NONLINEAR ANALYSES OF OGINOSAKA HORIZONTALLY CURVED BRIDGE IN THE 2016 KUMAMOTO EARTHQUAKE

S. Fujikura¹, T. Sasaki², H. Motohashi³ and T. Nonaka⁴

ABSTRACT

The 2016 Kumamoto Earthquake occurred in central Kyushu, Japan, on April 14th with Mw 6.2 followed by the Mw 7.0 mainshock on April 16th. These earthquakes were mainly caused by the Futagawa fault and Hinagu fault where surface ruptures extended about 34 km long. The earthquakes killed about 250 people and caused significant damage to buildings and infrastructure in Mashiki, Nishihara, and Minamiaso areas along these two faults. One of the important discoveries is the damage of relatively new bridges, designed by the bridge specifications after the 1995 Kobe earthquake, along Tawarayama bypass through Aso Mountains. The Oginosaka horizontally curved bridge was one of them and was examined by carrying out nonlinear analyses to evaluate the damage in the 2016 Kumamoto Earthquake using simulated near field ground motions.

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Introduction

The 2016 Kumamoto Earthquake occurred in central Kyushu, Japan, on April 14th with Mw 6.2 followed by the Mw 7.0 main-shock on April 16th. These earthquakes were mainly caused by the Futagawa fault and Hinagu fault where surface ruptures extended about 34 km long [1, 2]. The earthquakes killed 250 people and caused significant damage to buildings and infrastructure in Mashiki, Nishihara, and Minamiaso areas along these two faults. One of the important discoveries is the damage of relatively new bridges, designed by the bridge specifications after the 1995 Kobe earthquake, along Tawarayama bypass through Aso Mountains. The Oginosaka horizontally curved bridge was one of the damaged bridges along this bypass. Nonlinear analyses were carried out in order to evaluate the damage of the Oginosaka Bridge. A deck-abutment pounding interaction was considered in the analyses to investigate the post-impact

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response of the superstructure. The simulated near field ground motions were used for the analyses. The ground motions to be used for the analyses were simulated using the stiffness matrix method and kinematical model of fault rupture.

**Oginosaka Bridge and Damage**

The Oginosaka Bridge is one of the bridges damaged located in Tawarayama bypass. This bridge is a three-span continuous horizontally curved bridge with composite steel I-girders and has a longitudinal and transverse grade which is typical for bridges located in a mountainous area as shown in Fig. 1. Fig. 2(a) shows the cracks developed from the bottom of the abutment back-wall, which shows the evidence of the deck-abutment pounding in longitudinal direction. Fig. 2(b) presents the damage of the concrete block restrainer due to the collision of superstructure in transverse direction. Also due to the displacement of superstructure in transverse direction, there is a residual displacement of 150 mm in elastomeric bearing as shown in Fig. 2(b). There was almost no structural cracks in P1 and P2 columns. The superstructure was rotated in a plan view and displaced transversely at both abutments to the opposite side, whereas there was almost no obvious damage to the substructures.

![Figure 1. Side and plan view](image1.png)

![Figure 2. Damage of bridge: (a) Cracks at A1 back-wall; (b) Concrete block damage and bearing residual displacement](image2.png)
Simulation of Near Fault Ground Motions

The near fault ground motions having large permanent displacement due to a fault rupture were simulated by using the stiffness matrix method and the kinematic model of fault rupture in horizontally homogeneous layered half space [3, 4]. National Institute of Advanced Industrial Science and Technology extensively investigated the surface ruptures around the Futagawa fault and Hinagu fault [5]. Geospatial Information Authority of Japan examined the crustal deformation using InSAR and GNSS and the cracks of earth surface based on aerial photos [6]. Also, National Research Institute for Earth Science and Disaster Resilience and Koketsu et al. studied the process of seismic source rupture by the inverse seismic source using strong-motion earthquake records near the source [7, 8]. Based on these studies, the source parameters were decided as shown in Table 1 and used for the simulation. Fig. 3 presents the seismic source faults assumed in the simulation. The recorded ground motions close to these faults were used in order to compare with the simulated near fault ground motions. Fig. 3 shows the location of these recorded ground motions used for the comparison; KiK-net Mashiki, K-NET Ootsu and Nishihara-village Komori (JMA) along with the location of Oginosaka Bridge.

Table 1. Source parameters used in the simulation

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<tr>
<th>Fault type</th>
<th>SMGA1</th>
<th>SMGA2</th>
<th>SMGA3</th>
<th>SMGA4</th>
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<td>Fault width W_a (km)</td>
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<td>Fault area (km^2)</td>
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<td>Slip (m)</td>
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<td>Rise time (s)</td>
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<td>Seismic moment M_o (N·m)</td>
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<td>Moment magnitude M_w</td>
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<td>6.8</td>
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</table>

Figure 3. Seismic source faults assumed in the simulation
Figs. 4 and 5 respectively show the ground acceleration and displacement from the simulation for A1 abutment and P1 column of Oginosaka Bridge for example. From Fig. 5, the residual displacement of A1 abutment is about +77mm, +122mm and −77mm in NS, EW and UD directions, respectively. Also the residual displacement of P1 abutment is about +75mm, +119mm and −75mm in NS, EW and UD directions, respectively. Fig. 6 presents the acceleration response spectrum for A1, P1, P2 and A2 from this simulation.

Figure 4. Simulated ground acceleration

Figure 5. Simulated ground displacement

Figure 6. Simulated acceleration response spectrum
Fiber Element Analysis

Nonlinear time-history analyses were carried out in order to verify the damage observed in the field survey. A deck-abutment pounding interaction was considered in the analyses to investigate the post-impact response of the superstructure. The simulated near field ground motions were used for the analyses.

Analytical Model

Fig. 7 shows analytical model of Oginosaka bridge. Because this bridge is a curved bridge with slope in both longitudinal and transverse directions, the response in longitudinal and transverse directions should be coupled. Thus, the target bridge is idealized as 3D frame model as shown in Fig. 7.

![Analytical model](image)

Based on the damage inspections described above, the damage of the plastic hinges at the column base was limited. Therefore, only the plastic hinges at the column base are modeled using nonlinear fiber elements and the column outside of the plastic hinges are idealized as elastic beam elements. Fig. 8 shows constitutive models of concrete and reinforcement used in the fiber elements. Combination of an envelope curve proposed by Hoshikuma et al. [9] and unloading and reloading paths proposed by Sakai et al. [10] are used for cover and core concrete. Menegotto-and-Pinto model [11] modified by Sakai et al. to prevent the stress overestimation due to small unloading and reloading [12] is used for reinforcement. Concrete strength is 24 MPa and yield strength of longitudinal and tie reinforcements is 345 MPa.

Seven nodes for the deck are provided at the top of columns and abutments, and middle points between these points. The deck is idealized using elastic beam elements. The section of the deck is not uniform in the actual bridge, however for simplicity, the section of the deck is assumed to be uniform section. The five elastomeric bearings are modeled at each column and abutment. These bearings are idealized as linear spring elements.
The mass and stiffness proportional damping (Rayleigh damping) is assumed. The parameters of Rayleigh damping is evaluated based on the modal damping of predominant modes in longitudinal and vertical directions; 1st and 4th mode, respectively. To evaluate the pounding effect on the rotational response of the deck in a plan view, two different boundary conditions are considered. Case A is the model without poundings. There is no interaction between the deck ends and abutments. Case B is the model with poundings. Pounding springs are installed between the deck ends and abutments in longitudinal and transverse directions.

In longitudinal direction, the deck end directly pounds to the abutment if the deck displacement relative to the abutment exceeds its clearance. Thus, stiffness of the pounding spring becomes stiff enough after its relative displacement exceeds its clearance. Stiffness of the pounding springs are modeled based on the following [13]:

$$K_p = \begin{cases} 0 & (u < u_c) \\ \frac{EA_{Deck}}{L} & (u \geq u_c) \end{cases}$$  \hspace{1cm} (1)

where $EA_{Deck}$ is the axial stiffness of the deck, $L$ is the length of the deck, $n$ is the number of divisions and $u_c$ is clearance between the deck end and abutment. In transverse direction, the concrete restriction blocks are provided between the deck and abutment and rubber pads is installed as the impact absorbing devices. Therefore, the poly-linear elastic model, which can represent the force-displacement response of the rubber pads proposed by Uruta et al. [14], is used for pounding springs in transverse direction. Note that the stiffness in unloading paths is generally stiff and the rubber pad has small energy dissipation, however, it is neglected in this analysis.

**Analytical Results**

Fig. 9 shows displacement response at the top of P1 and P2 columns and acceleration response at the top of A1 abutment. As shown in Fig. 9(a) and (b), the displacement response at the column top in Case A is close to the one in Case B. On the other hand, the acceleration response at the deck end above A1 abutment shown in Fig. 9(c) significantly increases due to poundings. The acceleration response in Case A is 2.7 and that is 3.5 times larger than the one in Case B.
Figure 9. Time history of displacement and acceleration: (a) displacement response at the Top of P1 column; (b) displacement response at the top of P2 column; (c) acceleration response at the deck end above A1 abutment.

Fig. 10 shows moment vs. curvature hysteresis at the base of columns P1 and P2. The capacity of P1 and P2 columns evaluated by Japanese seismic design specification [15] is also shown in Fig. 10 for comparison. As shown in Fig. 10, both P1 and P2 columns are slightly yielded but do not suffer significant damage, which is consistent with the field investigation of P1 and P2 columns.

Fig. 11 shows the impact force of the pounding spring between the deck and A1 abutment. Note that G1 and G5 is the west end and east end girders, respectively. In Fig. 11, the strength of parapet walls of A1 abutment and concrete restriction blocks based on the original design is also presented for comparison. The deck pounds twice in longitudinal direction and twice in transverse direction. First the deck pounds at G1 girder and the impact force reaches 5.5 times of dead weight of the deck. This large impact force is more than twice of the strength of parapet
walls. This means that parapet wall suffers damage due to pounding, which is similar to the
damage observed after the earthquake as shown in Fig. 2(a). In transverse direction, the impact
force is 1.8 times of the dead weight of the deck, which is smaller than the one in longitudinal
direction. However, the strength of the concrete restriction blocks is 0.6 MN so that the impact
force is about 6 times of the strength. Therefore, it is clear that the restriction blocks suffered
significant damage. This damage corresponds to the damage observed after the earthquake as
shown in Fig. 2(b).

Figure 10. Moment vs. curvature hysteresis

Figure 11. Impact force (Top: Longitudinal Direction, Bottom: Transverse Direction)

Fig. 12 shows deck rotation in a plan view at the deck end above A1 abutment. The deck
rotation $\theta$ is evaluated as follow:

$$\theta = \frac{u_{G5} - u_{G1}}{B_{Deck}}$$

(2)

where $u_{G1}$ and $u_{G5}$ are response displacement at G1 and G5 girders relative to A1 abutment, respectively, and $B_{Deck}$ is distance between west and east end girders. As shown in Figure 17, the peak deck rotation is 0.65 degree in Case A while it increases to 0.83 in Case B due to pounding.

Figure 12. Deck Rotation due to Pounding

Conclusions

This paper presents the field investigation of the damage of the Oginosaka Bridge during the 2016 Kumamoto Earthquake occurred in central Kyushu, Japan. The superstructure was rotated in a plan view and displaced transversely at both abutments to the opposite side, whereas there was almost no obvious damage to the substructures. Nonlinear analyses were carried out in order to evaluate the damage of the bridge. The simulated near field ground motions were used for the analyses and the ground motions were simulated using the stiffness matrix method and kinematical model of fault rupture. A deck-abutment pounding interaction considered in the analyses could capture the post-impact response of the superstructure observed in the field survey.

Acknowledgments

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