Discussion Method by using Dynamic Analysis for Quakeproofing Long-Span Bridges

Manager, Bridge Section, Road Department. Takumi Nishikawa
Deputy Manager, Bridge Section, Road Department Mutsuji Kawakami
Assistant Manager, Bridge Section, Road Department Tatsumi Ujie
Bridge Section, Road Department Shunji Nagahashi

1. Introduction

The capital in Western Japan, Osaka City, is the second largest city in Japan, with only Tokyo surpassing it, and is the center for finance and culture. Osaka City is known as the "City of water", and its bridges have been loved by the Japanese, down through the ages. Currently, the Construction Section of Osaka City manages 764 bridges, which includes about 720,000 square meters of bridge. There are a great variety of bridges, including continuous viaducts where 100,000 or more cars a day pass through, long-span bridges across large rivers (Picture 1), and bridges (Picture 2) that have been loved by the locals throughout history and are closely connected to the lives of the average person.

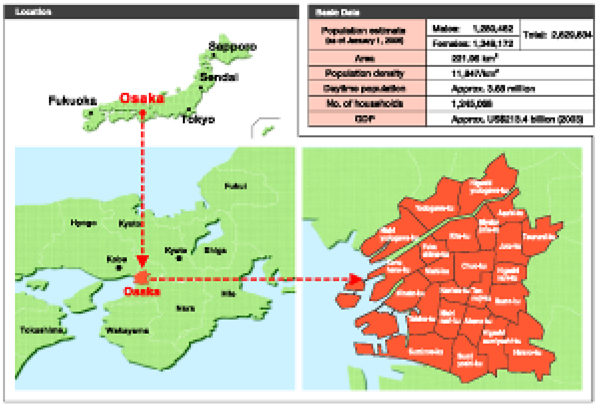


Figure 1 Location of Osaka City

Picture 1 Continuous viaduct
(New Midosuji line)

Picture 2 Historical bridge
(Namba Bridge)

The Southern Hyogo Prefecture Earthquake in 1995 caused much damage to many of the bridges. Even Osaka City (Figure 1), which is about 30 km east of Kobe City, suffered major damages, such as cracks in the reinforced concrete bridge supports of the viaduct, although no bridges actually collapsed. Thus, since 1996, Osaka City has advanced earthquake-resistant projects for 331 bridges (about 43%), which need earthquake-resistant measures, among the 764 bridges managed by the Construction Section. Earthquake-resistant projects have reinforced the bridge supports of viaducts, installed travel-limiting devices and devices for the prevention of collapse, and widened bridge seats. At present (March 2011), the project has been implemented to about 94%of the bridges that have been chosen for this project.

On the other hand, there have been many problems on quakeproofing long-span bridges that a river impediment ratio (Cabinet Order concerning Structural Standards for River Management Facilities, etc) is not satisfied as a building frame of bridge supports becomes thicker after a reinforcement, reinforcement constructions are prolonged because reinforcement construction periods are limited to non-flood seasons and it takes large money and time for temporary cofferdam. Therefore, we determined to improve better earthquake resistance of the long-span bridge by changing the support conditions of the bridge by replacing bearing supports and installing damper devices as well as adopting discussion methods with dynamic analyses. This paper will report on the earthquake-resistant project where support conditions of the long-span bridge, Nagara Bridge, were changed and dynamic analyses were used as a discussion method.

2. Outline of the Nagara Bridge

The Nagara Bridge, which will be reported in this paper, is located on the main road that connects the major cities of Osaka and Kyoto, and crosses the Yodo River, going to Osaka City. This river dates back a long way (Figure 2). A reliable book says that the bridge had already been constructed in the 9th century and many Japanese old poems, wakes, speak of the Bridge. The current Nagara Bridge was completed in 1983, with a Nielsen-Lohse beams located at the center of the Bridge, the 4 span continuous steel floor plate 2 main beam on the left bank of the Bridge (length 294 m) and the simple steel floor box-girder (length 76 m) and 2 span continuous steel floor plate 2 main beam (length 131 m) located on the right bank of the Bridge.

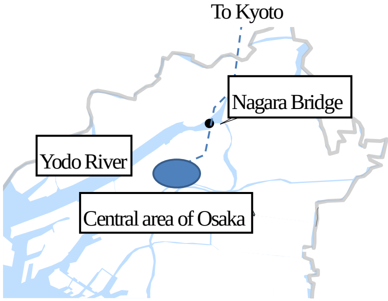
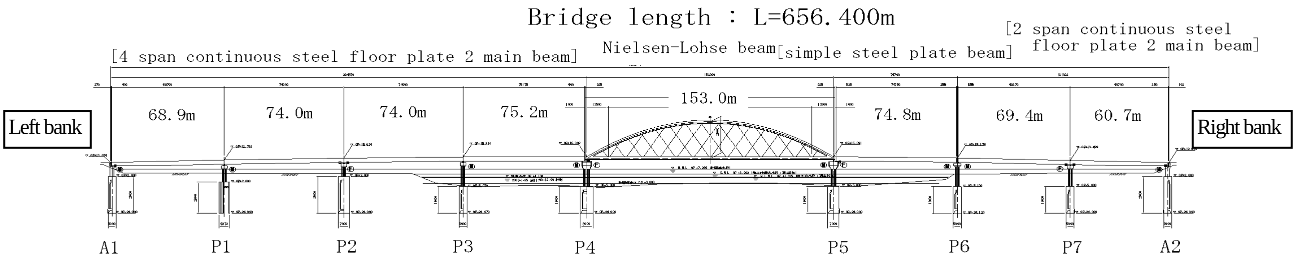


Figure 2 Location of Nagara Bridge

Picture 3 Nagara Bridge
(Nielsen-Lohse bridge)

Picture 4 Nagara Bridge
(Steel plate floor beam bridge)



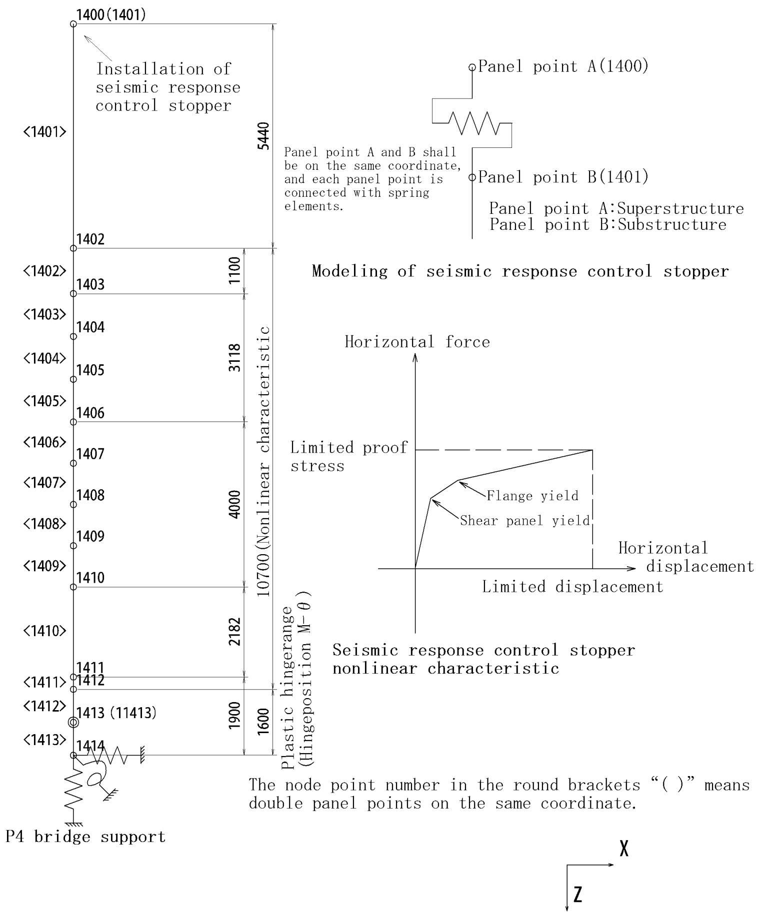
3. Earthquake-resistant design concept, analysis model and designed ground motion

The concept for the earthquake-resistant reinforcement design complied with the national design standard 1) and is designed for ground motion at 2 levels. That is, the purpose of considering the levels is to prevent damage to the Bridge (to insure the same function of the Bridge before an earthquake occurs) during ground motion (level 1 ground motion) which is very likely to occur during the in-service period of the Bridge, and to prevent fatal damage, such as the collapse of the Bridge, during an earthquake which occurs rarely (level 2 ground motion: Two types of earthquakes are assumed: the inner-plate earthquake that infrequently occurs, but that has large scale ground motion, and the inland local earthquake). Because the Nagara Bridge is designated as an emergency traffic route in the regional disaster prevention plan of Osaka City, it is especially necessary to prevent the bridge from sustaining extensive damages which may lead to collapse of the Bridge, and to insure its earthquake resistance so that it can function after an earthquake; therefore, we designed the reinforcement so that deformation of the bridge support foundation would not exceed the plastic deformation capacity in order to enable it to be easily repaired even if the Bridge suffers some damage.

Next, the analysis models used for discussion are described. The Nagara Bridge is composed of four bridges (4 span continuous steel floor plate 2 main beam , Nielsen-Lohse beam, simple steel floor box-girder, and 2 span continuous steel floor plate 2 main beam) and designed vibration units of the bridges are assumed to operate separately, without transferring the earthquake load to each other. For dynamic analysis modeling of the bridge axis direction, each bridge unit was used. At the transverse direction to the bridge axis, as the earthquake loads are influenced each other through the staggered bridge supports, the whole Nagara Bridge, with all four beams combined, could be seen as one analysis model. The superstructure is an elastic frame model, reinforced concrete bridge supports are a trilinear model (Takeda model 2)) and the foundation is a nonlinear spring element as to analysis conditions (Figure 4).

The most distinctive feature of improvement for earthquake resistance of the Bridge is the distribution of the load to make it resistant to earthquakes, and to make the structure of the bridge to be the structure with the primary natural period by changing the bearing support conditions of the existing bridges, which is a one-point fixation that bears inertia force from the bridge axis direction, to elastic bearing supports. Another feature is a change in the structure so that the bearing supports can be expected to have an attenuation effect by using shear and friction hysteretic dampers. The hysteresis of these dampers is shown below.

Superstructure		Elastic girder model	
Abutment		Elastic girder (total cross-section rigidity)	
Bridge support	Girder	Elastic girder model(rigid zone)	
	Column	General material	M-ϕ trilinear model(takeda model)
		Plastic hinge	M-θ trilinear model(takeda model)
Foundation		Intensive ground spring	



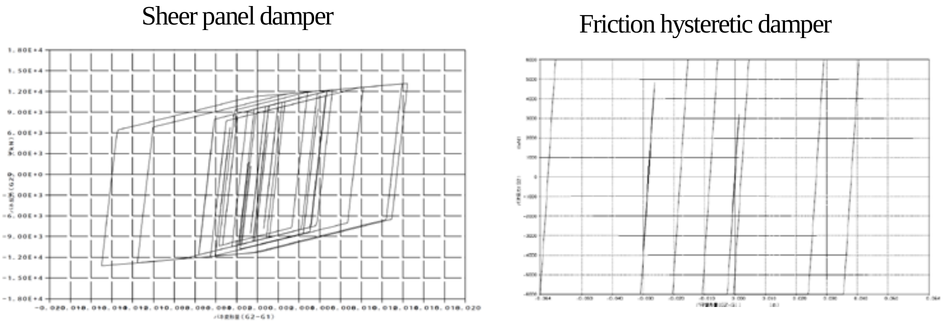


Figure 5　Example of response hysteresis

The seismic wave used as the national standard and input seismic wave 3) set by Osaka City were used as the input ground motion used for the dynamic analyses of earthquake-resistant design for the Nagara Bridge in this paper. These are seismic waves due to Uemachi faulting directly under Osaka City, and specify the standard input ground motion used for earthquake-resistant design for civil engineering structures, in response to the earthquake in the Southern Hyogo Prefecture Earthquake in 1995. Figure 6 shows the acceleration response spectrum of the input seismic wave adopted in this paper.

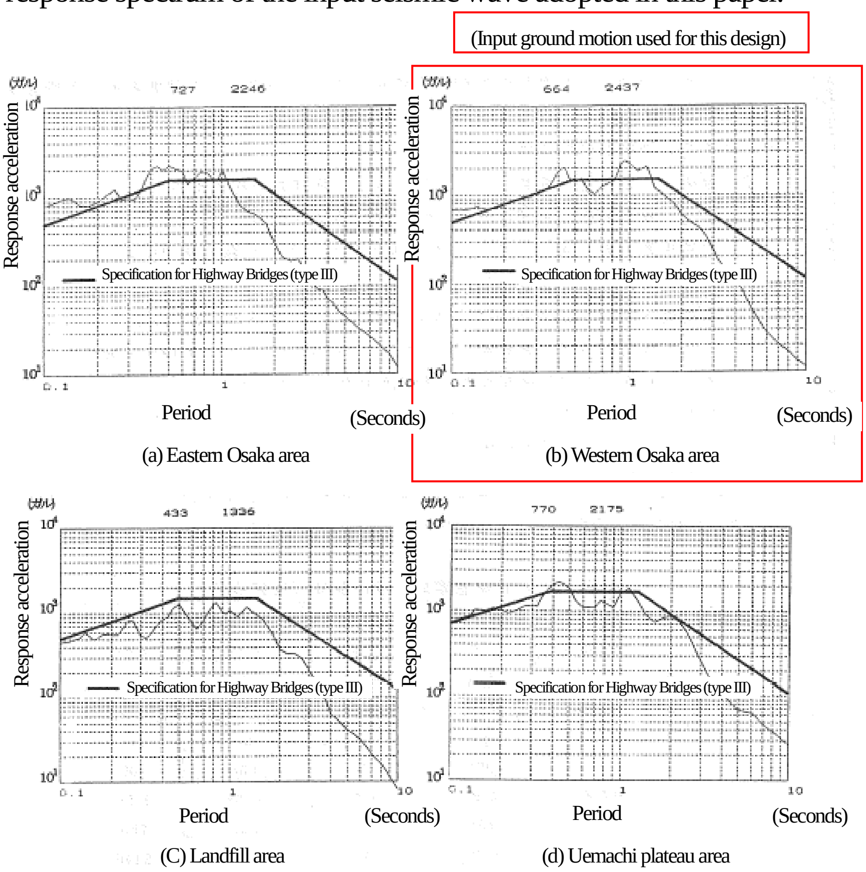


Figure 6 Seismic wave in Osaka City

4. Survey on earthquake resistance of the current reinforced concrete bridge support

The basis of earthquake-resistant reinforcement for a reinforced concrete bridge support is to change the failure mode to a flexure failure mode on the bridge support foundation where toughness is expected, while avoiding shear failures which show drastic failure on the bridge support.

The method described in “Existing bridge earthquake resistant reinforcement method cases” 4) published by the Japan Bridge Engineering Center was used to evaluate the damage on the cut-off area of longitudinal reinforcing steel. If the reinforcement of the bridge support foundation was implemented on a large scale, and it was difficult to change a failure mode to a flexure failure mode of the bridge support foundation, we surveyed that the plasticity rate of the cut-off area was to the extent that the proof strength will not lower (the maximum response yield curvature of the cut off area should be twice the initial yield curvature or less).

In order to confirm safety, the response rotation angles of the plastic hinges, the response shear force of the bridge supports, and the values of the residual displacement of the bridge supports, acquired from dynamic analyses were surveyed. Concretely, we surveyed that the maximum response rotation angle of the plastic hinges shall be the allowed rotation angle specified in Specification for Highway Bridges with Commentary, PART V:SEISMIC DESIGN Section 10.2, or smaller, the maxim response shear force of the bridge supports shall be shear force of the reinforced concrete bridge support or lower, and the residual displacements of the bridge supports calculated using the maximum response plastic rate shall be the allowed residual displacement (1/100 of bridge support height) or lower. As the bridge support is a wall type bridge support, the effects of the deep beams are considerate for calculation of the shear proof strength at the transversal direction to the bridge axis.

Earthquake resistance of a structure with the current bearing support condition (Table 2 CASE 1) being maintained was surveyed in the case of a ground motion at level 2. The bridge support foundation and cut-off area of P2, P4, P5 and P7 bridge supports lacked flexural and shear proof strength to the bridge axis direction (Table 3 CASE 1). P1 bridge support showed an adequate earthquake resistance. Although the cut-off area of P3 and P6 bridge supports reached to the plastic field, that range was only to the extent in which proof strength did not decrease.

All bridge supports showed the shear failure type to the transverse direction to the bridge axis. The shear proof strength calculated in consideration of the effects of the deep beams exceeded the maximum response shear force, and met the required earthquake-resistant strength. Therefore, if the bridge supports were mechanically reinforced against earthquake loads acting on each support, while the current bearing support conditions were maintained, the reinforcement of the cut-off area of P2, P4, P5 and P7 bridge supports, and the shear reinforcement and flexural reinforcement of the foundation would be necessary. Especially, the flexural reinforcement of the bridge support foundation should be large scale reinforcements in which longitudinal reinforcing steel is established to the footing.

5. Discussion on earthquake-resistant reinforcement

5-1. Determination of the policy for reinforcement

For discussion of the whole structure of the Bridge, under the conditions of bearing supports indicated in Table 2, a bridge support reinforcement method where reinforcement against earthquake loads acting on the bridge support was implemented (CASE 1), a seismic isolation method which makes vibration periods of the structures longer and enhances attenuation (CASE 2) and a seismic response control method which enhances attenuation and distribution of earthquake loads (CASE 3) were weighed. Table 3 shows the results of the need of bridge support reinforcement.

In CASE 2, the natural period was lengthened by using seismic isolation bearing supports, such as a laminated rubber bearing, and the inertial force was expected to be reduced by improving the attenuation performance. Reinforcement of P1 and P6 bridge supports was necessary, as well as reinforcement of P2, P4 and P5 bridge supports, and the scales of flexural reinforcement, shear reinforcement and the cut-off reinforcement were larger, resulting in the construction costs being high. The traveling distances of the beam on A1 abutment, P6 bridge support and A2 abutment were greater than those of the expansion gap, resulting in a collision of the beams.

CASE 3 was a construction method which controls deformation of the entire bridge and attenuates the inertia force transmitted to the bridge support by installing dampers that deforms during an earthquake and absorbs the energy around the bearing supports. The seismic response control dampers were located at A1 and P2 supporting points of the 4 span continuous beam ranging from A1 to P4, P4 supporting point of the Nielsen-Lohse beam ranging from P4 to P5, P5 supporting point of the simple beam ranging from P5 to P6, P7 supporting point of the 2 span continuous beam ranging P6 to A2, and A2 supporting point. In this structure, the inertia force transmitted to the substructure dramatically decreased, shear and cut-off reinforcements were necessary only for P2, P4 and P5 bridge supports. As flexural reinforcement (establishment of longitudinal reinforcing steel to the foundation) was unnecessary, the earthquake loads to the foundation did not increase. A travelling distance of the beam at staggered areas was shorter than the expansion gap with the adjacent beam, and beams did not collide with each other.

As a result of a comparison of the construction costs of these three methods, we decided on the earthquake-resistant measure based on CASE 3, which controlled deformation of the entire bridge and attenuated the energy by installing seismic response control dampers. For this basic structure, construction methods for the reinforcement of the bridge supports were compared, types of seismic response control dampers were compared, and shear plate type dampers and friction hysteretic dampers were designed.

Table 2 Bearing support conditions

M= Movable, F= Fixed, E= Elastic (rubber)

	A1	P1	P2	P3	P4		P5		P6		P7	A2
CASE 1	M	M	F	M	M	F	M	F	M	M	F	M
CASE 2	E	E	E	M	M	E	E	E	E	M	E	E
CASE 3	M Damper	M	F(M) Damper	M	M	F+ shear panel	M	F + shear panel	M	M	F Damper	M Damper

Table 3 List of need of bridge support reinforcements

- …Reinforcement: Unnecessary, ●…Reinforcement: Necessary

	A1	P1	P2	P3	P4	P5	P6	P7	A2	Remarks
CASE 1	-	-	●	┌	●	●	‾	●	-	◎…Plastic
CASE 2	-	●	●	-	●	●	●	-	-	Beams may collide.
CASE 3	-	-	●	-	●	●	-	-	-	

5-2. Bridge support reinforcement

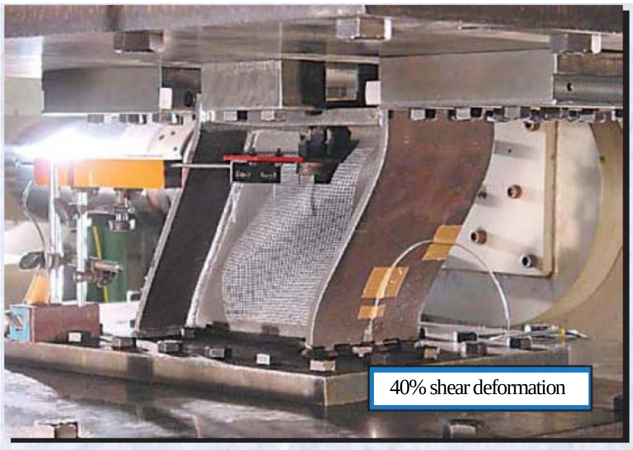
A PC confined method was adopted for reinforcement of the bridge support at the low-water channel of a river. The PC confined method enables improvement of the horizontal proof strength and toughness during an earthquake by prestressing the PC steel wires inserted into the sheath in the PC precast panel after the panel is installed around the bridge supports, as well as improvement of work efficiency at the site, by using precast panels manufactured at a plant.

A reinforced concrete lining method, a carbon fiber material lining method, and a steel plate lining method were compared and studied for the reinforcement of the bridge support at the high-water channel of a river. As the carbon fiber material lining method has excellent work efficiency but is vulnerable to fires and shocks, this method still has operation and maintenance problems. The least expensive method of these methods, the reinforced concreted lining method was adopted.

5-3. Seismic response control dampers

Seismic response control dampers have shear panel and friction hysteretic dampers. The shear panel damper (Picture 5) absorbs earthquake energy and reduces the cross-sectional force which transmits to the substructure due to the shear deformation of the panels which are composed of low yield point steel. This damper normally works as a fixation device and when a level 1 earthquake occurs. The shear panel yields with ground motion due to an earthquake at level 2 and this damper absorbs earthquake energy with hysteresis attenuation of shear plastic deformation. As this damper normally works as a fixation device, it was installed at one supporting point of each beam, so that the beam could be elastic during temperature changes (P4 supporting point of the Nielsen-Lohse beam and P5 supporting point of the simple steel plate 2 main beam I beam).

The friction hysteretic damper (Picture 6) absorbs vibration with the flow resistance force that occurs during piston movement in the high viscosity material filled into the cylinders, and normally works as a movable point. This damper is an elastic fixation against the large shaking of an earthquake. This damper can be installed at several supporting points in the 1 structure system to ensure elasticity during normal temperature changes. In this paper, this damper was installed at A1 and P2 supporting points of the 4 span continuous steel plate 2 main beam I beam and P7 and A2 supporting points of the 2 span continuous steel plate 2 main beam I beam.



Picture 5 Shear panel damper
(Static loading test)

Picture 6 Friction hysteretic damper
(Nagara Bridge)

5-4. Stimulated fixation of P2 bearing support

In the 4 span continuous steel floor 2 main beam bridge ranging from A1 to P4, only P2 supporting point was a fixation bearing and the rest of the supporting points were movable bearings. In this earthquake-resistant measure, all of the supporting points were changed to movable supporting points and the friction hysteretic dampers were installed at A1 and P2 supporting points. When all supporting points are made to be movable, the fixed point against temperature change is unclear. As a measure against this issue, we stimulated fixation to P2 bearing support by installing blocks which limit travelling toward the bridge axis so that the beam could normally stretch around P2 supporting point. In this case, the number and radius of the bolts were adjusted so that the seismic response control dampers would work effectively at ground motion at level 2 and the fixation bolts of the block would fracture with the horizontal loading due to ground motion at level 1 or higher.

6 Conclusion

For earthquake-resistant reinforcement, structures of a general reinforcement method which reinforces each bridge support against cross-sectional force acting on the bridge support while the current bearing support conditions are maintained were discussed as well as the seismic isolation construction method, and the earthquake response control method. Accordingly, the optimal method was selected where a high attenuation device, such as a seismic response control damper, could absorb the earthquake energy, reduce the earthquake loads transmitted to the bridge supports, and control the deformation of the entire bridge. Then, the friction hysteretic damper and shear panel damper that use a low yield point steel were designed, in consideration of the structure characteristics of the Bridge. Currently, the reinforcement construction of the Nagara Bridge has been conducted for early completion. We received guidance from Hirokazu Iemura (professor emeritus at Kyoto University) for this paper. We will express our sincere gratitude to him.

Reference

1) Japan Road Association: Specification for Highway Bridges with Commentary, PART V: SEISMIC DESIGN, Published by Maruzen, 2002-3

2) Takeda, T., Sozen, M.A. and Nielsen, N. M: Reinforced Concrete Response to Simulated Earthquakes, Journal Division, ASCE, Vol. 96, No. ST12, PP. 2557-2573, December, 1970

3) Osaka City Planning Section: Osaka City Civil Engineering and Architectural Buildings Earthquake Measure Technology Conference Report, 1997-3

4) Japan Bridge Engineering Center: Existing bridge earthquake resistant reinforcement method cases, 2005-4

5) Maruyama. T. Seismic Retrofitting for Bridges in Osaka City, Osaka and Its Technology, Osaka Municipal Government, No.32, PP7-18, 1998.1