SEIMIC REINFORCEMENT OF 1ST IBI VIADUCT

Sunao Kawabata[1]

Abstract

The 1st Ibi viaduct is a three-span continuous bridge with two abutments and two piers. The piers of the bridge are designed flexible to follow the superstructure's displacement caused by the temperature change and by others.

The bridge was designed according to the old seismic design specifications. After the evaluation by the latest specifications, it was found that the seismic performance of the bridge was not enough. Dissipation measures using viscous dampers were applied to reducing the seismic force at large-scale earthquakes. This reinforcement is expected not to make unfavorable influence on the bridge except when large-scale earthquakes occur.

1. Introduction

The 1st Ibi viaduct is in the Kobe-Awaji-Naruto Expressway, the most eastern route of three Honshu-Shikoku Expressways connecting Shikoku Island with Honshu Island, as shown in Fig. 1. The Akashi Kaikyo Bridge and the Ohnaruto Bridge are in the route. The 1st Ibi Viaduct began to service in 1985.

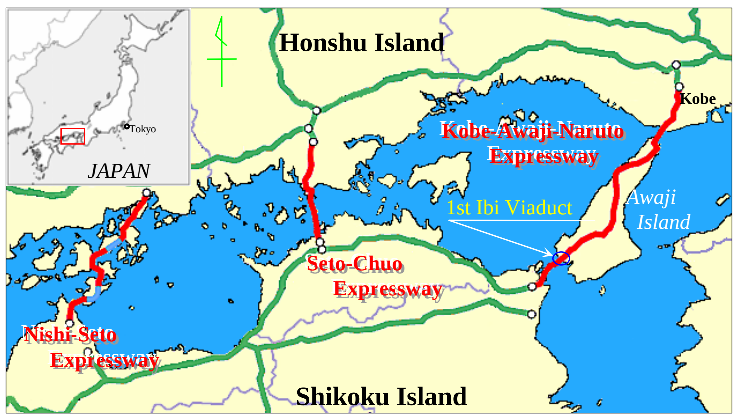

Fig. 1 LOCATION OF 1ST IBIVIADUCT

The 1st Ibi viaduct was designed according to the 1980's seismic design specifications for highway bridges. The design specifications, however, was revised in 1996 to require significantly higher performance against large-scale earthquakes due to 1995 Hyogo-ken Nanbu Earthquake. Also, according to the latest studies, it was found that major earthquakes will occur in the near future nearby the bridge. Therefore, we started to review its seismic performance, and then found that the bridge did not have enough performance to satisfy the latest specifications almost same as that of 1996. In addition, the expressway which includes the bridge has been decided as the emergency transportation route at the time of disaster. Under such a situation, we decided to reinforce the 1st Ibi viaduct in order to get enough performance against large-scale earthquakes.

[1] Bridge Engineer, Honshu-Shikoku Bridge Expressway Co., Ltd. Kobe, Japan

2. Outline of the 1st Ibi Viaduct

As shown in Fig. 2, the 1st Ibi viaduct is a three-span continuous bridge, and its length is 146m. Its superstructure is composed of six I-section steel plate girders and a reinforced concrete slab. There are two abutments (1A and 4A) at both ends of the bridge, and there are two flexible reinforced concrete piers (2P and 3P) between both abutments. The superstructure is supported at 1A, 2P and 3P against the longitudinal direction movement, and supported at all piers and abutments against the transverse direction movement. Since the fixed bearings are used at 2P and 3P, these piers have to follow the superstructure's displacement cased by temperature change, by live load and by others.

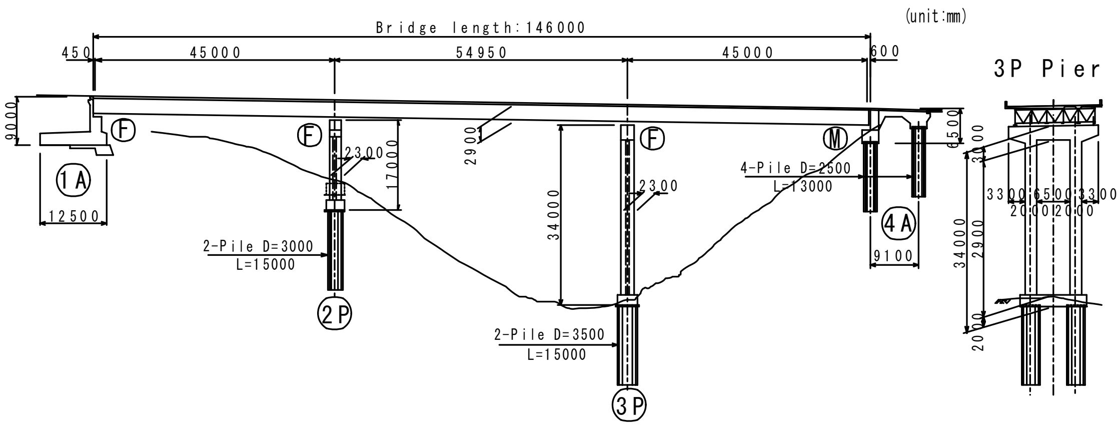

Fig. 2 ELEVATION OF 1ST IBI VIADUCT

3. Seismic vulnerability evaluation of existing bridge

3.1. Seismic performance and design earthquake motion

In the latest seismic design specifications for highway bridges, two levels of design earthquake ground motions have to be considered. The first level corresponds to a middle-scale earthquake (Level 1 earthquake ground motion). And second level corresponds to large-scale earthquakes (Level 2 earthquake ground motion). There are two types earthquake ground motion in the Level 2, one is earthquake ground motion generated by a Plate boundary type earthquake with a large magnitude (Type 1), and the other is earthquake ground motion generated by an Inland active fault type earthquake (Type 2). All the design earthquake ground motions are shown in Fig. 3. Any highway bridges have to have the

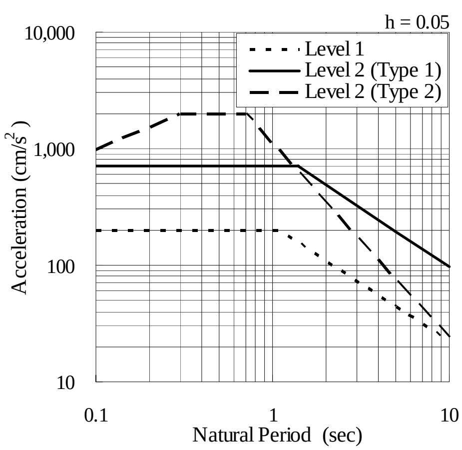

Fig. 3 DESIGN EARTHQUAKE GROUND MOTIONS

seismic performance shown in Table 1.

The 1st Ibi viaduct was designed not to exceed its elastic limit state during only the Level 1 earthquake ground motion in its original seismic design. It was not clear that the bridge had the seismic performance for the Level 2 earthquake ground motion. Therefore, we had to verify whether the bridge had enough seismic performance or not.

Table 1 SEISMIC PERFORMANCE

<table>
<tr><th colspan="2" rowspan="2">Earthquake Ground Motions</th><th colspan="2">Importance of Bridges</th></tr>
<tr><th>Common</th><th>High</th></tr>
<tr><td colspan="2">Level 1 (highly probable during the bridge life)</td><td colspan="2">Keeping sound functions of bridges
(Seismic Performance Level 1)</td></tr>
<tr><td rowspan="2">Level 2</td><td>Type 1 (a Plate boundary type earthquake with a large magnitude)</td><td rowspan="2">No critical damages
(Seismic Performance Level 3)</td><td rowspan="2">Limited seismic damages and capable of recovering bridge functions within a short period
(Seismic Performance Level 2)</td></tr>
<tr><td>Type 2 (an Inland active fault type earthquake)</td></tr>
</table>

3.2. Seismic analysis for large-scale earthquakes

The behaviors of the bridge at large-scale earthquakes were analyzed by non linear time history response analyses with the analytical model of the entire bridge system.

The results of analyses are shown in Table 2. In the longitudinal direction, reaction forces of bearings at 1A significantly exceeded the allowable values. So these bearings will be broken at large-scale earthquakes. On the other hand, in the transverse direction, reaction forces of bearings at 2P and 4A exceeded the allowable values, and shear force of 3P pier also exceeded the allowable values. So these components will suffer damage at large-scale earthquakes.

As the bearings at 2P and 3P are fixed type, it is necessary to know the bridge behavior in the longitudinal direction after broken of 1A fixed bearings. The fixed bearings at 1A were transposed to movable bearings in the analytical model, and seismic response analysis was performed again. This result is shown in Table 3. It shows that 2P pier and 3P pier may be broken by the bending moment when 1A fixed bearings were broken. From these results, it was found that the bridge did not have enough seismic performance for large-scale earthquakes, and needed to be reinforced.

Table 2 SEISMIC VULNERABILITY EVALUATION OF EXISTING BRIDGE

Direction of Input Ground Motion	Evaluatin Items	Type of Level 2 E.Q.	1A		2P		3P		4A	
			Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value
Longitudinal	Curvature (10^{-6}/m)	Type 1	---	---	64	2,812	685	2,457	---	---
		Type 2	---	---	255	7,213	3,155	5,305	---	---
	Shear Force (kN)	Type 1	---	---	1,519	4,316	3,102	4,897	---	---
		Type 2	---	---	3,792	4,796	5,006	5,442	---	---
	Reaction force at bearing (kN)	Type 1	**23,510**	**9,408**	1,439	6,402	3,207	6,402	---	---
		Type 2	**63,276**	**9,408**	2,908	6,402	4,323	6,402	---	---
Transverse	Curvature (10^{-6}/m)	Type 1	---	---	634	1,260	83	892	---	---
		Type 2	---	---	2,216	3,268	472	2,233	---	---
	Shear Force (kN)	Type 1	---	---	9,868	13,295	7,841	10,225	---	---
		Type 2	---	---	12,865	13,534	**12,016**	**10,530**	---	---
	Reaction force at bearing (kN)	Type 1	2,721	9,408	**7,843**	**6,402**	2,482	6,402	**5,602**	**3,942**
		Type 2	7,385	9,408	**9,450**	**6,402**	4,397	6,402	**9,258**	**3,942**

*) value at the most critical section is described at each

Table 3 SEISMIC VULNERABILITY EVALUATION WITH BEARING FAILURE AT 1A

Direction of Input Ground Motion	Evaluatin Items	Type of Level 2 E.Q.	1A		2P		3P		4A	
			Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value
Longitudinal	Curvature (10^{-6}/m)	Type 1	---	---	**12,659**	**3,018**	**2,615**	**2,551**	---	---
		Type 2	---	---	**7867**	**7,213**	1,052	5,628	---	---
	Shear Force (kN)	Type 1	---	---	3,000	4,024	2,477	4,897	---	---
		Type 2	---	---	2,917	4,796	3,605	5,442	---	---
	Reaction force at bearing (kN)	Type 1	---	---	2,708	6,402	3,207	6,402	---	---
		Type 2	---	---	2,757	6,402	4,323	6,402	---	---

*) value at the most critical section is described at each

4. Seismic reinforcement

4.1. The measures in the longitudinal direction

It is necessary for the measure of reinforcement to avoid giving the original structural system any unfavorable influence except when large-scale earthquakes occur, and to have ease of execution and economical efficiency. One solution for improving of the seismic performance is to strengthen the function to fix the superstructure horizontally at the bearings of 1A. However, since the horizontal reaction force of bearings at 1A was very large, and there was no place in which such huge devices could be installed in 1A abutment. Therefore, other solutions were studied considering an entire bridge system, on assumption that 1A is free in the longitudinal direction.

Since the broken of the fixed bearings at 1A causes the destructions of 2P and 3P piers, the measure to install the viscous dampers connecting the superstructure and abutments was selected. The damper is mainly composed of a steel cylinder, a steel piston, and silicone resin filling, as shown in Fig. 4. The relationship between the resisting force and the displacement is as shown in Fig. 5, and its resisting force is proportional to the 0.1st power of its velocity ($F = c \times V^{0.1}$, F : resisting force, V : velocity, c : constant value).

Because the resisting force of this damper is small enough in low velocity to make the dampers give no influence to the structural system except when large-scale earthquakes occur.

In the study of the damper specification, the seismic response analyses with varying resisting force of dampers were performed. It was found that the required resisting force, in order to reduce the responses of both piers lower than the allowable values, and in order to avoid the collision between the superstructure and the abutments, was 16,000kN/bgridge or more. Then, considering types of the existing damper products and their installing space, we decided that 6 dampers, each 1,500kN/damper, install at both abutments (the whole resisting force 1,500kN/damper x 6 dampers x 2 abutments = 18,000kN/bridge). The result of the seismic response analysis in consideration of resisting force of dampers is shown in Table 4. It shows that the responses of both piers and the displacement of the superstructure are within their allowable values.

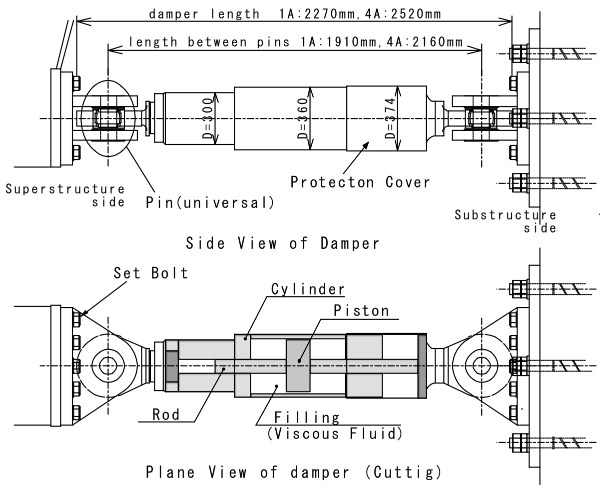

Fig. 4 SCHEMA OF VISCOUS DAMPER

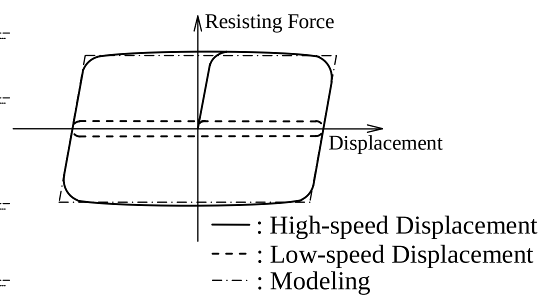

Fig. 5 MECHANICAL CHARACTERISTIC OF DAMPER

Table 4 SEISMIC VULNERABILITY EVALUATION WITH DAMPERS

Direction of Input Ground Motion	Evaluatin Items	Type of Level 2 E.Q.	1A		2P		3P		4A	
			Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value
Longitudinal	Curvature (10^{-6}/m)	Type 1	---	---	53	2,701	590	2,551	---	---
		Type 2	---	---	1427	7,213	4,006	5,638	---	---
	Shear Force (kN)	Type 1	---	---	1,591	6,016	4,863	6,826	---	---
		Type 2	---	---	2,714	4,796	5,357	5,674	---	---
	Reaction force at bearing (kN)	Type 1	---	---	2,288	6,402	3,674	6,402	---	---
		Type 2	---	---	3,677	6,402	5,435	6,402	---	---
	Displacement of girder (mm)	Type 1	8.3	100	---	---	---	---	7.9	250
		Type 2	69.9	100	---	---	---	---	70.3	250

*) value at the most critical section is described at each , Considering influence of reinforcement of the transverse direction

4.2. The measures in the transverse direction

In the transverse direction, the measures of the bearings at 2P and 4A, and the measures against the shear force at 3P pier are required.

The horizontal forces at 2P and 4A cause destruction of these bearings, but they are small enough not to make these substructures any damage. Therefore the equipments, which prevent the superstructure movement in the transverse direction, at 2P and 4A were selected to install.

The 3P pier needs reinforcement against the shear force. It is important to minimize the increase in its mass and rigidity of 3P pier as little as possible to avoid the influence of seismic behaviors in another direction. We decided that the wall thickened 500mm with reinforced concrete in order to increase the shear capacity of 3P pier, as shown in Fig. 6.

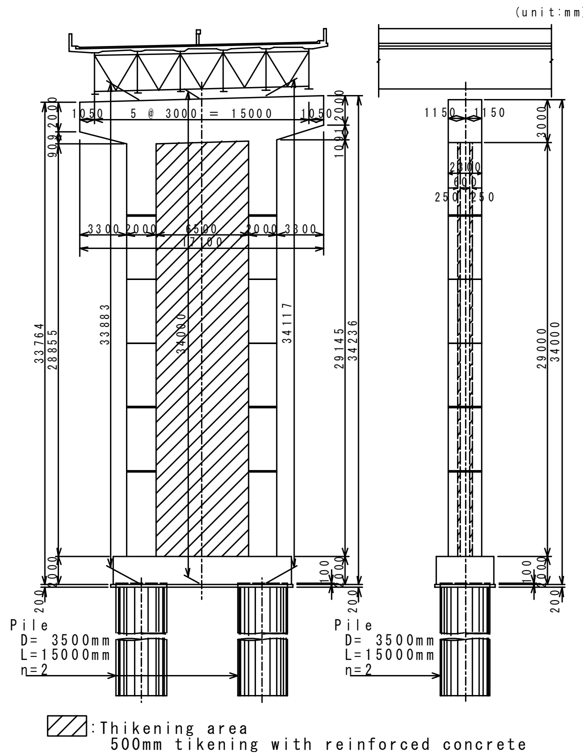

Fig. 6 THICKENING OF WALL AT 3P PIER

4.3. Dampers

Photo 1 shows the dampers which were installed in the 1st Ibi viaduct (at 4A). Since the safety of the bridge at large-scale earthquakes is dependent on these dampers, they are required to have their functions for a long time, and to work certainly in case of any earthquakes. It is necessary to get their condition correctly, and to maintain them in good condition.

It is thought that the dampers would have high durability, since it has been checked that the silicone resin, which is used as the resistance object of the damper, has high chemical stability and durability by the accelerated weathering test. However, the durability of the dampers has not yet checked


Photo 1 DAMPERS INSTALL (4A)

directly. And the dampers had begun to apply to bridges recently, so there are not so many dampers installed for long period. The information about their durability is not enough. Therefore, in order to get reliable information more, we and the maker of the dampers have started the damper's exposure test.

In this test, the small damper of same type installed in the bridge is used. This test damper is designed to have the resistance of 100kN. After its resistance is checked, it will be placed on the open air test site, in order to be exposed to the actual atmosphere such as sunlight and rain. This exposure test is planed to be continued for ten years, and the performance of the test damper is planed to be examined two or three times within this period. This test will show useful data about the durability of the damper. We are going to determine the maintenance plan of dampers, from this data.

5. Conclusion

For seismic reinforcement of the existing bridge, it is very important to know the bridge’s condition. The measures of reinforcement have to be selected according to the situation of the bridge, and from the viewpoint of economical efficiency.

In the 1st Ibi viaduct, as shown in this paper, the viscous dampers, which were expected to reduce the seismic force, were used. It is thought that dampers are effective for some types of bridges, in case the seismic reinforcement of their substructures is difficult like the 1st Ibi viaduct. In the Kobe-Awaji-Naruto Expressway, dampers were applied to no less than four bridges except this bridge.

Maintenance of the dampers is also very important after installation in view of the permanency of the reinforcement effect. However, there is not enough information about the durability of the dampers, and its maintenance plan is not established clearly. Therefore, it is necessary to accumulate data about durability, and to establish the maintenance plan.

References

Japan Road Association; Specifications for Highway Bridges Part 5 Seismic design 1980
Japan Road Association; Specifications for Highway Bridges Part 5 Seismic design 2002
K.Endo, S. Kawabata, S. Ogo; A Study on Seismic Retrofit of the Honshu-Shikoku Bridges Using Isolation and Dissipation Devices, 2007IABSE, Sep. 2007