Journal of Structural Engineering Vol.53A (March 2007) JSCE

# Seismic Response of 3-Span Bridge Considering the Effect of Failure of Bearings

Takashi Matsumoto*, Kazuhiko Kawashima**, Gakuho Watanabe***

* Graduate Student, Dept. of Civil Eng., Tokyo Institute of Technology, O-Okayama, Meguro, Tokyo 152-8552
** Dr. of Eng., Professor, Dept. of Civil Eng., Tokyo Institute of Technology, O-Okayama, Meguro, Tokyo 152-8552
** *Dr. of Eng., Assistant Professor, Dept. of Civil Eng., Tokyo Institute of Technology, O-Okayama, Meguro, Tokyo 152-8552

Although bearings and restrainers have been included in seismic response analysis of bridges, effect of failure of those structural components has not yet been fully analyzed. This paper presents an analysis of a 3-span simply supported bridge which is supported by elastomeric bearings. Steel plate type restrainers or PC strand restrainers are accommodated between decks. It is shown from the analysis that bearings fail progressively from a bearing located at one of the extreme edges due to rotation of decks. It is also shown that design concept to evaluate demand of bearings and unseating prevention devices by simply dividing the total lateral force by number of devices tends to underestimate real demand.

*Key Words: seismic design, bridges, restrainer, bearing, progressive failure*

## 1. Introduction

Based on extensive damage of bridges in 1995 Kobe, Japan earthquake, seismic design practice for bridges was enhanced after the earthquake. A large number of bridges collapsed due to insufficient shear and ductility capacity of piers. Steel bearings with insufficient strength and limited movable displacement capacity failed extensively, and this resulted in collapse of bridges or progress of damage. Unseating prevention devices were not adequately strong and ductile enough to prevent collapse of bridges. Consequently, extensive enhancement of seismic design was included in the design codes after the Kobe earthquake[1)2)].

One of the major changes in design practice after 1995 Kobe earthquake is extensive use of elastomeric bearings. Because seismic force builds up until failure in steel bearings, elastomeric bearings which allow relative displacement to take place between the superstructure and substructures are superior to steel bearings to mitigate the build up of seismic force. It is known that steel bearings were vulnerable to seismic force with shock, and they suffered damage in past earthquakes. However it was always an argument in past earthquakes whether damage of steel bearings mitigated for transferring excessive seismic force from the superstructure to substructures. But extensive damage of bridges during 1995 Kobe earthquake clearly showed the importance of preventing failure of bearings.

As a consequence, elastomeric bearings including lead rubber bearings and high damping rubber bearings started to be used in bridges after 1995 Kobe earthquake. Steel bearings were mostly used with exceptional use of rubber pads before the Kobe earthquake however elastomeric bearings are extensively used after 1995 Kobe earthquake. It is expected that the change of practice contributes to mitigate seismic damage resulted from failure of bearings in future earthquakes.

However, response displacement of superstructure relative to the substructures increase in a bridge supported by elastomeric bearings inherent to lower lateral stiffness of elastomeric bearings. Even in a standard viaduct supported by 10 m tall piers, deck displacement relative to substructures becomes over 0.3 m under near-field ground motions. Because gap between adjacent decks is generally 100-200 mm, large deck displacement can result in damage of expansion joints and poundings between decks. Based on past experience, poundings resulted in limited damage at faces of the deck where poundings occurred. However poundings transfer a large lateral force from one deck to the other, and it subsequently results in failure of bearings or collapse of the decks one another. Therefore analysis of bridge response under significant ground motions is extremely important by taking account of nonlinear interaction of superstructures, substructures, bearings, expansion joints and unseating prevention devices.

It should be noted that inelastic behavior and failure of piers, bearings, expansion joints and unseating prevention devices have to be properly included in analysis to reasonably estimate the complex

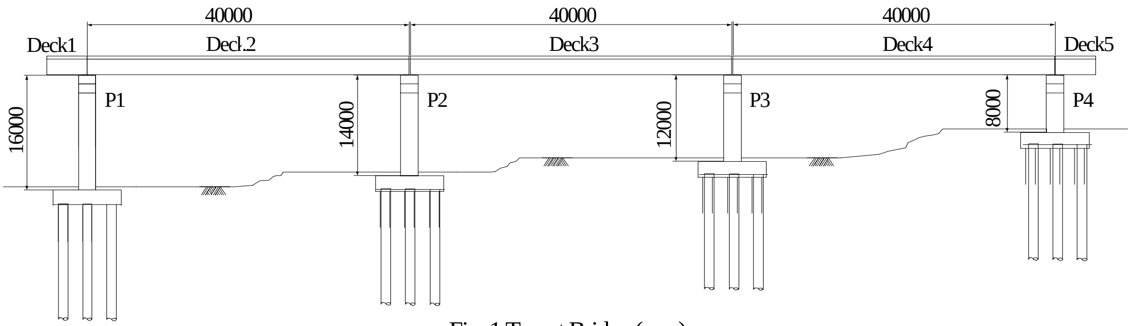


Fig. 1 Target Bridge (mm)

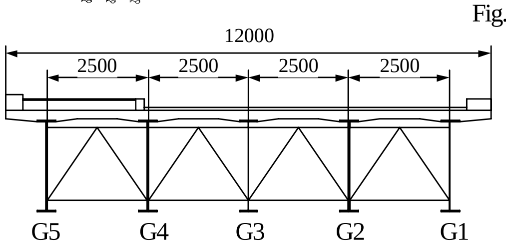


Fig. 2 Superstructure (mm)

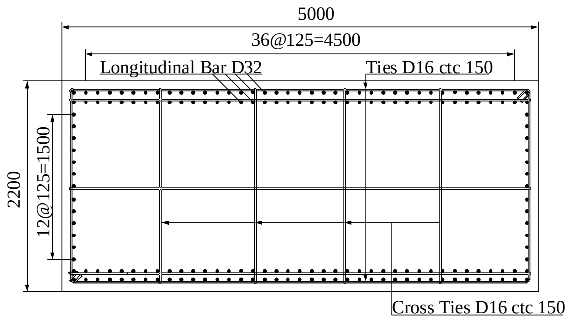


Fig. 3 Reinforcement of piers (mm)

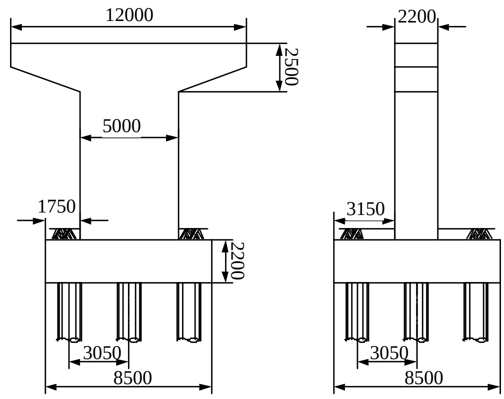


Fig. 4 Pier section (mm)

response of a bridge to failure.

It is also noted that there exist progressive failure modes among damage of main structural components under extreme ground motions. Damage of a bearing or a restrainer can result in an increase of lateral force induced in the rest of bearings and restrainers, and this result in damage of the reset of bearings and restrainers. It should be noted that fracture of bearings result in failure of restrainers. Therefore failure interaction among bearings, restrainers and columns is important. Although there are studies which show complex nonlinear behavior of bridges with poundings and restrainers[3,4,5], this progressive failure has not yet been investigated.

This paper presents an analysis of progressive failure of elastomeric bearings and restrainers on a 3-span simply supported bridge.

## 2. Target Structure

A 3-span simply supported steel I-girder bridge as shown in Fig. 1 is analyzed here. The deck is consisted of concrete slab and 5 steel girders (G1-G5). Each deck is 40-meter long and the gap between decks is 100 mm. Decks with a weight of 6.53 MN each are supported by 8-16 m tall T-shaped cantilevered RC piers. The bridge was designed based on 2002 JRA design code[1].

Cross sections of the superstructure, arrangement of reinforcement in piers, and pier section are shown in Figs. 2, 3 and 4, respectively. Deformed longitudinal bars with the nominal strength of 295 MPa (SD295) are provided. SD295 tie bars with 16 mm diameter are provided at every 150 mm interval. The longitudinal reinforcement ratio is 0.657 % and the volumetric tie reinforcement ratio is 0.53 %. The design concrete strength is 30 MPa.

Soil condition is Type II (moderate) based on the JRA design code[1]. The piers are supported by pile foundations.

Decks are supported by elastomeric bearings consisted of natural rubber, with shear modulus of 1MPa. Elastomeric bearings are 96 mm tall and 440 mm wide and long, respectively. They are designed assuming that the ultimate shear strain of rubber is 250 %.

Two types of restrainers are accommodated at each girder between Decks 2 and 3, and Decks 3 and 4. One is steel plate type restrainers without and with gap between bolts and holes, and the other is PC strand type restrainers.

## 3. Idealization of Bridge and Analytical Conditions

Decks are idealized by 3-D linear beam elements as shown in Fig. 5. Effect of Decks 1 and 5 was approximately taken into account in analysis by lumping a half of the deck mass of Decks 1 and 5 at the top of Piers 1 and 4, respectively. The strut action of slab is idealized by grids of the elements.

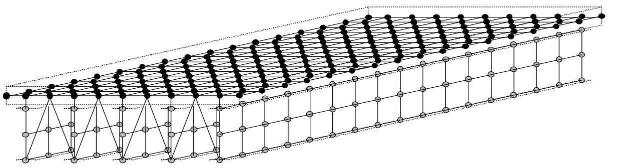


Fig. 5 The model of the deck

The piers are idealized by fiber elements at their plastic hinge regions. Hoshikuma model[6] and Sakai and Kawashima model[7] are used for representing lateral confinement of concrete including unloading and reloading paths. Menegotto and Pinto model is used to represent hysteric behavior of reinforcements[8]. Post yield stiffness of reinforcement is set 1 % of its elastic stiffness. The piers other than the plastic hinge are idealized by elastic beam element with the yield stiffness.

Poundings which occur between adjacent decks are idealized by impact springs as shown in Fig. 6 (a). Stiffness of impact springs is determined in terms of $\gamma$ defined by Eq. (1) based on axial stiffness of the decks as[9]

$$\gamma = \frac{k_I L}{nEA} \tag{1}$$

where, $k_I$: impact spring element, $L$: length of deck, $EA$: axial stiffness of the deck, and $n$: number of beam elements for a deck. Here $\gamma$ is assumed to be 5 in Eq. (1).

Fig. 6 (b) and (c) shows idealization of hystereses of restrainers without and with gap, respectively. Based on JRA design code (JRA 2002), strength demand of a restrainer $F_y$ is evaluated as

$$F_y = k_h \frac{\sigma_y}{\sigma_a} \frac{R_d}{n} \tag{2}$$

where $\sigma_y$ and $\sigma_a$: yielded strength and allowable stress of steel plate and $\sigma_y/\sigma_a$ is set 1.13, $k_h$: the seismic coefficient equal to 1.5 according to JRA code, $R_d$: reaction force due to dead weight of the superstructure, and $n$: number of restrainers. Strength of steel plate restrainer is 1.11 MN for both tension and compression. Due to lack of information about the deformation capability of steel restrainers, it is assumed that restrainers fail in either tension or compression when relative displacement over gap exceeds +/-1 mm. After failure restrainers do not resist deck opening and closure. The restrainer gap is set +/-20 mm.

Hysteretic behavior of PC strand restrainers is idealized as shown in Fig. 6 (d). Tension strength of a PC restrainer is 0.574 MN and deformation capacity is 16.5 mm, respectively. It consists by a PC strand with a diameter of 26 mm. Movable gap of PC restrainer is assumed as 50 mm.

Rupture of elastomeric bearings is taken into account in analysis by analytical model shown in Fig. 6 (e)[8]. Lateral force vs. lateral displacement hystersis is linear until bearings rupture, however the restoring force becomes zero once shear strain induced in rubber reaches 250 %. Although there must be interaction between failure in the longitudinal and transverse directions, rupture of the bearings associated with the longitudinal and transverse oscillations is assumed to be independent. Shear strength of the bearing is 0.566 MN.

Once an elastomeric bearing suffers extensive damage, there is a possibility that a part of failed bearing locks with the rest of bearing so that relative displacement between the deck and substructures is restricted[10]. There can be several failure modes of a bearing and related devices which result in "lock." Rough surface of ruptured bearing result in large restriction for bearing movement. Steel plates which are pealed out from failed surfaces could prevent bearing

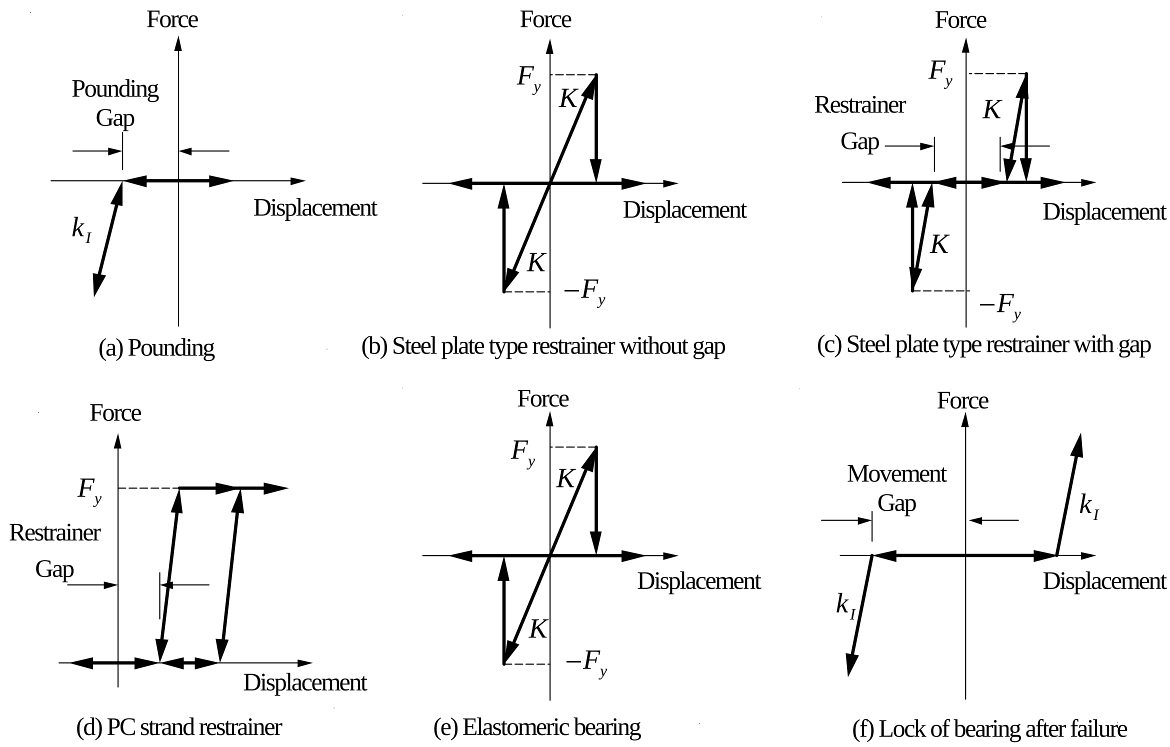
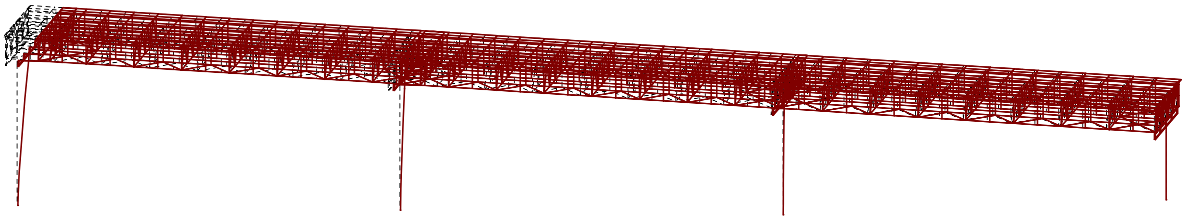
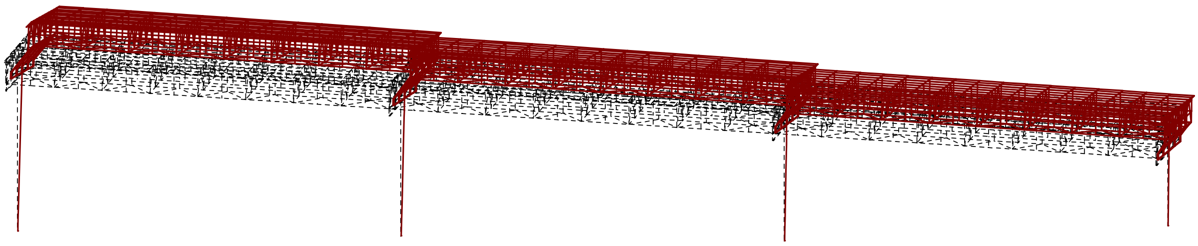



Fig. 6 Idealization of pounding, elastomeric bearing, steel plate restrainers, PC strand restrainers and lock of a bearing after failure

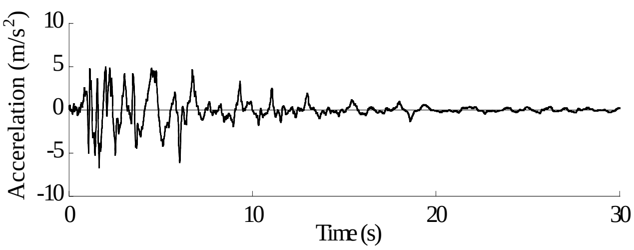


(a) 1st mode (longitudinal translation, T=1.33 sec)

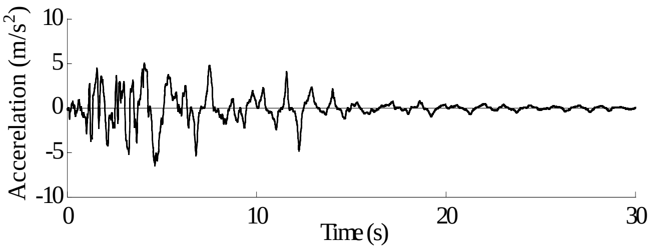


(b) 2nd mode (transverse translation, T=1.19 sec)

Fig.7 1st and 2nd mode shapes




(a) NS Component




(b) EW Component

Fig. 8 JR Takatori station accelerations recorded during 1995 Kobe earthquake

movement. Lock of bearings could result in collapse of bridges because excessive inertia force is transferred from a deck to substructures. Lock of an elastomeric bearing is idealized as shown in Fig. 6 (f), in which gap is assumed as 50 mm here.

When structural components are in the elastic, the 1st and 2nd natural periods of the bridge are 1.33 and 1.19 second, respectively. The first mode is predominant in the longitudinal direction due to deformation of elastomeric bearings, while the second mode is predominant in the transverse direction as shown in Fig. 7.

Damping ratio of the deck, piers and foundations is assumed as 0.02, 0.05 and 0.2, respectively. Fault normal and parallel components of JR Takatori station record (refer to Fig. 8) during the 1995 Kobe, Japan earthquake are imposed to the bridge in the longitudinal and the transverse directions, respectively.

## 4. Seismic Response of the Bridge without Restrainers

Fig. 9 shows deck responses in the longitudinal direction, when restrainers are not provided. All bearings failed, and displacement of Decks 2 and 3 is excessively large. Residual displacements reached nearly 0.33 m and 1.23 m at Decks 2 and 3, respectively, at the end of excitation. As shown in Fig. 10, bearings which support Deck 3 on P2 failed between 4.345-4.369 s. It is interesting to note here that among five bearings on P2 the bearing which supported G1 girder (designated hereinafter as G1 bearing) of Deck 3 failed first at 4.345 s followed by G2, G3 and G4 bearings, and G5 bearing finally failed at 4.369 s.

Response displacement of Deck 2 at both P1 and P2 sides in the transverse direction is shown in Fig. 11. Due to failure of bearings, residual displacement of the deck in the transverse direction reached 1.02 m and 0.76 m at P2 and P1 side, respectively. This resulted in rotation of Deck 2 as shown in Fig. 12. The rotation of Deck 2 becomes significantly large after 2.40 s because five bearings each which support Deck 2 at both P1 and P2 sides failed at 2.43 s as shown in Fig. 13.

Fig. 14 shows lateral force vs. lateral displacement hysteresis of P2 at the plastic hinge in the longitudinal direction. Displacement ductility factor is 3.1. After bearings on P2 failed in the longitudinal direction during 4.345 s (G1 bearing of Deck 3) and 4.442 s (G1 bearing of Deck 2), the inertia force of Decks 2 and 3 was not

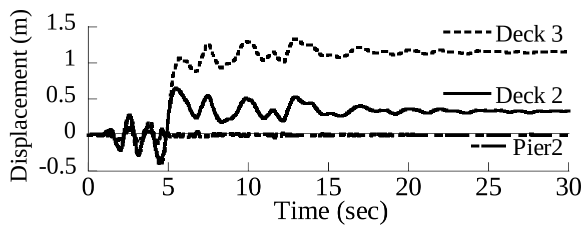
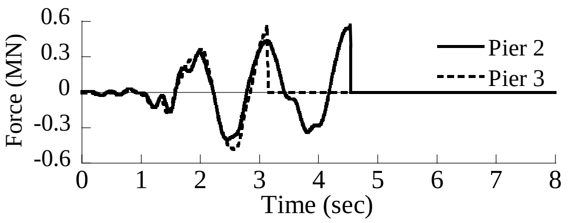
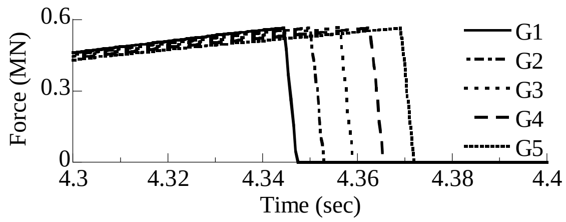



Fig. 9 Deck response in the longitudinal direction
(restrainers are not accommodated)




(a) Restoring force of G3 bearing on Pier 2




(b) Restoring force of G1-G5 bearings during 4.345-4.369 s (Deck 3 on Pier 2)

Fig. 10 Progressive failure of bearings which support Deck 3 on Pier 2 (restrainers are not accommodated)

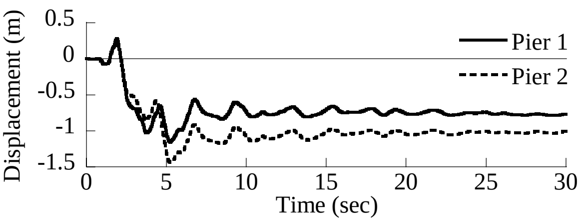



Fig. 11 Displacement of Deck 2 in the transverse direction (restrainers are not accommodated)

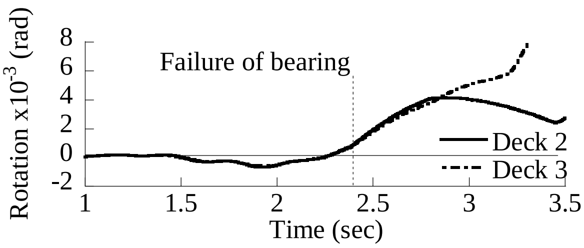



Fig.12 Rotation of Decks 2 and 3 (restrainers are not accommodated)

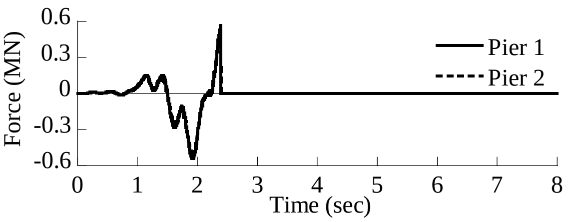



Fig. 13 Restoring force of G3 bearing (Deck 2) in the transverse direction (restrainers are not accommodated)

transferred to P2, which resulted in limited inelastic hysteresis of P2.

Fig. 15 shows pounding forces which were developed between Decks 2 and 3. Pounding occurred during 3-5 s at the extreme edges

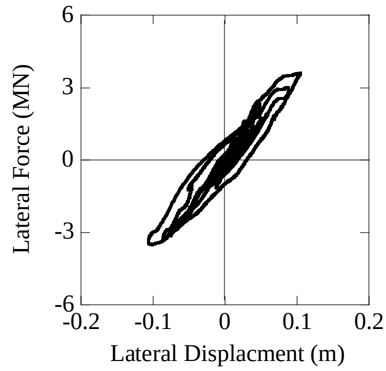



Fig. 14 Lateral force vs. lateral displacement hysteresis of Pier 2 (restrainers are not accommodated)

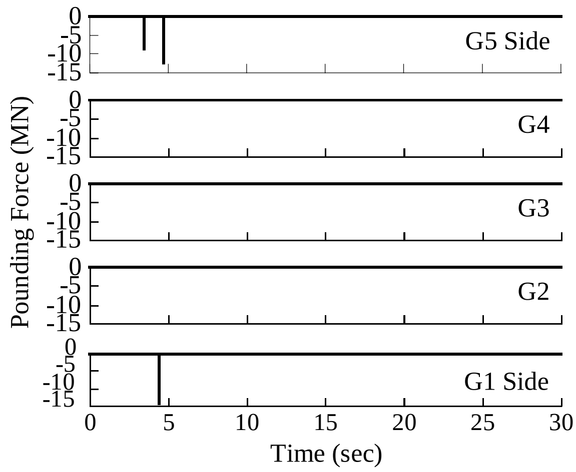



Fig. 15 Pounding force between Decks 2 and 3 (restrainers are not accommodated)

near G1 and G5 and this resulted in high pulse accelerations in the deck. It is important to note that pounding did not occur between decks at G2, G3 and G4. This is resulted by combined rotation around the vertical axis and translation of decks. This shows that protection of decks is required at the extreme edges. The peak pounding force at the extreme edge of decks near G1 reached 14.2 MN which is 2.2 times a deck weight.

## 5 Effect of Steel Plate Type Restrainers

Fig. 16 shows longitudinal response displacements of Decks 2 and 3 and P2 when steel plate type restrainers without gap are accommodated. Similar to the response of the bridge without

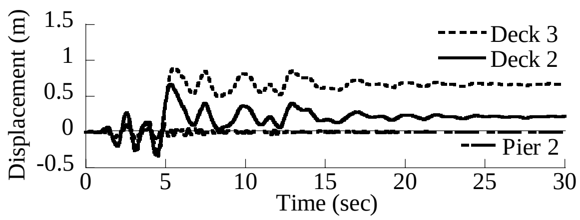



Fig. 16 Deck response in the longitudinal direction (steel plate restrainers without gap are accommodated)

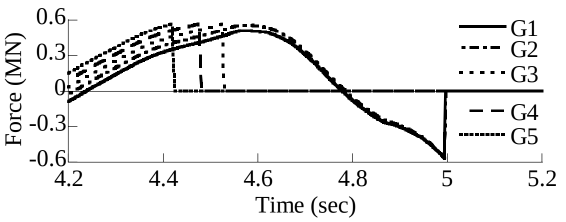



Fig. 17 Restoring force of bearing at Deck 3 on Pier 2 in the longitudinal direction (steel plate restrainers without gap are accommodated)

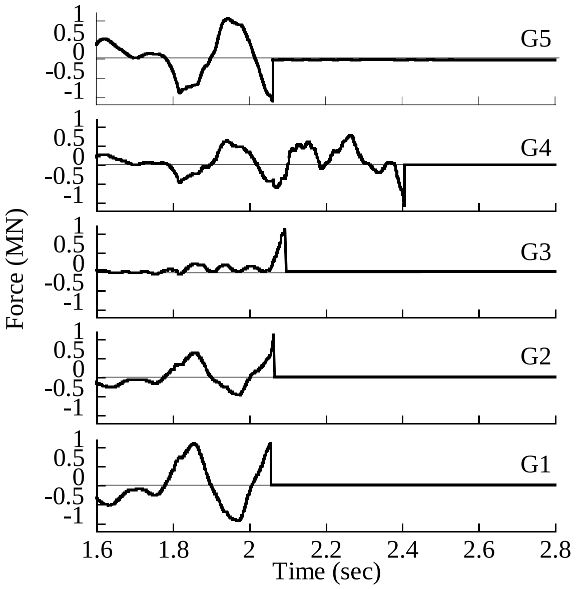



Fig. 18 Restoring force of steel plate restrainers between Decks 2 and 3 (steel plate restrainers without gap are accommodated)

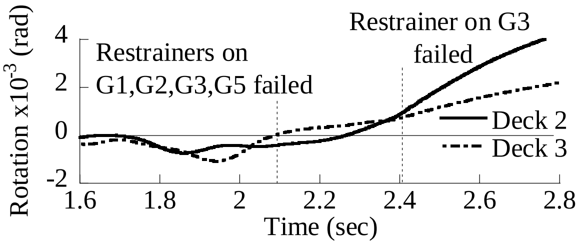



Fig. 19 Rotation of the Decks 2 and 3 when steel plate restrainers are accommodated (steel plate restrainers without gap are accommodated)

restrainers, the response displacements of Deck 3 becomes extensively large with a permanent residual displacement of 0.69 m. As will be described later, since restrainers failed at the early stage of excitation (during 2.056 and 2.407 s), the overall response of the decks is similar to that of the bridge without restrainers.

Fig. 17 shows how bearings which support Deck 3 on P2 in the longitudinal direction failed. Failure was initiated at G5 bearing at 4.418 s, followed by G4 bearing at 4.475 s and G3 bearing at 4.524 s. Then, G1 bearing failed at 4.994 s followed by failure of G2 bearing at 4.995 s. This is resulted from combined rotation and translation of decks.

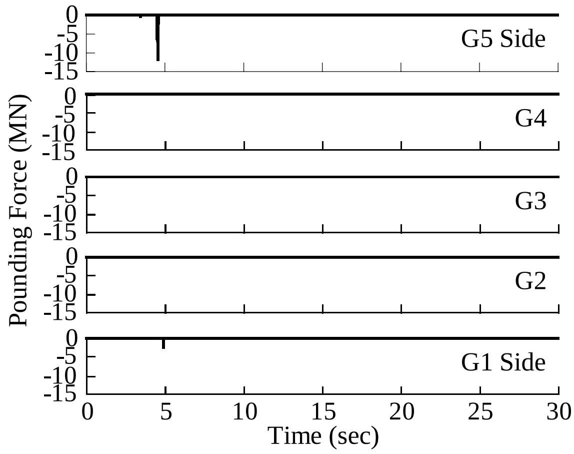



Fig. 20 Pounding force between Decks 2 and 3 (steel plate restrainers without gap are accommodated)

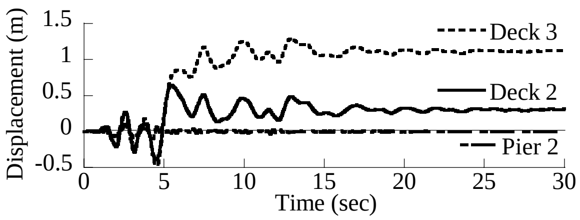



Fig. 21 Deck response displacement in the longitudinal direction (steel plate restrainers with gap are accommodated)

Restrainers ruptured earlier than the failure of bearings. As shown from restoring force of the restrainers between Decks 2 and 3 shown in Fig. 18, restrainers progressively failed from the restrainers accommodated at G1 girder (designated hereinafter as G1 restrainer), i.e., G1 restrainer fails first at 2.056 s and subsequently G5, G2, G3 and G4 restrainer fail at 2.062, 2.065, 2.095 and 2.407 s, respectively. It should be noted here that G1 restrainer ruptured in tension while G5 restrainer ruptured in compression. Because there was not gap at restrainers, small deck rotation (refer to Fig. 19) as well as translation resulted in rupture of a restrainer located near an extreme edge in tension and rupture of restrainer located near the other extreme edge in compression.

Pounding occurred once between Decks 2 and 3 at the extreme edge of decks near G1 and G5 as shown in Fig. 20. The maximum pounding force at extreme edge near G1 is 11.8 MN, which is 83 % the maximum pounding force developed between Decks 2 and 3 when restrainers are not accommodated.

Fig. 21 shows deck response displacements when there is a +/-20 mm gap at the restrainers. All bearings failed in this case, too, and response displacement of Deck 3 becomes extremely large with a residual displacement of 1.13 m. Fig. 22 shows how restrainers between Decks 2 and 3 failed sequentially. G1 restrainer failed first at 1.986 s, followed by G2, G3, G4 and G5 restrainers at 1.989, 1.991, 1.994 and 1.997 s, respectively. Although deck rotations are

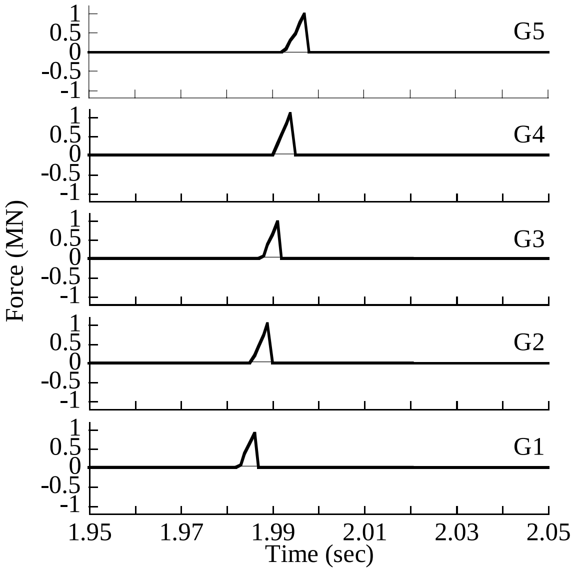



Fig. 22 Restoring force of steel plate restrainers between Decks 2 and 3 (steel plate restrainers with gap are accommodated)

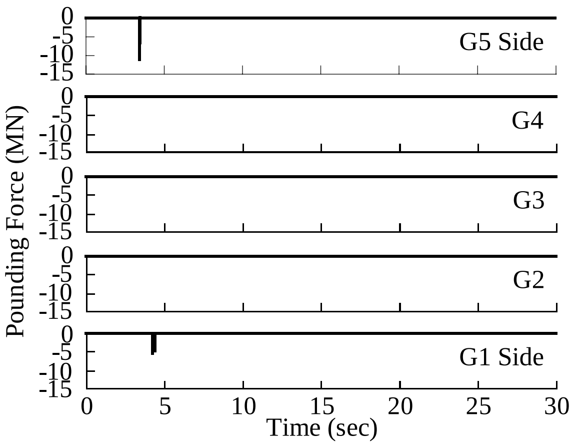



Fig. 23 Pounding force between Decks 2 and 3 (steel plate restrainers with gap are accommodated)

not presented here, progressive failure of restrainers from G1 restrainer to G5 restrainer is resulted from deck rotations as well as translation. Strength demand of restrainers is generally determined by simply dividing the total lateral force by number of restrainers at present. However, the above analysis implies that such a simple assumption is inadequate to estimate the strength demand of restrainers. Enhancement of strength demand is required at restrainers located at the extreme edges. translation.

Pounding occurred between Decks 2 and 3 at the extreme edge near G1 and G5 as shown in Fig. 23. The maximum pounding force is 11.0 MN, which is close to that of the bridge with restrainers without gap.

## 6. Effect of PC Strand Restrainers

Fig. 24 shows deck response displacements when PC strand restrainers are accommodated. Bearings which support Deck 3 on P2 failed at 4.527-4.543 s as shown in Fig. 25. G1 bearing failed first at 4.527 s and subsequently G2, G3, G4 and G5 bearings failed at 4.535, 4.539, 4.541 and 4.543 s, respectively. Residual displacements of Decks 2 and 3 are 0.78 m and 0.75 m, respectively. They are close because restrainers did not rupture as shown later. However response displacements of Decks 2 and 3 are smaller than those of the bridge without restrainers.

Poundings occur at the extreme edge of decks near G1 and G5 as shown in Fig. 26. The maximum pounding force is 8.6 MN, which is 60.5 % the maximum pounding force developed when restrainers are not accommodated.

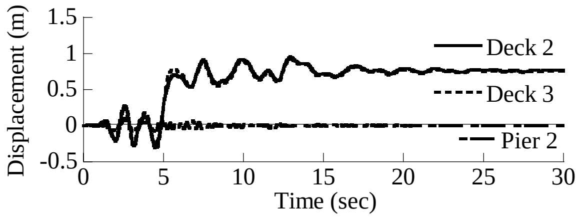



Fig. 24 Deck response displacement in the longitudinal direction (PC strand restrainers are accommodated)

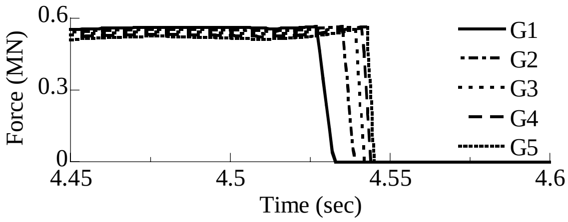



Fig. 25 Progressive failure of bearings which support Deck 3 on Pier 2 (PC strand restrainers are accommodated)

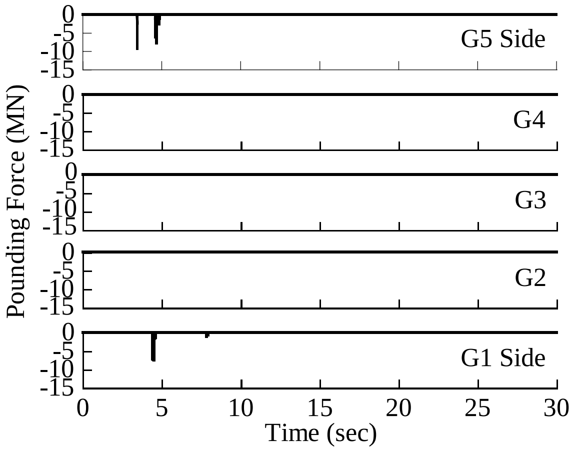



Fig. 26 Pounding force between Decks 2 and 3 (PC strand restrainers are accommodated)

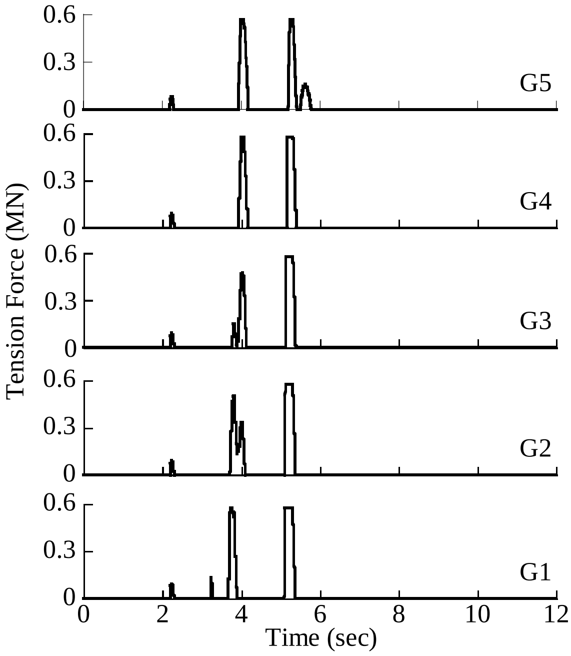



Fig. 27 Tension force induced in the PC strand restrainers between Decks 2 and 3 (PC strand restrainers are accommodated)

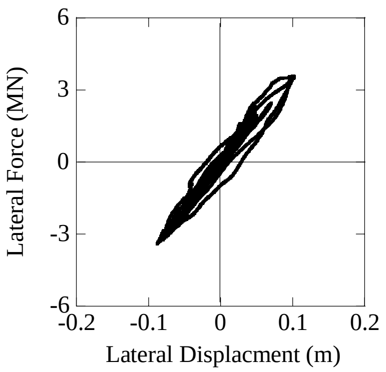



Fig. 28 Lateral force vs. lateral displacement hysteresis of Pier 2 (PC strand restrainers are accommodated)

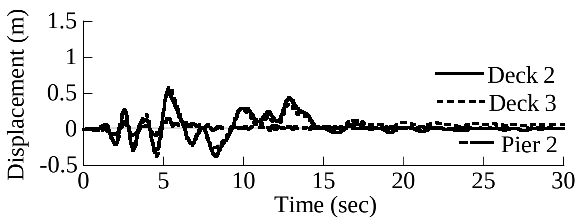



Fig. 29 Deck displacement in the longitudinal direction (PC strand restrainers are accommodated with bearing locks)

Fig. 27 shows tension forces induced in restrainers between Decks 2 and 3. G1 restrainer first yielded at 3.772 s. After direction of deck rotation changed, G5 restrainer subsequently yielded at 3.992 s followed by yield of G4 restrainer at 4.007 s. Again, after direction of deck rotation changed, G1 restrainer yielded (second time) at 5.108 s, followed by yield of G2 restrainer at 5.121 s and G3 restrainer at 5.148 s. This is resulted from combined rotation and translation of the decks. It is important to note that although PC strand restrainers did not rupture, they yielded such that restrainer located at an extreme edge first yielded followed by restrainers located inside.

Fig. 28 shows lateral force vs. lateral displacement hysteresis of P2 in the longitudinal direction. Displacement ductility factor is 2.9, which is close to the ductility factor of P2 (3.1) without restrainers.

## 7. Effect of "Lock" of a Bearing after Failure

An analysis was conducted for the same bridge with PC strand restrainers assuming that G1 bearing of Deck 3 on P2 fails and locks. Fig. 29 shows the deck response of Decks 2 and 3 and P2 in the longitudinal direction under this condition. Because the lock at G1 bearing prevented excessive movement of decks, response displacements of Decks 2 and 3 are 0.53 m and 0.58 m, respectively, and they are smaller than those of bridge when lock of G1 bearing does not occur.

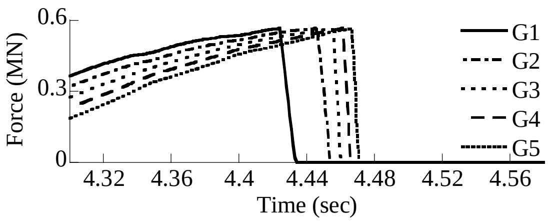



Fig. 30 Progressive failure of bearings which support Deck 3 on Pier 2 (PC strand restrainers are accommodated with bearing locks)

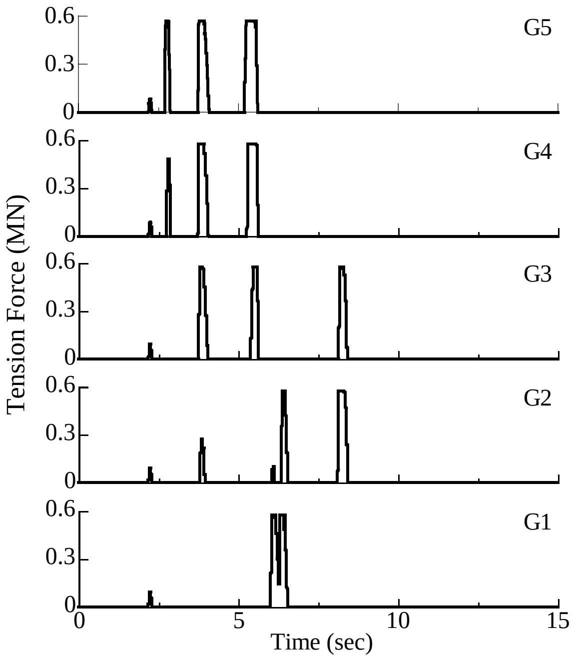



Fig. 31 Tension force induced in the PC strand restrainers between Decks 2 and 3 considering the bearing lock (PC strand restrainers are accommodated with bearing locks)

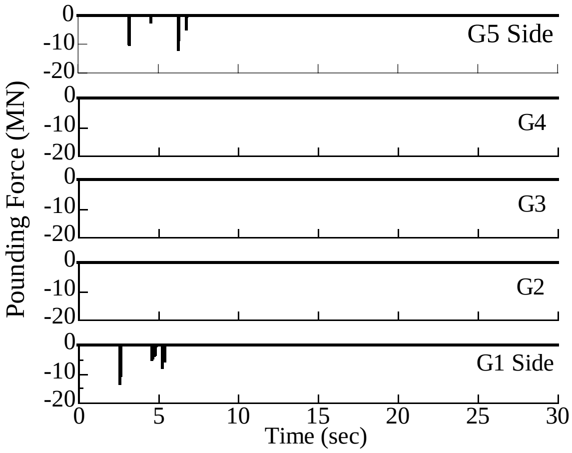



Fig. 32 Pounding force between Decks 2 and 3 (PC strand restrainers are accommodated with bearing locks)

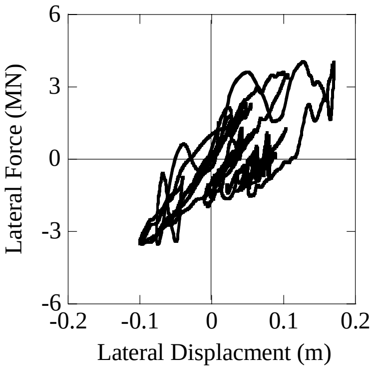



Fig. 33 Lateral force vs. lateral displacement hysteresis of Pier 2 (PC strand restrainers are accommodated and bearing locks)

Failure of bearings of Deck 3 on P2 was initiated at G1 bearing at 4.424 s, and subsequently G2, G3, G4 and G5 bearings failed at 4.446, 4.455, 4.461 and 4.467 s, respectively, as shown in Fig. 30. G1 bearing locked at 4.571 s first, and it subsequently locked 16 times. Maximum locking force of 21.1 MN, which is 3.2 times a deck weight, occurred at G1 bearing by second lock at 5.070 s.

Lock of G1 bearing resulted in more frequent yield of PC strand restrainers as shown in Fig. 31 compared to the response of decks without lock. Yield occurred first at G5 restrainer at 2.733 s and 2.788 s (second time) followed by yield of G4 restrainer at 3.766 s, G5 restrainer at 3.772 s (third time), etc. On the other hand, G1 restrainer first yielded at 6.061 s, and yielded again at 6.295 s, followed by yield of G2 restrainer at 6.392 s, 6.444 s (second) and 8.137 s (third time), etc.

As shown in Fig. 32, pounding between Decks 2 and 3 occurred more frequently than the response of bridge without lock, and the maximum pounding force which occurred at the extreme edge of decks near G1 reached 13.4 MN, which is 94 % the maximum pounding force in the bridge without lock of G1 bearing.

Lateral force vs. lateral displacement hysteresis of P2 in the longitudinal direction is shown in Fig. 33. Because of the large lateral force transferred by lock of G1 bearing which supports Deck 3 on P2, P2 exhibits significant hysteretic behavior with a displacement ductility factor of 4.9.

## 8. Conclusions

Effect of failure of bearings and restrainers was clarified for a 3-span bridge based on nonlinear seismic response analysis. Failure paths and progressive failure modes are studied. Based on the resulted presented herein, the following conclusions may be deduced:

・Relative opening and closure between two adjacent decks are generally larger at the extreme edge of decks resulted from combined rotation and translation of decks. As a result, larger lateral seismic force applies to the bearings and restrainers located at the extreme edge of decks. Consequently, it is likely that rupture and failure are initiated from the bearings and restrainers located at an extreme edge, and propagate to bearing and restrainers located at the other extreme edge. Thus progressive failure occurs at bearings and restrainers.

・Evaluation of strength of restrainers by simply dividing the total lateral force by number of restrainers underestimates real strength demand of restrainers located at the extreme edge of decks. Enhancement of strength demand of restrainers at the edge is required.

・Poundings between two adjacent tend to occur at the extreme edge of decks first as a result of rotation and translation of decks.

・PC strand restrainers yielded but did not rupture while steel plate restrainers ruptured at the early stage of excitation. Because deformation characteristics of steel plate restrainers depend on simple assumption, their performance should be carefully re-evaluated based on reliable properties. However restrainers are very important to control both peak response and residual displacement of decks in translation and rotation.

・Lock of bearing which could occur due to rupture results in transfer of large lateral force from deck to piers. This results in large plastic deformation of piers. Because it is difficult to predict the locations where lock occurs, worst scenario has to be clarified based on engineering experience and analysis.

## References

1) Japan Road Association, (1996 and 2002), *"Part V Seismic Design, Design Specifications of Highway Bridges,"* Maruzen, Tokyo, Japan

2) Kawashima, K. and Unjoh, S., (1997), "The Damage of Highway Bridges in the 1995 Hyogo-ken Nanbu Earthquake

and Its Impact on Japanese Seismic Design," *Journal of Earthquake Engineering*, Vol. 1, No. 3, pp. 505-541.

3) Kawashima, K. and Shoji, G., (2000), "Effect of Restrainers to Mitigate Pounding between Adjacent Decks subjected to a Strong ground Motion," *Proc. 12 WCEE*, Paper No. 1435 (CD-ROM), Auckland, New Zealand.

4) Saiidi, M.S., Maragakis, E. and Feng, S., (1996), "Parameters in Bridge Restrainer Design for Seismic Retrofit," *Journal of Structural Engineering*, ASCE, Vol. 122, pp. 55-61.

5) Iemura, H., Miyamoto, A. and Takahashi, Y., (1998), "Influence of Failure of Steel bearings on Damage Modes of Bridges under Strong Earthquake Motion," *Journal of Structural Engineering*, Vol. 44A, pp. 659-666.

6) Hoshikuma, J., Kawashima, K., Nagaya, K. and Taylor, A.W., (1997), "Stress-Strain Model for Confined Concrete in Bridge Piers," *Journal of Structural Engineering*, ASCE, 123(5), pp. 624-633.

7) Sakai, J. and Kawashima, K., (2006), "Unloading and Reloading Stress-Strain Model for Confined Concrete," *Journal of Structural Engineering*, ASCE, 132 (1), pp. 112-122.

8) Menegotto, M. and Pinto, P. E., (1973), "Method of Analysis for Cyclically Loaded R.C. Plane Frames Force and Bending," *Proc. IABSE Symposium on Resistance and Ultimate Deformation of Structures Acted on by Well Defined Repeated Loads*, pp. 15-22

9) Watanabe, G. and Kawashima, K., (2004), "Numerical Simulation of Pounding of Bridge Decks," *Proc. 13th World Conference on Earthquake Engineering*, Paper No. 884, Vancouver, Canada.

10)Tirasit, P. and Kawashima, K., (2005), "Torsion Response of Skewed Bridge Piers," *Journal of Earthquake Engineering*, 28, CD-ROM, No. 116, Japan Society of Civil Engineers, Tokyo, Japan

(Received September 11, 2006)