the slope moved with the landslide.
2) In particular, considering the evidence of the heavy damage and cracks at the backside soil of A1 abutment, A1 abutment and P1 pier moved to the direction of A2 abutment. At the same time, the superstructure was pushed to the direction of A2 abutment and P2 pier. That resulted in the collapse of P2 pier and damage to the parapet wall of A2 abutment, then the superstructure fell down like a jackknife.
3) P1 pier was separated into 3 parts. The column had two termination sections of longitudinal re-bars at mid-height. It was estimated that the terminated section became relative weak sections when the column was subjected to the extreme large displacement. The upper section with termination of longitudinal re-bars of P2 pier firstly failed because of the large displacement of superstructure, the top part fell down to the A2 abutment side. Then the superstructure was broken and settled down because of dead weight with losing the support from P1 pier and the continuous displacement from the slope, finally the superstructure fell down as a jackknife. When the falling down of the superstructure to the ground, the middle part of column of P2 pier was overturned to the P1 side with the falling superstructure.

3. ANLYTICAL SIMULATION

3.1 Bridge Model

To simulate this particular bridge collapse mode, the preliminary analytical study was conducted. There were discussions that the damage was caused by the ground shaking, ground displacement or both. But the critical damage was developed only at P2 pier but almost no damage to P1 pier which had completely the same dimensions and re-bar arrangement with P2 pier. Therefore it was estimated the damage was caused by the displacement rather than the ground shaking. This estimation corresponded to the response spectrum of observed data. Therefore, the pushover analysis method applying the displacement to the foundations of A1 abutment and P1 pier was employed in this study. Dynamic analysis was also made using the observed strong motion data but in this preliminary study the pushover analysis is shown here.

Fig.5 shows the mathematical model of the bridge. The superstructure was modeled as concrete slab and 4 steel girders considering the nonlinear behavior of materials. Movable bearing was modeled as nonlinear spring element considering the strength and fail of side stoppers. RC columns were modeled using fiber element model. The termination sections of the longitudinal re-bars were also considered. Abutments were modeled as linear element but the parapet walls were modeled as nonlinear fiber element model. Also to simulate the push-out failure of the parapet wall, the nonlinear shear spring element was provided at the bottom of the parapet wall. Backfill soil was model as spring element and the general soil stiffness was assumed because of the lack of the accurate soil data. Since foundations were spread type on the stiff ground, they are model as fixed to the ground.

In the pushover analysis, the displacements were given in the 3 dimensional directions. The displacement at A1 abutment and P1 pier used in this analysis was by the detailed measurement of the bridge by the Iwate prefectural government [4].

3.2 Preliminary Analytical Results

Fig.6 shows the preliminary results obtained from the pushover analyses. Upper section of the termination of longitudinal re-bars was found to be failed firstly and the top part of P2 pier pushed to the direction of A2 abutment. It is estimated from the analyses that the failure and falling down of the top part of the P2 pier was caused by the large displacement of the superstructure. The failure of the parapet wall of A2 abutment also found because of the push-into of the end of the superstructure. Therefore, the failure mode in the first stage was almost reproduced by the simulation analysis.

4. LESSONS LEARNED AND DISCUSSIONS

The damage mode of the Matsurube Bridge was particular. The landslide of the backward slope of the bridge caused the large displacement of about 10m and the abutment and pier were displaced with the landslide. Then the pier failed and the superstructure broken as a jackknife. The collapsed pier was separated into 3 parts and the parapet wall pushed into backfill soils by the superstructure. These damage modes had not found in the past earthquakes.

The preliminary lessons and discussions are summarized as follows.

1) The damage was basically caused by the landslide. Therefore, it is important to investigate the possibility of the occurrence of landslides around the bridge construction sites.
2) At this moment, it is very difficult to evaluate the earthquake shaking intensity accurately when the landslides occur at each slope. It is important to study and to accumulate the data in order to improve the slope investigation method and the stability evaluation method.
3) It is impossible to stop the movement of large slope by a bridge structure. In the structural design point of view, it is essential to select the route carefully and to study the design consideration on the placement of foundations. Also, it is important to study the methods to improve the redundancy of road networks.

It should be noted here that Iwate prefectural government established the investigation committee to study the damage mechanism of the Matsurube Bridge (Chairman: Prof. Motoyuki Suzuki, Tohoku University) more in detail. The final report will be published soon. The interim report already can be downloaded from the Iwate prefecture web site as [4].

5. REFERENCES

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3. MLIT TEC-FORCE Team: Report of Investigation of Matsurube Bridge on National Highway Route #342, Reported to the Iwate Prefectural Government and Media, June 18, 2008 (In Japanese)
4. Iwate Prefectural Government : Committee Report on Damage Investigation of Matsurube Bridge on natibal Higway Roure #342 (In Japanese) (http://www.pref.iwate.jp/view.rbz?of=1&ik=0&pnp=14&cd=17790)

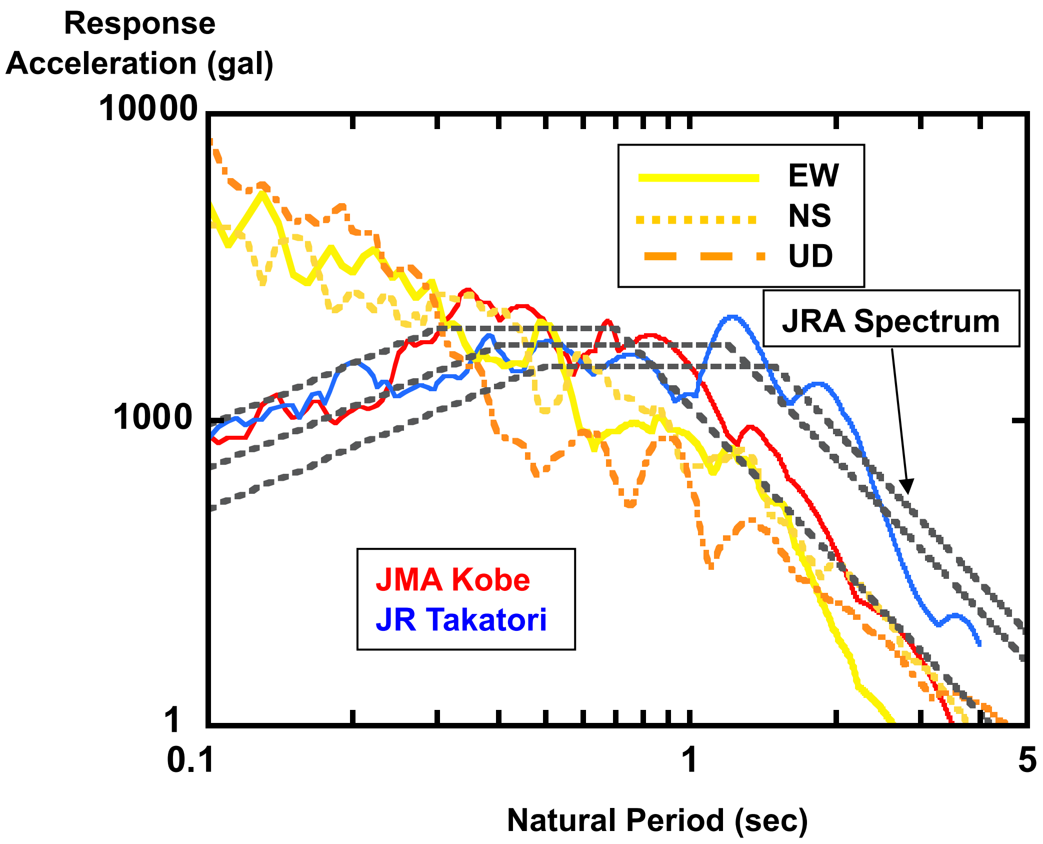


Fig. 1 Response Acceleration Spectrum of IWTH25 Station (NIED, KiK-Net)

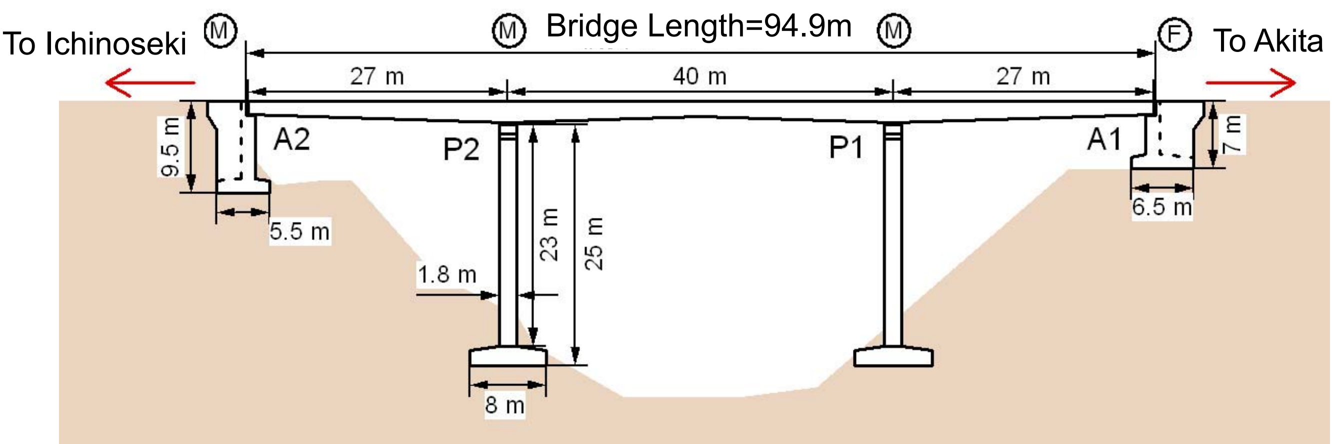


Fig. 2 Side View of the Matsurube Bridge

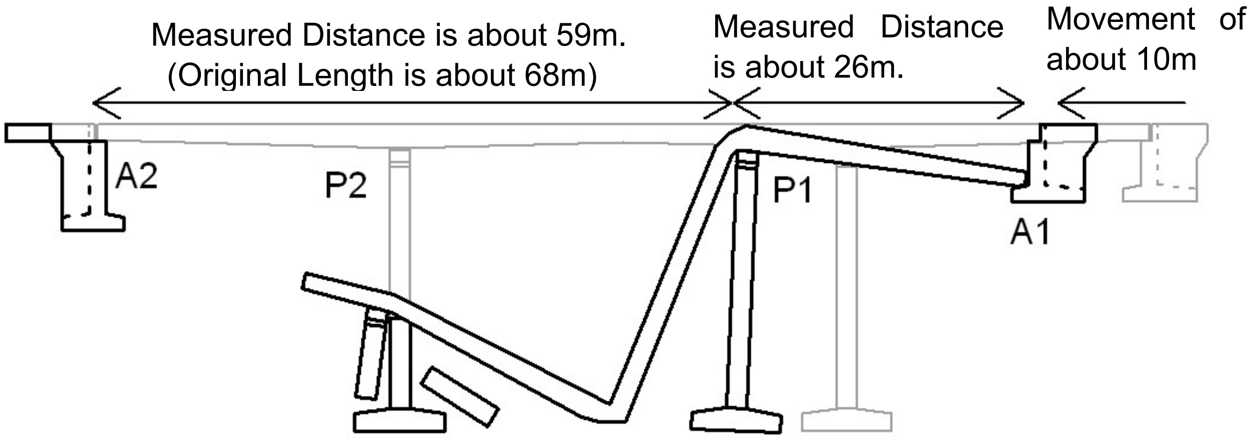


Fig. 3 Measured Distance Changes between Substructures

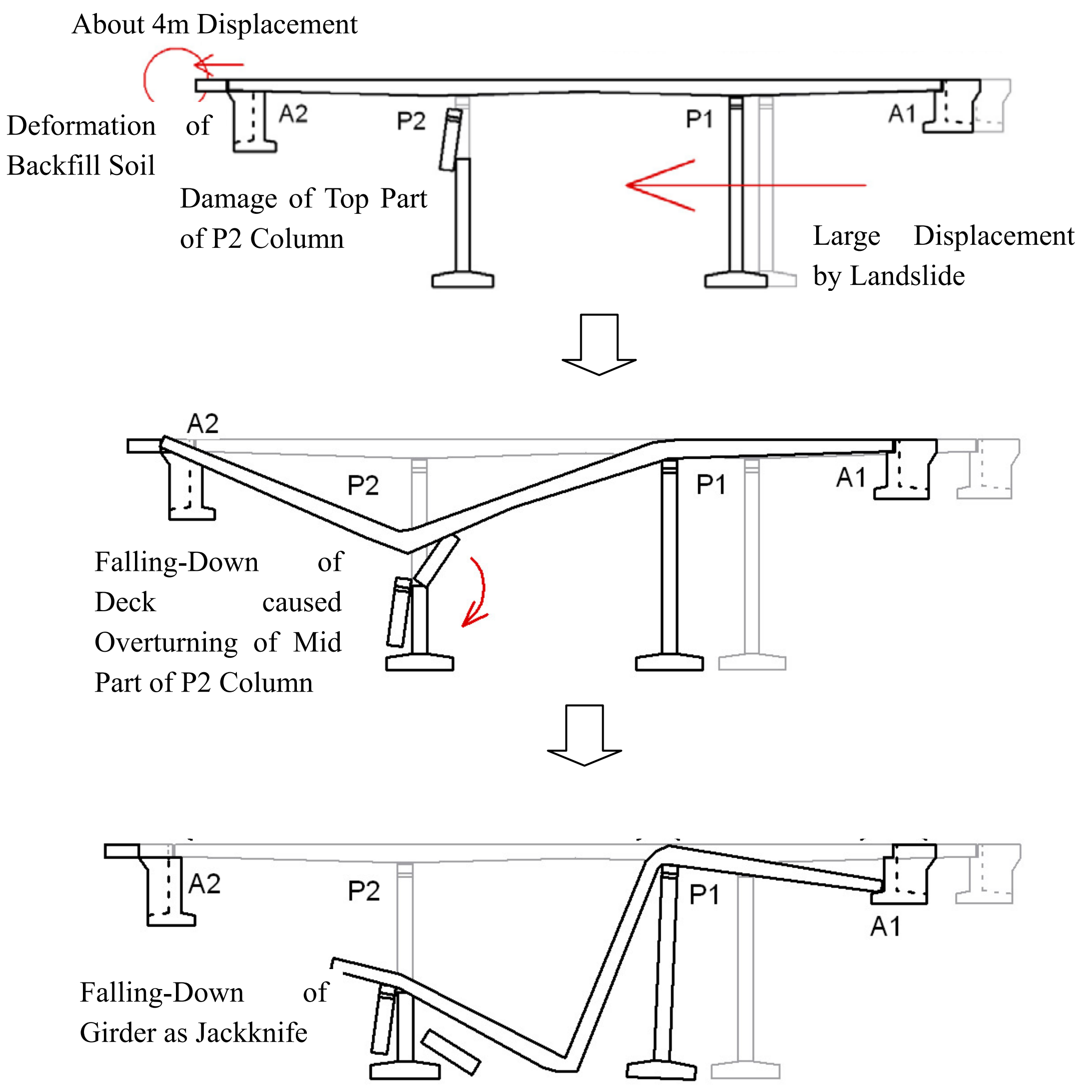
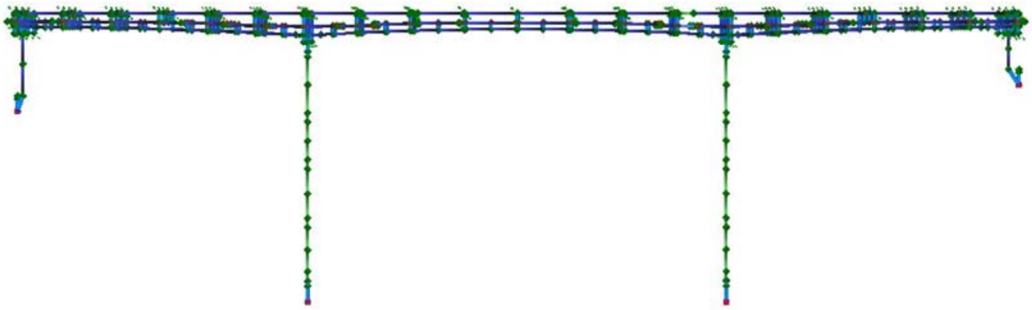
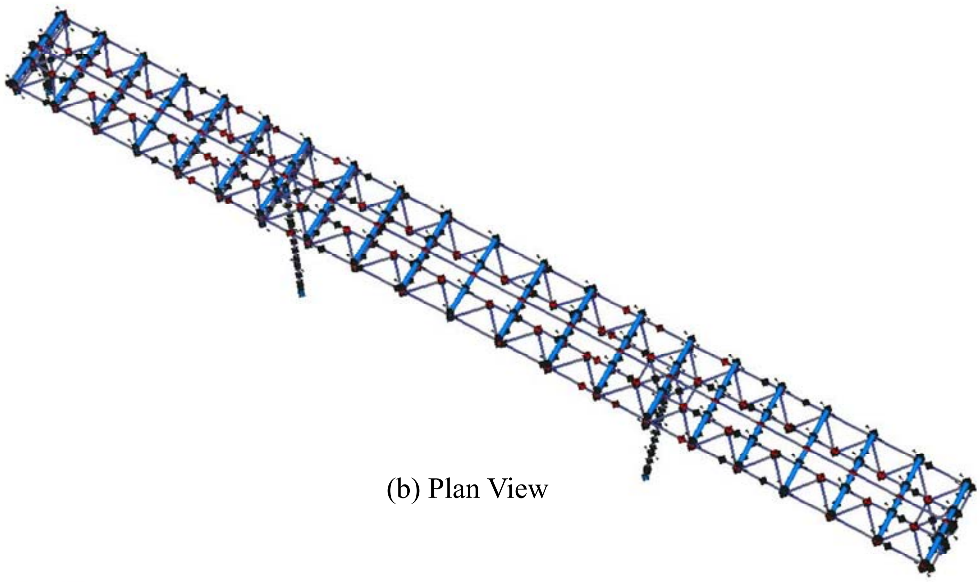


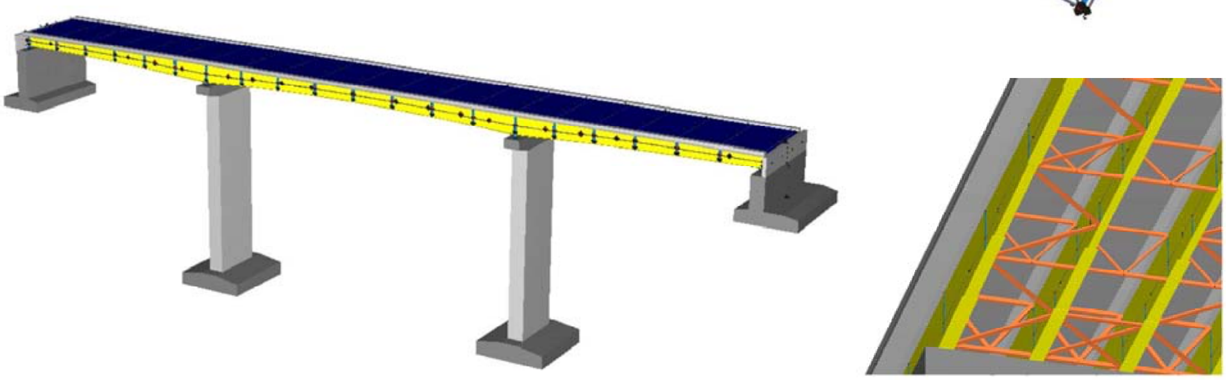
Fig. 4 Estimated Sequences of the Bridge Collapse



(a) Side View

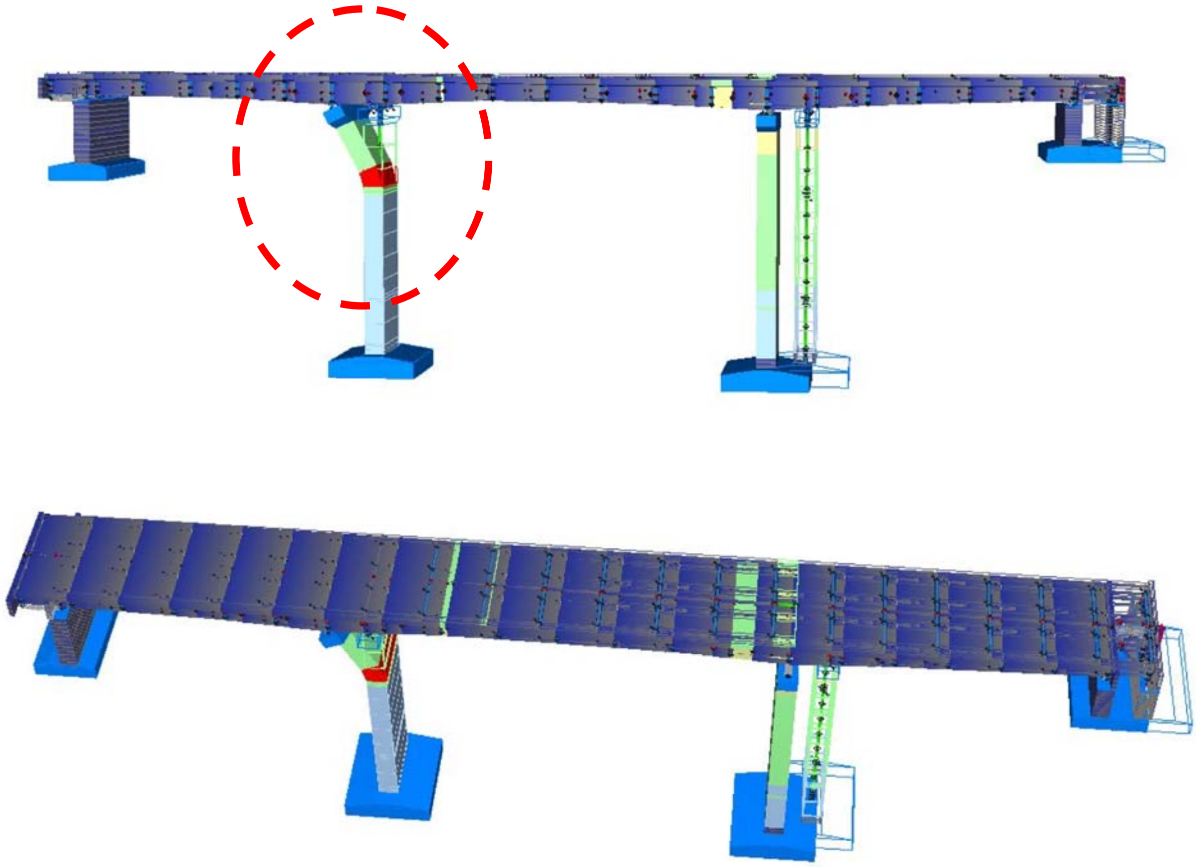


(b) Plan View

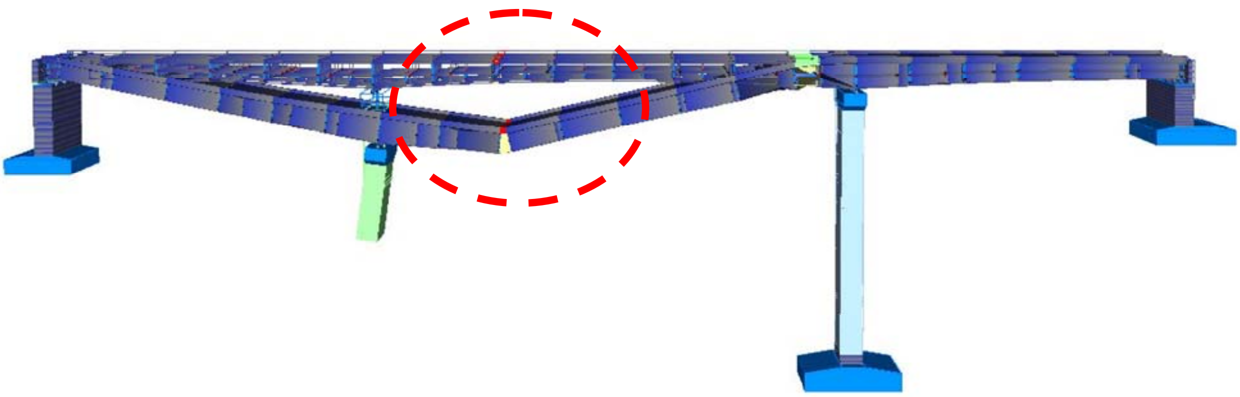


(c) Solid Illustration

Fig. 5 Mathematical Model to Simulate Bridge Collapse



(a) Failure of Top Part of P2 Pier caused by the Displacement



(b) Possibility of Jackknife Failure of Girder with Losing Vertical Support by P2 Pier

Fig. 6 Preliminary Results of Push-over Analyses to Simulate the Bridge Collapse

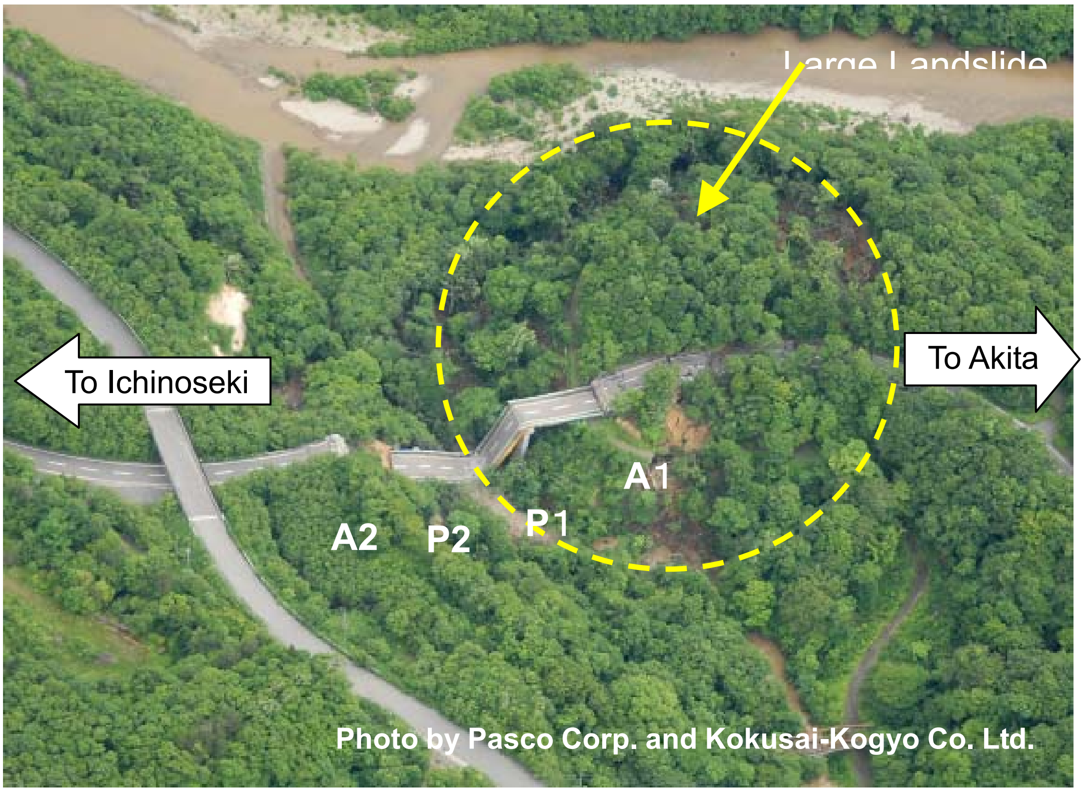


Photo 1 Aerial Photo of Matsurube Bridge

Photo 2 Collapse of Superstructure

Photo 3 Damage of Girder on P1 Pier

Photo 4 Large Cracks and Slips of Backfill Soils at A1 Abutment

Photo 5 Crack at Slopes in the Back of A1 Abutment

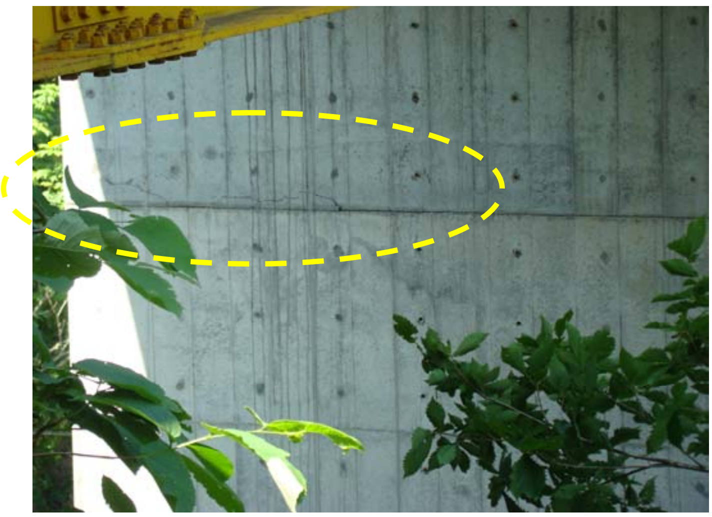


Photo 6 Crack at the Pier P1 (Around Joint of Concrete Placement)

Photo 7 Damage of P2 Pier

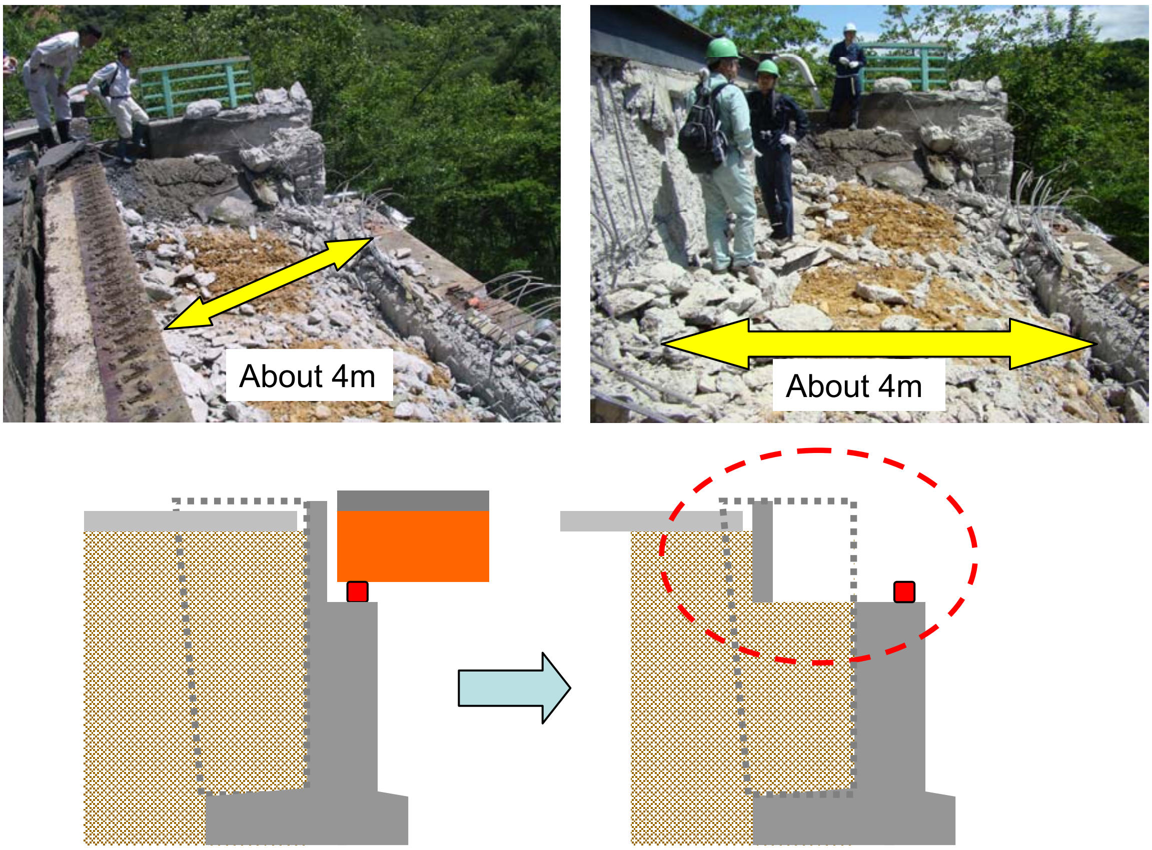
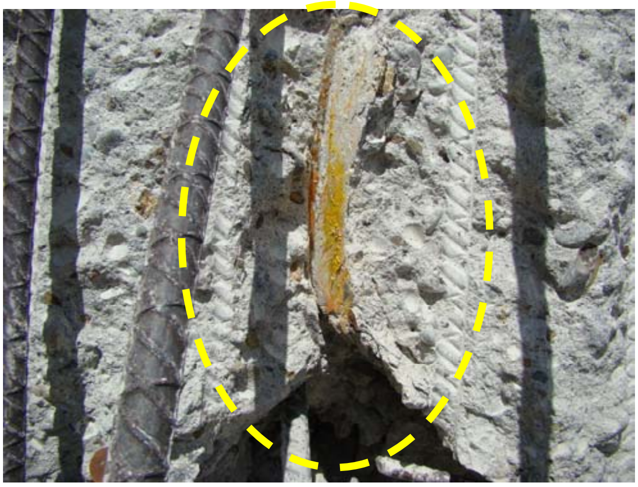


Photo 8 Damaged of A2 Abutment (Failure of Parapet Wall)



(a) Surface of Parapet Wall

(b) Girder End

Photo 9 Evidence of Impact between Parapet Wall and Girder End