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Damage analyses on the collapse of Aso-ohashi during the 2016 Kumamoto Earthquake

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Abstract

During the 2016 Kumamoto Earthquake, Aso-ohashi on National Highway route #325 was missed totally. Aso-ohashi was a bridge which consisted of a trussed arch bridge with 3 spans continuous girder and simple girder bridges at the both ends. The large slope failure occurred at the side of the continuous girder bridge. It was pointed that the possible causes of this collapse were strong ground motions, heavy mudslide from the adjacent slope, failure and loss of the arch abutment foundation, and large fault displacement crossing the bridge, as well as the multiple complex effect of these causes.

This research was conducted to clarify the effects of these possible causes on the collapse of the bridge and to estimate the dynamic behavior of the collapse sequence. In this paper, the effects of the strong ground motions, the fault displacement crossing the bridge, and the forced displacement from the failed slopes, were investigated. Dynamic and static analyses were made using 3-D full bridge mathematical model to simulate the bridge behavior. This model consisted of mass, beam, truss and spring elements with consideration of material nonlinearity. Through the analyses shown below, the whole bridge behavior and the damage sequence of each member were investigated.

Dynamic time history response analyses were made using the input ground motion recorded at the nearest observation station from the bridge, “Otsu.” Three components of the ground motions were input to the longitudinal, transverse and vertical directions of the model. The response of the most impacted member was obtained as only 39% of the yield strength of the member. Thus, this indicated that the strong ground motions did not affect significantly on the bridge collapse.

Static push-over analyses were made by applying the forced displacement to the foundations for two cases. In the first case, the forced displacement was given to all foundations in the slope side, including arch abutment. In the other case, the displacement was given to the foundations except the arch abutment in the slope side. In both cases, the step by step static analyses were made by giving the displacement up to 2.0m. As the forced displacement was increased, the members at the springing sections of the arch bridge yielded at first and the damage was expanded around the springing sections. With the larger displacement, the damage spread more to whole the arch bridge. The damage mode of the arch bridge was significantly different between two cases.

From these results, the conclusions were made in the following. (1) It was estimated that the effect of the earthquake ground motions was not significant on the collapse of the Aso-ohashi. (2) When the displacement was imposed to the foundations in the slope side, the members of the springing sections of the arch bridge yielded at first and the damage spread from there. (3) Depending on the displacement condition of the arch abutment, the shape of deformation of the whole structure subjected to the forced displacement was significantly different.

Keywords: Kumamoto Earthquake, Aso-ohashi, damage analysis, ground motion, fault displacement, slope failure

1. Introduction

On April 16, 2016, a strong earthquake, of which hypocenter was at a depth of 12km and with JMA magnitude of 7.3, struck Kumamoto prefecture. This earthquake was supposed to be caused at the Futagawa fault zone and a fault slip up to 2.2m was observed. Twenty-eight hours before the main shock, an earthquake with JMA magnitude of 6.5 occurred at the adjacent Hinagu fault zone. Then, JMA seismic intensity of 7 was recorded twice at the same areas within two days. During the earthquake, many landslides occurred by strong ground motions at mountainous districts near the epicenter, particularly, at Tateno district in Minami-Aso Village. As shown in Fig.1, a large slope failure, with approximately 700m length, 200m width, and estimated gravel volume of 500,000 m^3, occurred at the west side of Aso-ohashi on National Highway route #325 which crossed the Kurokawa river [1], then Aso-ohashi collapsed destructively. Aso-ohashi was built in 1964 and with a bridge length of 205.96m, and consisted of a trussed arch bridge with 3 spans continuous steel girder and a simple steel girder bridges at the both ends (see Fig.2) [2]. Substructures of the continuous girder bridge section were inverted T-type abutments and piers, and ones of the arch bridge section were gravity-type abutments. Aso-ohashi was seismically retrofitted in 2015[3]. In this work, the springing parts of arch ribs and end columns were strengthened and fixed rigidly to the abutments by reinforced concrete jacketing.

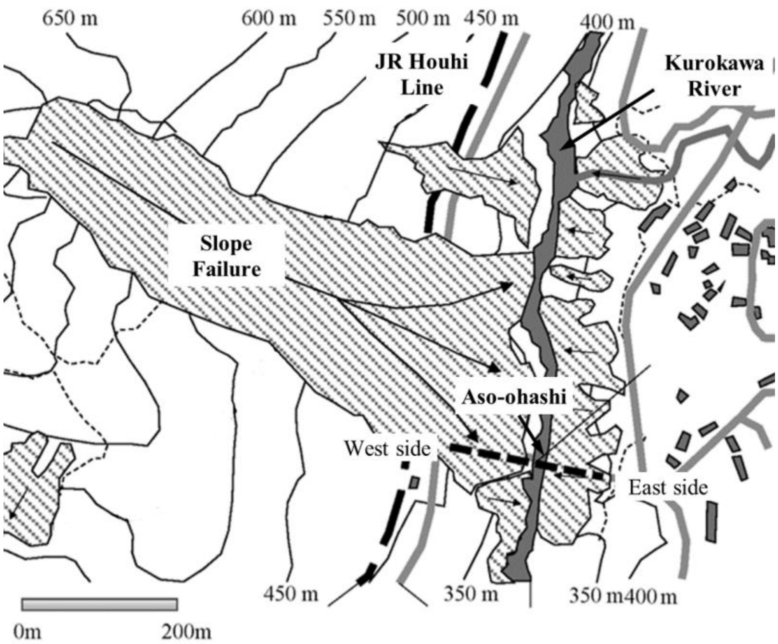


Fig. 1 – Slope failure and location of Aso-ohashi (modified from Ref. [1])

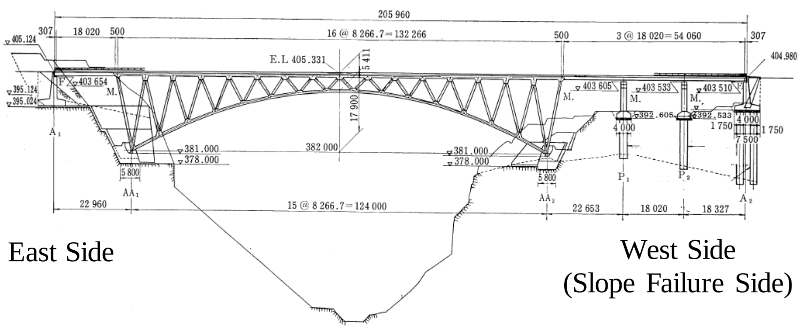


Fig. 2 – Side view of Aso-ohashi (modified from Ref. [2])

During the main shock of the Kumamoto earthquake, superstructures and piers of Aso-ohashi were collapsed and washed away with only its abutments at the both ends and arch abutments left. What was the main reason of this collapse? It was pointed that the possible causes were strong ground motions, heavy mudslide from the adjacent slope on the west side, failure and loss of the arch abutment foundation, and large fault displacement crossing the bridge, as well as the multiple complex effects of these causes. Since there exist several bridges with similar vulnerable conditions in Japan, to consider the possible countermeasures for them, it is important to investigate the damage mechanism and its progress for Aso-ohashi as a typical example of such damage.

Chida et al. and Hara et al. conducted the analytical study to estimate the damage of Aso-ohashi during the Kumamoto earthquake [4,5]. However, these studies focused on only the arch bridge section without any consideration of girder bridge sections at the both ends. Chida et al. measured the amount of generated ground displacement by aero survey. Accordingly, both side of grounds of Aso-ohashi were supposed to be displaced by 2.24m toward the direction to compress the bridge [4]. And, FEM analyses with regard to the effect of the mudflow on the bridge deck surface were conducted, and it was concluded that the possibility that this bridge collapsed by the mudflow on the deck was small.

Based on the above, the objective of this research is to estimate the progress of deformation of the Aso-ohashi to the critical conditions and the collapse mechanism by analyzing statically and dynamically against the strong ground motions and ground deformations. With modeling the whole bridge of Aso-ohashi, including a trussed arch bridge, continuous girder and simple girder sections, non-linear FEM analyses were conducted. The strong ground motions observed around the bridge site－at the observation station of “Otsu”, was used as an input ground motion, and the forced ground displacement was given to its abutments and pier foundations on the west side (slope failure side) toward the opposite side. The attribute of its deformation and possible members which initiated the collapse were studied.

2. Simulation Model of Aso-ohashi and Analytical Conditions

2.1 Simulation Model of Aso-ohashi

In this study, the whole of Aso-ohashi was modeled as a frame type structure as shown in Fig.3. For superstructures, decks were modeled as linear beams with lumped masses on nodes giving its weight as shown in Fig.4. Lumped masses were connected with main girder vertically by rigid members based on the JSCE Specifications of Steel Structures [6]. Major parts including main girders, arch members, diagonal members and cross beams, of which material was SM490, were modeled as non-linear beam members with bilinear hysteresis having its material property. Assuming that the lateral and sway bracing members resisted against only axial force, they were modeled as a truss member. Clearance of expansion joints between trussed arch bridge and girder bridge sections was ΔW= 0.150m as shown in Fig.5. For simulating the contact effect at the expansion joints, ‘Contact spring model’ was set in this gap. This model had a non-linear force-displacement property in the longitudinal direction as shown in Fig.6. The damping coefficient was assumed as 0.03 for piers and concrete decks, and 0.02 for steel members based on JRA Design Specifications for Highway Bridges [7]. For the numerical simulations, a structural analysis software TDAPIII was used.

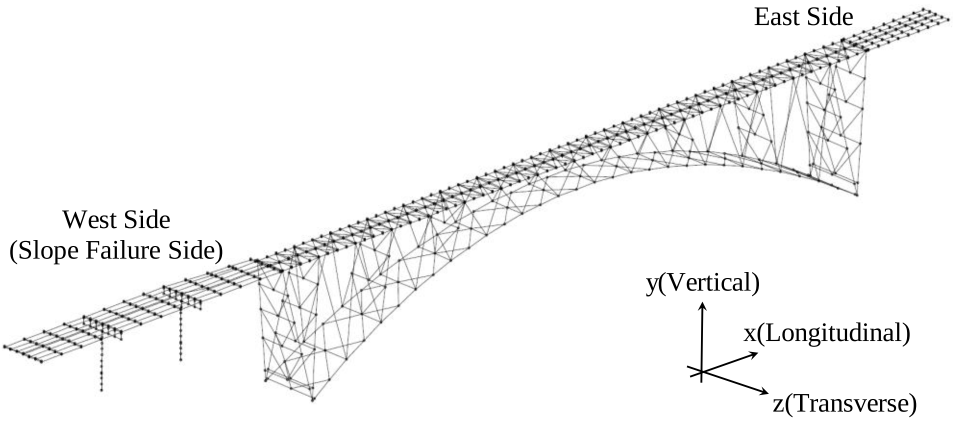


Fig. 3– Simulation model of Aso-ohashi

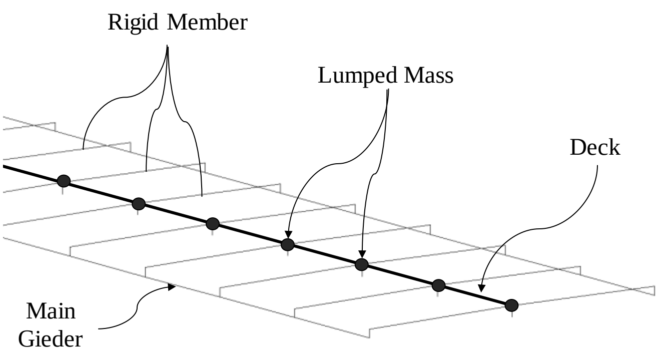


Fig. 4– Beam-mass model of superstructure

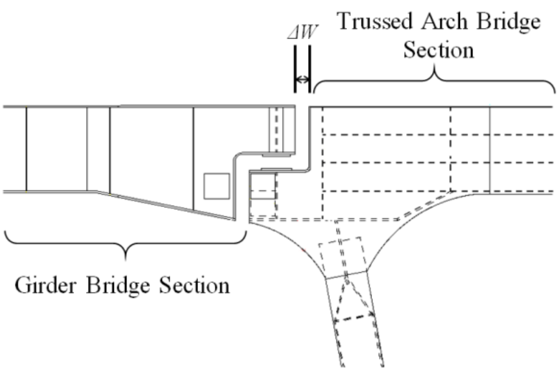


Fig. 5– Detail of joints between trussed arch (modified from Ref. [3])

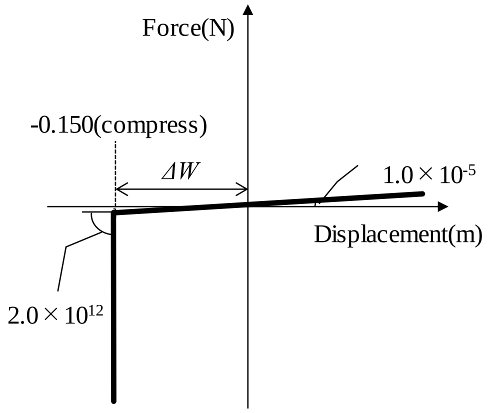


Fig. 6 – Force-Displacement relation to simulate the contact effect at expansion joints between trussed arch bridge and girder bridge sections

2.2 Analytical Conditions

In this study, two kinds of analyses, including dynamic time history analysis against the observed earthquake ground motions and static pushover analysis to the ground deformation, were conducted.

For the dynamic analysis, the time history data recorded at the nearest K-NET observation station, "KMM05 Otsu," was used as an input ground motion. The data was consisted of NS, EW and UD components and the data of main shock with 60 seconds was extracted from the original data for the analysis. Since the centerline of Aso-ohashi was rotated by 102.036° counterclockwise from the NS direction, horizontal NS and EW component data was converted to the longitudinal and transverse directions. The analyses were conducted for 4 cases with the input ground motions as longitudinal only, transverse only, vertical only, and 3 directions simultaneously.

For the pushover analysis, the forced ground displacement was given to the foundations. Based on the bridge site survey, the forced ground displacement was assumed to increase at the west side (slope failure side) abutments and foundations. The forced ground displacement was gradually increased by the increment of 0.001m up to 2.000m. It was reported in the aerial photo investigation [4] that both side of grounds of the bridge were displaced about 2m relatively toward the direction to compress the bridge. In this study, 2 cases of the pushover analyses in terms of the ground forced displacement was conducted. In Case 1 as shown in Fig.7(a), equivalent amount of displacement, x_g, was given simultaneously to all abutments and piers in the west slope failure side. This case simulated the case assuming that the fault crossed over the trussed arch bridge section acted during the earthquake. In Case 2 as shown in Fig.7(b), the abutment and piers of continuous gilder section in the west side were displaced simultaneously but assuming that the abutment of trussed arched bridge section (arch abutment) did not moved. Fig.8 shows the ground condition of the west side section. Since the arch abutment was founded at the andesite while the continuous gilder section was piled in the surface soil layer, it was assumed that this surface layer was displaced with the slope failure. According to the bridge site survey, it was confirmed that the arch abutments were left at the original location. In addition, as shown in Fig.9, the expansion joint between the simple girder and the abutment in the east side got into under the pavement on the backfill embankment. From these conditions, it was estimated that the girders contacted each other and moved into the east side.

The pushover analyses were conducted assuming that the initial condition was in the state with dead load condition, and the progress of deformation and damage of members with exceeding the yielding were studied.

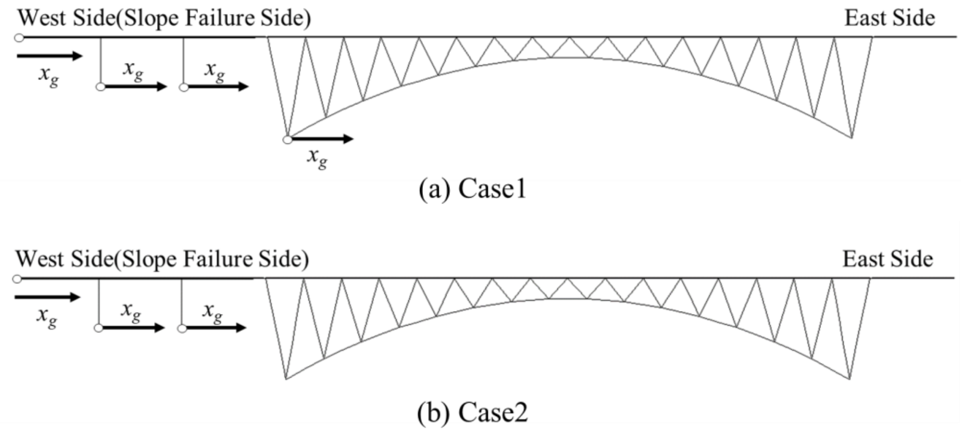


Fig. 7 – Assumed forced ground deformation for the analyses

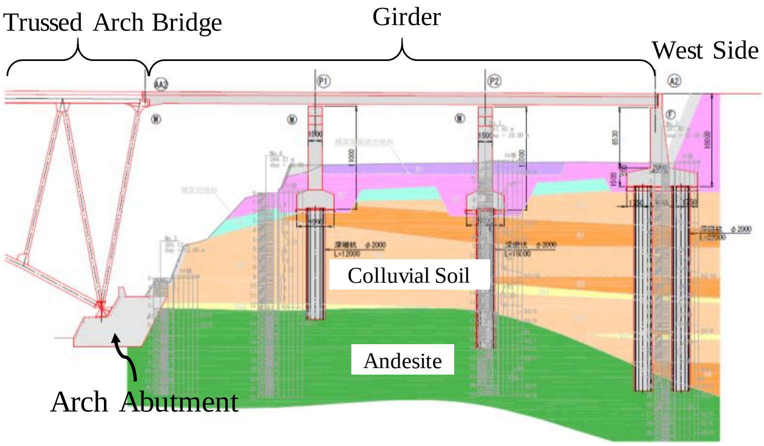


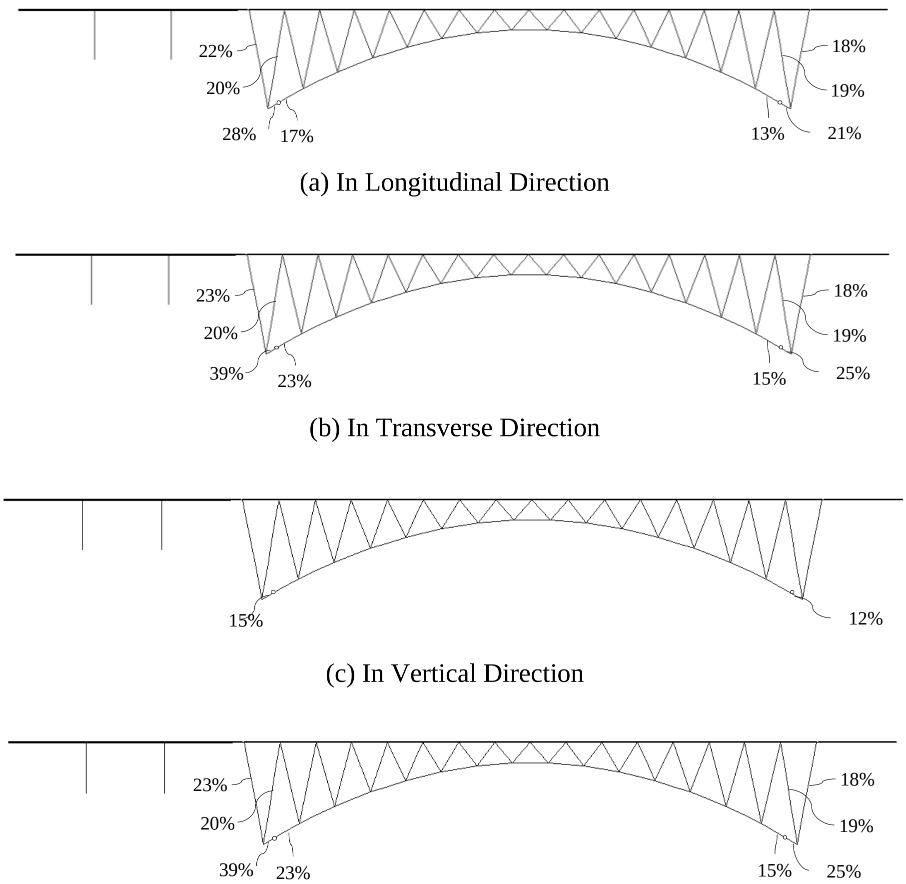
Fig. 8 – Ground condition at arch abutment and continuous girder section (modified from Ref. [3])

Fig. 9 – Collision or compression of explosion joint at east side abutment

3. Analysis Result

3.1 Dynamic Time History Analyses

Fig10 shows the maximum response curvatures depending on the directions of input ground motions. Ground motions were given as a single direction in the longitudinal, transverse, and vertical, as well as in 3 directions simultaneously. The rates of response curvature to the yield state of members which responded exceeding 10% are shown. In the case in which the ground motion was input in the longitudinal direction only, the maximum response was 28% at the springing sections of the arch ribs, and 22% in the end columns in the west side. In the case in which the ground motion was input in the transverse direction only, the responses indicated equal or larger totally than the former case, and the response at the springing sections became 39% in the west side. In the case in which the ground motion was input in the vertical direction only, not so much responses arose and only the members at the springing sections in the west side reached the rate over 15%. Based on these results shown in Fig.10 (a)-(c), the larger responses were observed at the springing sections but were not so much as the member failure. Since the responses of the case for the transverse direction and the case for 3 directions are almost the same on the whole, Aso-ohashi supposed to be most affected by the transverse ground motion. Since these analyses were made using observed earthquake records and there were no members of which curvature reached the yield state, the possibility that Aso-ohashi collapsed by only the strong ground motion seems to be low.



(a) In Longitudinal Direction

(b) In Transverse Direction

(c) In Vertical Direction

(d) In 3 Directions

Fig. 10 – Maximum response curvatures depending on the directions of input ground motions in which members with relatively large response curvatures are shown (equal or large than 10%)

3.2 Pushover Analyses

Table.1 shows the progress of the location of members of which the response curvatures exceeded the yield point with the increase of the forced ground displacement x_g for Case 1 and Case 2, in which the boundary condition for the arch abutment was different. As x_g was increased, it was seen that members which yielded spread out gradually.

For Case 1, the springing sections of the arch ribs in the west side yielded first of all at the forced ground displacement x_g = 0.155m. Subsequently, the same sections of the arch ribs in the east side yielded at x_g = 0.204m. It was estimated that the response curvatures in these sections were easy to increase by the forced ground displacement because the springing sections of the arch ribs and end columns were fixed rigidly to abutments. Fig.11 shows the relations between the deformation of the contact spring models on the west and east sides and the forced ground displacement x_g for the Cases 1 and 2, respectively. In Case 1, the deformation of both side of contact models reached 0.150m, which was the gap between trussed arch bridge and girder bridge sections (ΔW) and the deformation became constant around at x_g = 0.300m. At x_g = 0.370m, the end columns yielded. It was supposed that these responses of the end columns became larger because the continuous girders pushed the trussed arch bridge and simple girder bridge sections from the west to the east by fixing at the joints, and the arch ribs deformed upward. Fig.12(a) shows the locations of members which exceeded the yield point.

Table 1 – Locations of members of which response curvatures exceeded yield point with increase of forced ground displacement, x_g

(a) Case1

x_g (m)	Members and Sections which exceed Yield Point
0.155	Springing sections of arch ribs(West side)
0.204	Springing sections of arch ribs(East side)
0.370	Under sections of end columns(West side)
0.446	Under sections of end columns(East side)
0.731	Upper sections of end columns(West side)
1.048	Arch ribs of center and quarter of span
1.857	Main guilder of center of span

(b) Case2

x_g (m)	Members and Sections which exceed Yield Point
0.410	Springing sections of arch ribs(West side)
0.700	Springing sections of arch ribs(East side)
1.145	Lateral bracing of upper sections of end columns(West side)
1.280	Lateral bracing of under sections of end columns(West side)
1.401	Under sections of end columns(West side)

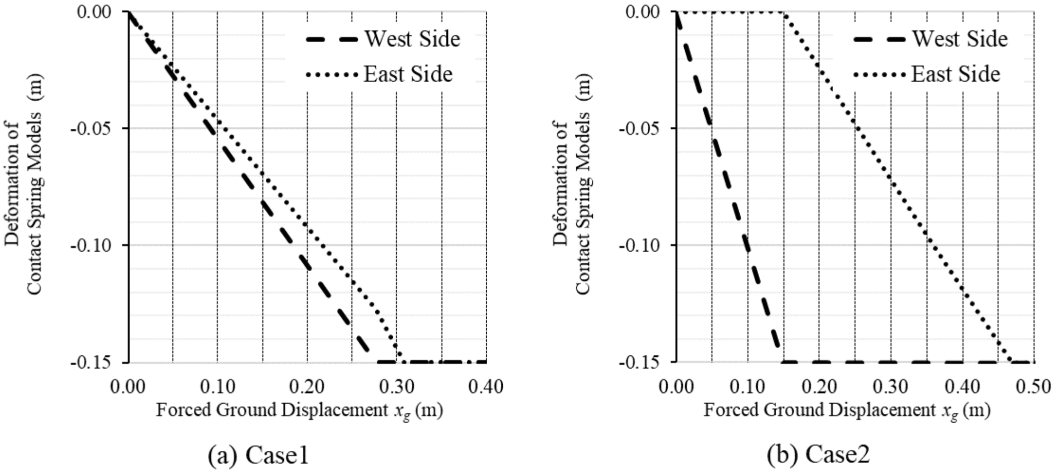


Fig. 11 – Collision or compression of explosion joint at east side abutment

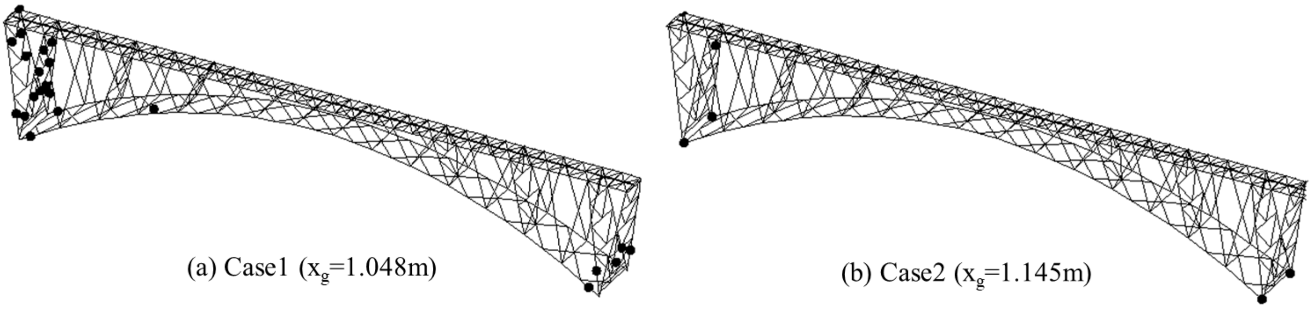


Fig. 12 – Locations of members of which response curvatures exceeded yield point

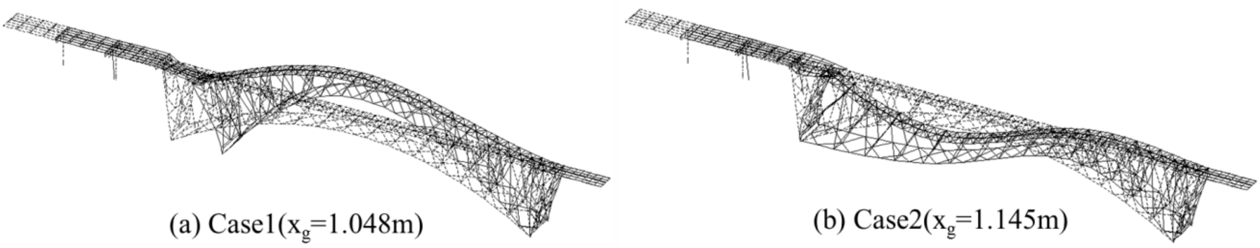


Fig. 13 – Deformation mode of whole bridge

For Case 2, the progress of the location of members of which the response curvatures exceeded the yield point is shown in Table.1(b). From Fig.11(b), it was confirmed that the contact spring model in the west side became rigid at x_g=0.150m and the trussed arch bridge started to deform after the contact. At x_g = 0.410m, the springing sections of the arch ribs in the west side yielded at first. Following this, the contact spring model in the east side also became rigid at x_g=0.460m and the superstructures became continuous in the longitudinal direction. When the displacement x_g reached 0.700m, the springing sections of the arch ribs in the east side yielded. Fig.12(b) shows the locations of members which exceeded the yield point.

Fig.13 shows the deformaton modes of whole bridge for two cases. The deformation mode of arch bridge section of Aso-ohashi was bending upward for Case 1, while that was like the shape of a character 'S' for Case 2. Based on these results, it was found that the modes of deformation of whole bridge were quite different depending on the boundary conditions between giving the forced ground displacement to the arch abutment on the west side (slope failure side) or not. In both cases, the springing sections of the arch ribs in the west side yielded firstly and the damage was spread from the end columns.

4. Conclusions

In this research, the causes of the collapse of Aso-ohashi during 2016 Kumamoto earthquake was studied through the numerical simulations using the dynamic time history analysis with the observed ground motions and the static pushover analysis with the forced ground deformation. Conclusions were obtained as follows.

(1) The results of the dynamic time history analysis considering the observed ground motion showed that no serious damage was estimated in the main parts of Aso-ohashi. The member of in which the curvature most responded was at the springing sections of the arch ribs in the west side, but it was only about 40% of the yield curvature. Therefore, it was estimated that the ground motion was unlikely to cause directly the collapse of Aso-ohashi.

(2) The results of the static analysis giving the forced ground displacement including a fault slip on the west side where the large slope failure occurred showed that the members around the springing sections of the arch ribs and end columns yielded firstly, especially in the west side. Hence, it was estimated that the damage that spread from the springing sections in the west side induced continuous guilder and trussed arch bridge to the collapse, then dragging the simple guilder bridge section, and consequently Aso-ohashi was missed totally.

(3) In the case in which the forced displacement was given to all abutments and piers in the west side, the deformation mode of the arch bridge section was bending upward overall. On the other hand, in the case in which the displacement was given to the abutments and pier foundations in the west side excluding the arch abutment, the deformation mode was like the shape of a character 'S.' The deformation mode of the arch bridge section of Aso-ohashi differed greatly depending on the boundary condition of the foundations.

5. References

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