FAILURE MECHANISM OF THE FURYO DAIICHI BRIDGE IN THE 2016 KUMAMOTO EARTHQUAKE

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Abstract

In the Kumamoto Earthquake that occurred in western Japan in 2016, the Furyo Daiichi Bridge, a 60 m-long overbridge, collapsed onto the busy Kyushu Expressway. The bridge, constructed in 1974, had a skew angle at one end of the superstructure. To confine displacements in the perpendicular direction, displacement confining devices were installed at the abutment on the skewed side of the superstructure. The two piers located on the dividing strips were rocking piers. The rocking piers had pivot bearings at both the upper and lower ends of the columns. The pivot bearings were incorporated to provide a rotating function and a vertical load support function.

Authors inferred a failure mechanism of the bridge by examining the actual damage. We focused on the punching shear failure at the connection of the displacement confining device and the bridge seat, because it was rather an unordinary damage. We estimated the punching shear resistance of the connection area and compared it with the design seismic force. We also estimated the allowable displacement of the rocking piers. Through these estimations, we inferred that after the collision of the main girder and the displacement confining device, the large colliding force was transferred to the device-bridge seat connection area and caused a punching shear failure at that position. The superstructure was further moved to the perpendicular direction because of large colliding force, and fell onto the ground when the displacement limit of the rocking piers was exceeded.

Next, we conducted a dynamic analysis to evaluate the reproducibility of the inferred damage mechanism. A 3D nonlinear beam model was adopted for the analysis. We also created four analysis cases using different strengths of the plate bearing and the displacement confining device. Of the four cases, Case 3 was assumed as the most likely case to show the behavior closest to the actual behavior.

According to the analysis results on Case 3, the bridge collapsed when the pivot bearings lost a vertical load support function by exceeding a rotation limit at 24.30 sec. It coincided with the time when a large waveform appeared for the first time in the waveform chart recorded at the nearby location. It was found that the failure order of the bridge components was the plate bearings at Abutment A1, plate bearings at Abutment A2, displacement confining device at Abutment A1, and pivot bearings at Pier P2. It was also found that the bridge collapse was caused not only by the rotational movement of the superstructure due to a skew angle, but also by the horizontal movement of the superstructure due to the seismic forces.

As the improvement measures for this type of bridge, the authors propose to increase the punching shear resistance at the connection area of the displacement confining device and the bridge seat, and to increase the number of displacement confining devices installed.

Keywords: overbridge; rocking pier; pivot bearing; displacement confining device; punching shear failure

1. Introduction

The Kumamoto Earthquake occurred in Kyushu Island of Japan on April 16, 2016. Earthquake ground motion with a moment magnitude of 7.0 (Mw) caused serious damage to a number of bridges. One of the damaged bridges is the Furyo Daiichi Bridge in Kumamoto Prefecture. It was an overbridge crossing over the Kyushu Expressway. The bridge collapsed onto the expressway due to large displacements to the perpendicular direction, and it seriously hampered the transport of emergency materials.

The Furyo Daiichi Bridge was constructed in 1974. It was a three-span continuous prestressed concrete hollow slab bridge, about 60 m in length and 8.5 m in width. The bridge was built on two rocking piers. The rocking piers had pivot bearings at both the upper and lower ends of the columns. Displacement confining devices were installed at the abutment to restrict perpendicular displacements of the superstructure. The bridge is considered to have collapsed when one of the displacement confining devices fell off and pivot bearings lost their functions.

In this paper, we firstly show the damage occurred to the Furyo Daiichi Bridge. Secondly, we estimate a possible damage mechanism inferred from the sustained damage. Lastly, we perform dynamic analysis to identify an actual failure mechanism and propose improvement measures for the bridge.

2. State of damage

The Furyo Daiichi Bridge overcrossing the Kyushu Expressway had two abutments (A1, A2) and two rocking piers (P1, P2). The rocking piers had pivot bearings that play a rotating function and a vertical load support function. Displacement confining devices were installed on both sides of the main girder at Abutment A1 only, because the bridge was skewed (60°) at this position. The displacement confining devices were square-shaped (1×1 ×0.4 m) reinforced concrete blocks anchored to the bridge seat as an added structure. They were expected to confine displacements in the perpendicular direction.

According to the West Nippon Expressway Company, the displacement confining device on the north side was damaged when a foreshock (Mw 6.2) occurred two days earlier. The bridge showed slight displacement at the time. But, when the main shock (Mw 7.0) came two days later, the bridge collapsed to the ground.

Photo 1 shows the collapsed portion of the bridge near Abutment A1. Photo 2 shows the damage to the displacement confining device. Fig. 1 gives a plan view of the Furyo Daiichi Bridge before and after the earthquake.

As shown in Fig. 1, the bridge was displaced to the north about 5 m at Abutment A1 and about 3 m at Abutment A2. It is considered that the skewed superstructure made a rotating movement to the north and collided against the displacement confining device. A punching shear failure occurred at the connection area of the confining device and the bridge seat, forcing the device and the superstructure to fall off, as seen in Photo 2. Unlike Abutment A1, a displacement confining device was not installed at Abutment A2 as there was no skew angle at that position. Considering the fact that the rocking piers are vulnerable to movements, we can point out that the displacement confining devices of this bridge were not sufficient to resist large movements in the axial and perpendicular directions.

3. Evaluation of deformation behavior

This bridge suffered unordinary damage, namely, a punching shear failure occurred at the connection area of the displacement confining device and the bridge seat, as seen in Photo 2. To evaluate the behavior of the connection area, we derived two different punching shear resistances of the area and compared them with the design seismic force. For this calculation, two typical shear stress values specified in Japan's Bridge Specifications were used [1].

One is the shear stress specified for bridge seat calculation (τ c=0.22N/mm^2). The other is the shear stress specified for the punching shear resistance calculation of various bridge portions (τ c=0.85 N/mm^2) .

3.1 Calculation of punching shear resistance using 0.22 N/mm^2

The punching shear resistance of concrete (Vc) at the connection area is calculated as 933 kN from Eq. (1) if the shear stress value 0.22 N/mm^2 is adopted.

$$Vc = \tau c \cdot Ac \tag{1}$$

where, τc: shear stress of concrete, 0.22 N/mm^2
Ac: resisting area of concrete
The total resisting area of concrete is 4,242,000 mm^2 from Ac = A1 + A2 + A3 in Fig. 2.

The punching shear resistance of reinforcing steel (Vs) at the connection area is calculated as 143 kN from Eq. (2).

$$Vs = \beta \cdot (1\text{-}hi/da) \cdot \sigma sy \cdot As \tag{2}$$

where, β : correction factor, 0.5
hi: depth of reinforcing steel from the surface of the bridge seat,150 mm
da: length from the center of the anchor bolt in the back row to the edge of the bridge seat, 1,000 mm
σ sy: yield point of reinforcing steel, 295 N/mm^2
As: cross sectional area of reinforcing steel, the total cross sectional area of nine D13 reinforcing steels is adopted, 114 mm^2 (12.67×9).

Using Eq. (3), the punching shear resistance (V) of the connection area is calculated as 1,076 kN (equivalent to 1.7 Rd, where Rd is a dead load reaction).

$$V = Vc + Vs \tag{3}$$

The values for Vc and Vs are 933 kN and 143 kN, respectively, from the earlier calculation.

3.2 Calculation of punching shear resistance using 0.85 N/mm^2

As the shear stress value used in the above calculation is a conservative value to keep safety, we also used less conservative shear stress value for the punching shear resistance calculation.

The punching shear resistance of concrete (Vc) at the connection area is calculated as 3,606 kN from Eq. (1) if the punching shear stress 0.85 N/mm^2 is adopted. As for the punching shear resistance of reinforcing steel (Vs), 143 kN obtained in Subsection 3.1 is used. Then, the punching shear resistance (V) of the connection area becomes 3,749 kN from Eq. (3) (equivalent to 6.0 Rd, where Rd is a dead load reaction).

The derived punching shear resistance 3,749 kN is three times of 1,076 kN derived in Subsection 3.1. In other words, the connection area in Subsection 3.2 has three times larger punching shear resistance capacity than the connection area in Subsection 3.1.

3.3 Check against seismic forces

Here, we assume that the weight of the superstructure (w) is supported by the two displacement confining devices. Then, the design seismic force (Hs) is derived as 1,554 kN from Eq. (4).

$$Hs = 3 \cdot kh \cdot W/2 \tag{4}$$

where, kh: horizontal seismic coefficient, 0.15
W: weight of the superstructure estimated from the design drawing, 6,906 kN

If the derived design seismic force 1,554 kN is compared with the punching shear resistance of the connection area 1,076 kN which was derived in Subsection 3.1, it means that the displacement confining device will fail. On the other hand, if 1,554 kN is compared with the punching shear resistance of the connection area 3,749 kN which was derived in Subsection 3.2, it means that the displacement of the superstructure will be confined although cracking will occur to the displacement confining device.

3.4 Estimation of allowable displacement

1) *Clearance between the main girder and the displacement confining device*

There was an 11 cm clearance between them. A neoprene buffer (150×900×50 mm) was inserted there. This space could accommodate displacements in case of an earthquake.

2) *Displacement of rocking piers*

The rocking pier shown in Fig. 3 is 245 cm in height. The allowable displacement of the rocking pier is obtained by Eq. (5).

$$Dm = H \cdot \tan(\theta) \tag{5}$$

where, H: pier height, 245 cm

θ : allowable rotation of pivot bearing, 0.06 (rad)

If 0.06 (rad) is adopted as the allowable rotation of pivot bearings based on the experiment conducted at the Railway Technical Research Institute, the rocking pier will fail when it is displaced 14.7 cm by reaching the rotation limit of the pivot bearing 0.06 (rad) [2].

3.5 Estimation of damage mechanism

Based on the above results, the damage mechanism of the bridge was estimated as follows:

1) The plate bearings on the bridge seat failed due to a large seismic force in the perpendicular direction.
2) The superstructure was displaced 11 cm and collided against the displacement confining device on the north side. The displacement confining device did not suffer a shear failure because the device, or a concrete block, was short in height. Instead, the colliding force was transferred to the connection area of the device and the bridge seat, and caused a punching shear failure at that position. As the colliding force was so large, the superstructure was further moved to the perpendicular direction.
3) The bridge collapsed when the rocking piers were displaced more than the allowable displacement 14.7 cm by exceeding the rotation limit of the pivot bearings 0.06 (rad).
4) Considering from this failure mechanism, the effective improvement measures are to increase the punching shear resistance of the device-bridge seat connection area and to increase the number of devices to be installed.

4. Outline of dynamic analysis

There is a possibility that the cause of collapse of the Furyo Daiichi Bridge was not only the rotation of the rocking piers but also the insufficient strength of the displacement confining device at Abutment A1. To investigate this possibility, we performed dynamic analysis with a focus on the strengths of the horizontal displacement confining device and the plate bearings installed at the abutment. As the input waveforms, the waveforms of the main shock recorded at the nearby Mifune Interchange were used. According to the acceleration response spectrum, large acceleration occurred around 0.5 sec and 1.0 sec. Some researchers pointed out that the acceleration response at that location was the amplified response due to the effect of the ground conditions compared with the responses at other locations [3]. A 3D nonlinear beam model shown in Fig. 4 was employed for the analysis.

A linear model was used to model all the abutments and piers, because there was no buckling damage to the steel pier columns and no shaking damage to the abutment body. A foundation spring (concentrated spring) was placed at the bottom of the abutments and piers. As the damping constant, 2% was adopted for the substructure, and 20% for the foundation spring in view of the ground type, Type II - intermediate quality. Free rotation was assumed for the pivot bearings at the upper and lower ends of the pier columns. The viscous damping was set in proportion to the rigidity of each element.

Fig. 5 shows the presumed relationship of nonlinear characteristics vs. actual behavior for the displacement confining device and the plate bearings at the abutment. The damage to the displacement confining device was said to be the ductile failure type in which shear reinforcing steels yielded progressively [4]. The model adopted was the type that can absorb energy through hysteresis damping, and the damping constant for each member was set as 0%. We adopted this model although it was unclear how much colliding energy would be absorbed by the rubber buffer and how much energy would dissipate after the failure of the plate bearings and the displacement confining device. The plate bearing resistance at Abutment A2 was about 1.7 times larger than the plate bearing resistance at Abutment A1 because the pier span length was not equal. Also, we did not set a collision spring in the axial direction because there was no trace of the girder-parapet collision in the axial direction [5].

Table 1 shows the four analysis cases that used different strengths of the plate bearing and the displacement confining device as the parameters. It is because a nonlinear behavior of the plate bearing is difficult to express in the analysis model. Of the four analysis cases, Case 3 was assumed as the basic case showing a behavior closest to the actual behavior by taking the following into account: **a)** In an actual earthquake, the friction between the superstructure and the substructure would have a dominant effect after the failure of the plate bearing; and **b)** From the past experimental results, it was considered that the strength of the displacement confining device was close to the strength obtained by using 0.22 N/mm^2 as the shear stress value.

5. Analysis results of Case 3

The analysis results of Case 3 (basic case) are explained here. Fig. 6 shows the time history response of the rotation of the pivot bearings installed at the upper and lower ends of Pier P2. The horizontal and vertical axes show the elapsed time and the rotation angle of the pivot bearings, respectively. The two parallel lines in Fig. 6 indicate the rotation angle 0.06 (rad) which is the rotation limit of the pivot bearings. The figure shows that the rotation angle of the pivot bearings exceeded the rotation limit 0.06 (rad) at 24.30 (sec). The figure also shows that the pivot bearings rotated to the obtuse angle side (south side) as well before the failure. Fig. 7 depicts the displacement of the entire bridge structure at 24.30 (sec) when the pivot bearings failed and the bridge collapsed. It is also the time when a large waveform appeared for the first time in the recorded waveform chart.

Figs. 8 and 9 show the time history response of the plate bearings at Abutments A1 and A2, respectively, when subjected to seismic forces. The horizontal and vertical axes show the elapsed time and the horizontal force acting on the plate bearings, respectively. The parallel lines in those figures indicate the yield strength of the plate bearings. As the rigidity after the yielding point is set at 1/10000, the hysteresis curve follows the yield strength line even though the yield strength of the plate bearings has exceeded those lines.

Fig. 8 indicates that the plate bearings at Abutment A1 collided six times to the north and four times to the south, including small collisions, before the collapse of the bridge. Fig. 9 shows that the plate bearings at Abutment A2 collided three times to the north and four times to the south before the collapse of the bridge. On the other hand, the displacement confining device on the north side at Abutment A1 was damaged at 24.28 (sec) and fell off almost immediately with the bridge. Fig.10 shows the displacement of the piers P1 and P2 at 24.30 (sec). Fig.11 shows the displacement of the girders at Abutment A1 and Abutment A2 at 24.30 sec.

From the analysis results of Case 3, we found that the failure order of the bridge members at the Furyo Daiichi Bridge prior to the bridge collapse was: ① plate bearings at Abutment A1 (20.85 sec) → ② Plate bearings at Abutment A2 (21.16 sec) → ③ displacement confining device on the north side at Abutment A1 (24.28 sec) → ④ pivot bearings at Pier P2 (24.30 sec). The analysis also reproduced the fact that no serious damage had occurred to the displacement confining device on the south side of Abutment A1 during the earthquake [5]. This in turn means that our dynamic analysis captured the actual failure process rather well.

Fig. 12 describes the time history response of the end portion of the superstructure at Abutment A1 under seismic forces. The horizontal and vertical axes show the elapsed time and the horizontal displacement in the perpendicular direction, respectively. The horizontal displacement moved back and forth before the bridge collapse time of 24.30 sec. At 24.30 sec, the displacement at Abutment A1 was 158 mm and the displacement at Abutment A2 was 144mm, as shown in Fig. 7 and 11. The end portion of the superstructure was moved to the north significantly due to the effect of not only the rotational motion but also the large acting force to the north. This suggests that a displacement confining device with sufficient strength should be installed on both sides of the main girder at both abutments without regard to whether there is a skew angle or not.

6. Conclusions

We conducted a dynamic analysis to investigate the damage mechanism of the Furyo Daiichi Bridge that collapsed in the 2016 Kumamoto Earthquake due to the failure of the rocking piers. The following conclusions were drawn from the analysis results:

1) The most probable damage process derived from the dynamic analysis is: the superstructure was displaced when the plate bearings and the displacement confining device on Abutment A1 were damaged. Immediately after that, the pivot bearings on the rocking piers lost a vertical load support function by exceeding a rotation limit, which caused the collapse of the bridge.
2) According to the analysis results, the damage to the plate bearings and the displacement confining device at Abutment A1 was caused, not only by the rotation of the superstructure due to a skew angle, but also by the movement of the superstructure to the perpendicular direction due to the seismic force.
3) The superstructure is often moved back and forth in the perpendicular direction when subjected to seismic forces. Therefore, as the improvement measures for this type of bridge we propose to increase the punching shear resistance of the connection area of a displacement confining device and the bridge seat, and to increase the installation of displacement confining devices to all four positions, on both sides of the main girder at both abutments.

7. References

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Photo 1 Collapsed bridge near Abutment A1

Photo 2 Damage to the displacement confining device

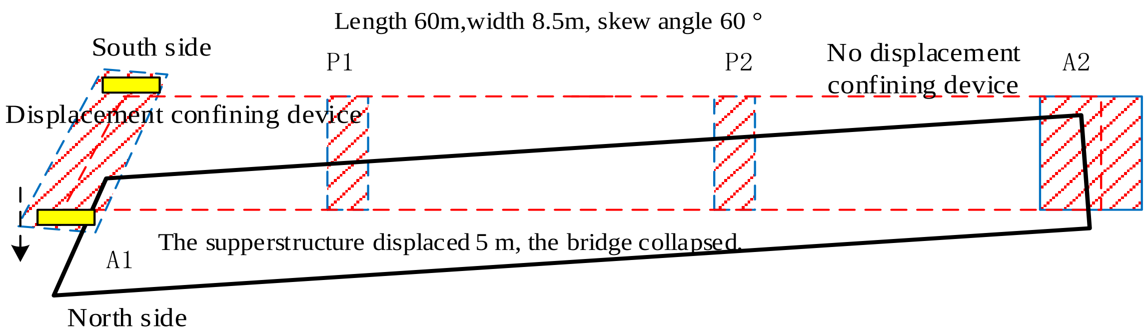

Fig. 1 Plan view of the Furyo Daiichi Bridge before and after the earthquake.

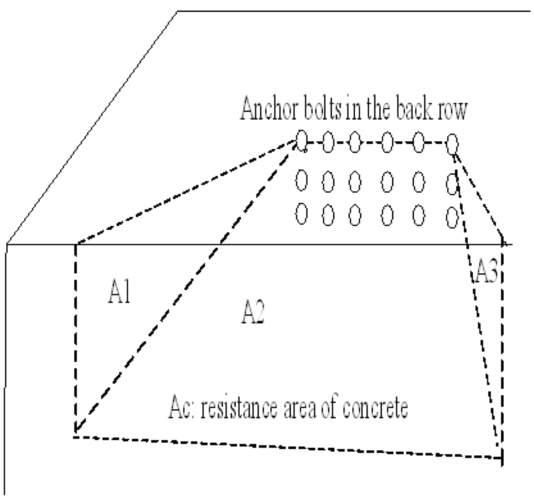

Fig. 2 Planes of punching shear failure

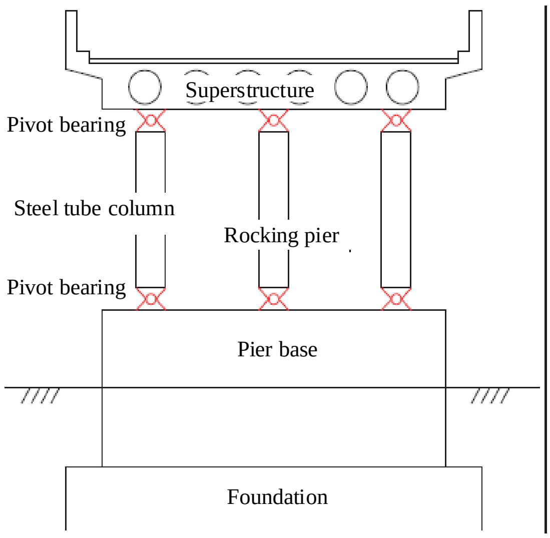

Fig. 3 Cross sectional view of the rocking pier

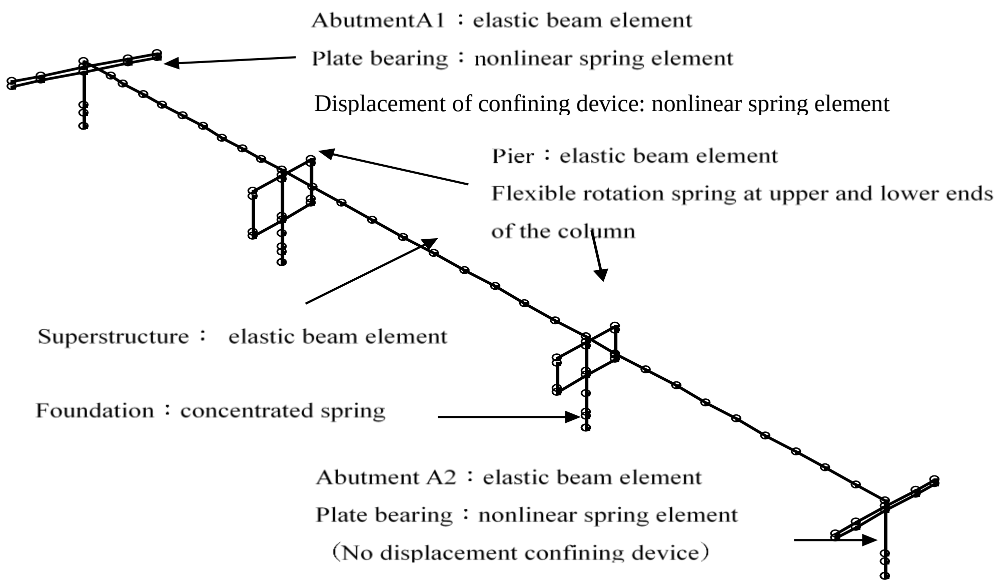

Fig. 4 Analysis model

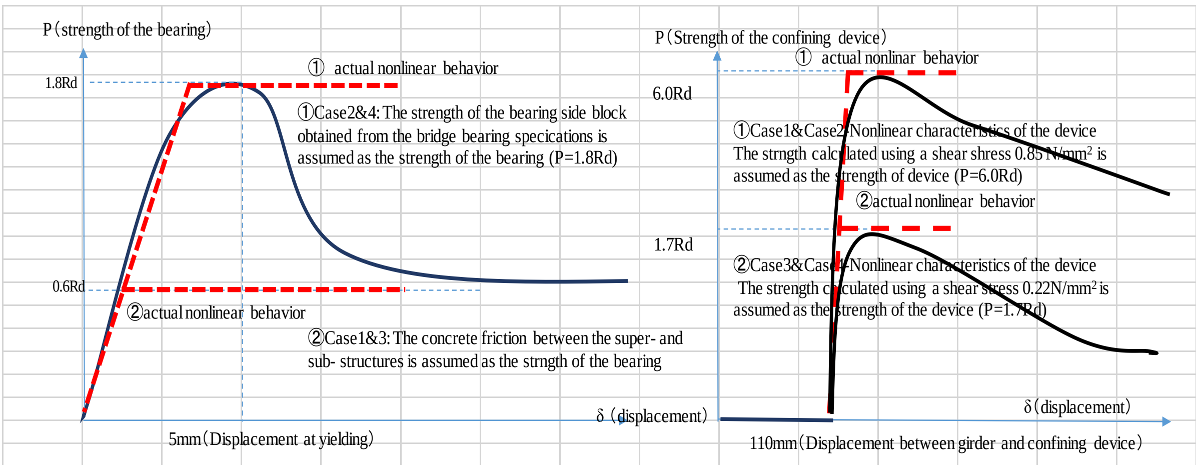

Fig. 5 Presumed relationship of plate bearing and confining device

Table 1 Analysis cases

Case	Strength of the bearing	Strength of the device	Remarks
Case 1	0.6 Rd (friction of concrete planes)	6.0 Rd (when shear stress is assumed as 0.85 N/mm^2)	The design strength of the confining device is evaluated.
Case 2	1.8 Rd (strength of bearing side block)	6.0 Rd (when shear stress is assumed as 0.85 N/mm^2)	The strengths of the bearing & device are evaluated most highly.
Case 3 (Basic case)	0.6 Rd (friction of concrete planes)	1.7 Rd (when shear stress is assumed as 0.22 N/mm^2)	The case that shows the behavior closest to the actual behavior
Case 4	1.8 Rd (strength of bearing side block)	1.7 Rd (when shear stress is assumed as 0.22 N/mm^2)	The actual strength of the bearing side block is evaluated.

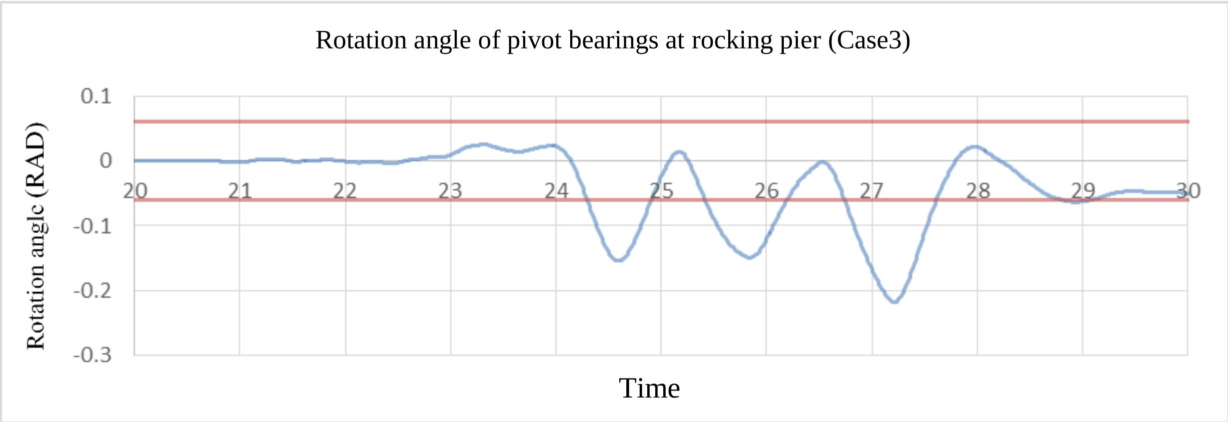

Fig. 6　Time history response of pivot bearings at rocking pier P2 (Case3)

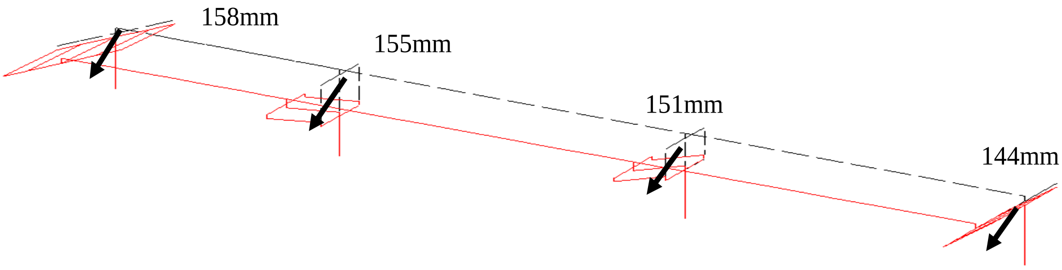

Fig. 7　Displacement of the entire bridge at 24.30 (sec) when pivot bearings failed (Case3)

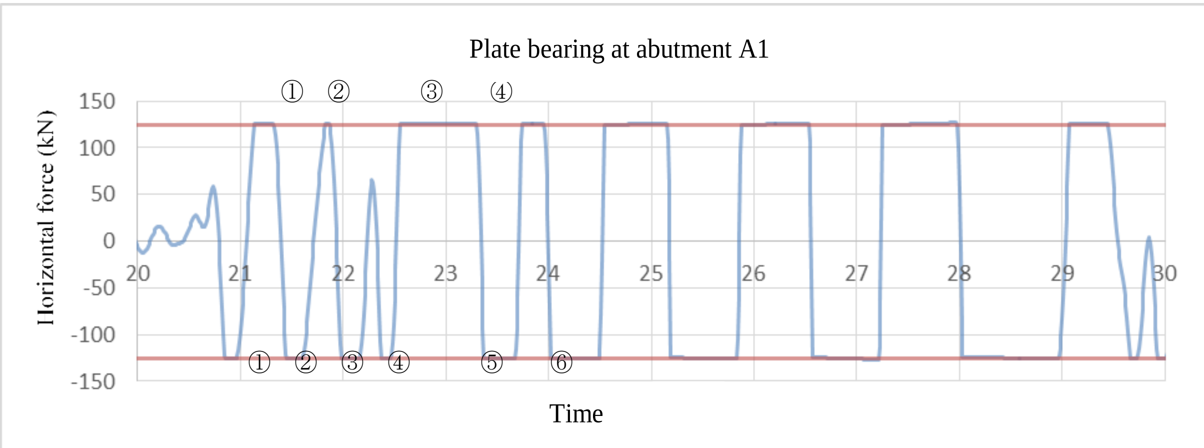

Fig. 8　Time history response of the plate bearing at abutment A1 (Case3)

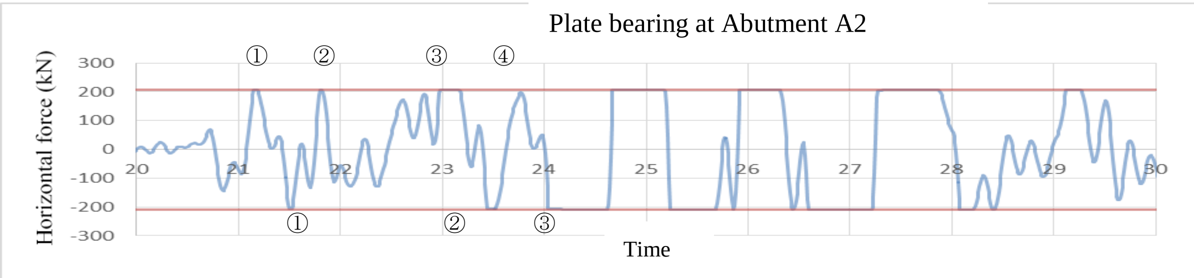

Fig. 9 Time history response of the plate bearing at Abutment A2 (Case3)

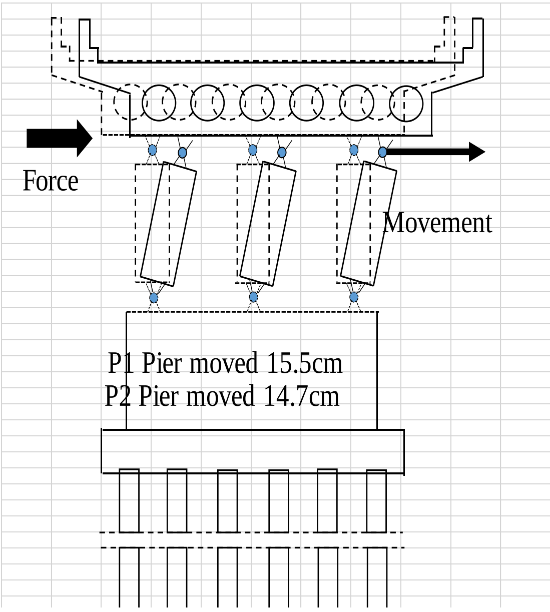

Fig. 10 Displacement of Piers P1 and P2 at 24.30 sec

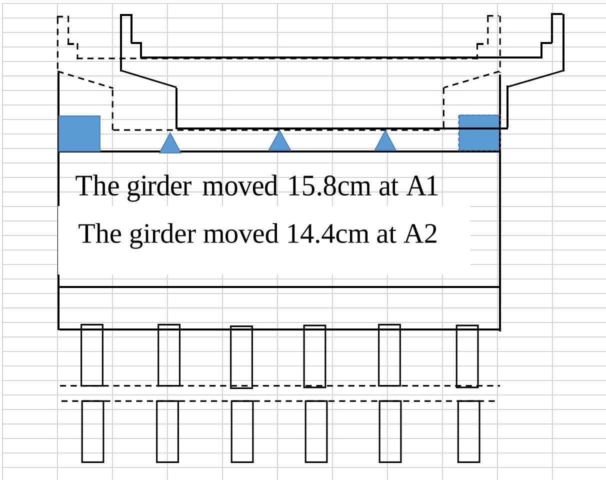

Fig. 11 Displacement of the girders at A1 and A2 at 24.30 sec

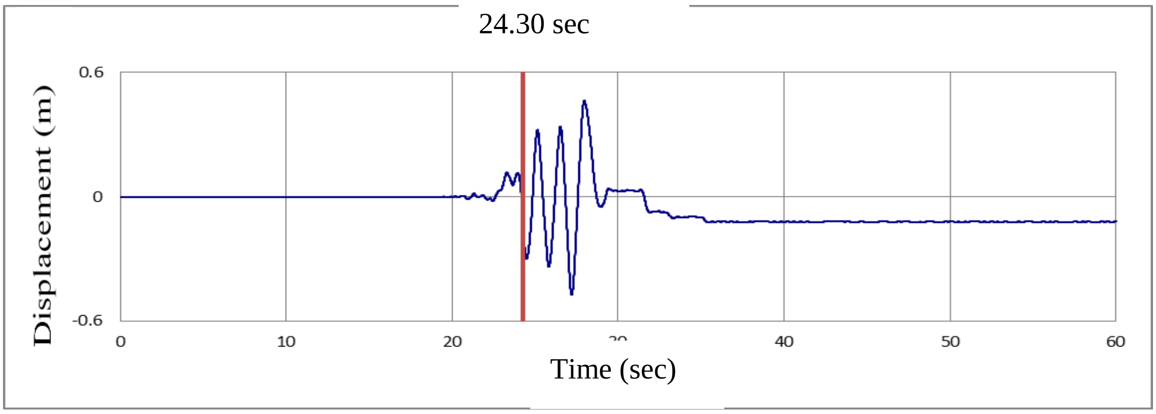

Fig.12 Time history response of the girder at Abutment A1(Case3)