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SEISMIC RETROFIT OF THE LONG-SPAN PC RAILWAY BRIDGE USING ISOLATION BEARINGS AND DAMPERS

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Abstract

The isolation bearing and/or dampers have been widely used for seismic retrofit of existing railway bridges, particularly for structures where conventional measures such as steel jacketing are not applicable depending on conditions of the construction site. In this paper, one of the typical seismic retrofit works for a long-span PC railway bridge is introduced. This bridge is a five-bay continuous PC box girder whose total span is approximately 450m, and four out of six piers are constructed into the sea. According to the preliminary investigation with regard to the seismic performance, it was clarified that some pier foundations were needed to be strengthened in order to avoid the devastating damage under expected motions, by which securing rapid resume of train operation and enabling quick repair of bridge. However, it was found almost impossible to carry out construction works for the foundation in the sea from the viewpoint of both workability and cost. In order to overcome this problem, application of isolation bearings and dampers that replace the existing steel bearings were investigated, by which the inertial force of the girder will be drastically reduced and damage of pier and foundation are reduced accordingly.

The performance demands were as follows: intermediate four piers (three of which are located at the sea) should be almost intact without retrofitting their foundations, whereas some damage small enough to enable quick repair was permissible for rest of two piers at both ends of bridge. In the retrofitting design, types and specifications of isolation bearing as well as seismic damper were numerically determined so as to meet given performance requirements under estimated earthquake at the site. In a numerical simulation, however, large number of response analyses should be conducted, considering combinations with regard to several seismic motions and structural conditions. A reduced mass-spring-damping model expressing a dominant behavior of the target bride as a whole was proposed in order to carry out numerous number of calculations in an economical manner. The possible specifications of isolation bearing and seismic damper for all piers that met the aforementioned performance requirements were then found by employing the model. It was found that the lead rubber bearing was suitable for all intermediate piers. However, a height between girder and pier top at the abutment is too small to install normal rubber bearings at both ends of bridge. It followed that a slide bearing was alternatively employed that is deployable to narrow space, A viscous damper was also deployed together with the slide bearing to accumulate large displacements and avoid pounding between adjacent viaducts. In addition, seismic dampers were also used for reducing the response of girder and avoid pounding between girder and adjacent abutment. Given specifications of isolators and dampers that met performance demands, detail analysis and verifications were carried out using nonlinear space frame models. It consequently followed that the designed isolators and dampers could successfully meet the performance demands under several severe constraints.

All the construction works including manufacturing bearings and dampers, replacement of bearings and supplemental seismic retrofit have been finished as of end of January, 2020. The construction of supplemental facilities for inspection is under way. Particularly, the large-scale hydraulic jacks were employed to support the heavy girder and replace bearings from steel to lead rubber bearings, without suspending the daily train operations. Details of these construction works until now is also mentioned.

Keywords: long-span PC railway bridge, seismic retrofit, isolation bearings, seismic dampers

1. Introduction

The isolation bearing and/or dampers have been widely used for seismic retrofit of existing railway bridges, particularly for structures where conventional measures such as steel jacketing are not applicable depending on conditions of the construction site. In this paper, one of the typical seismic retrofit works for a long-span PC railway bridge is introduced. According to the preliminary investigation with regard to the seismic performance, it was clarified that some pier foundations were needed to be strengthened in order to avoid the devastating damage and secure rapid resume of train operation and quick repair of bridge under expected motions. However, it was found almost impossible to carry out construction works for the foundation in the sea from the viewpoint of both workability and cost. In order to overcome this problem, application of isolation bearings and dampers that replace the existing steel bearings were investigated, by which the inertial force of the girder will be drastically reduced and damage of pier and foundation are reduced accordingly.

2. Overview of the Retrofitting Bridge

Fig. 1 illustrates the schematic of the Kitaura Port Bridge. It is a total 450m long 5-span continuous PC box girder bridge located on the Shikoku side of the Honshi Bisan Line, which opened in 1988. There are six piers to support girder, described as 1P,2P.. and 6P in the following discussions as shown in Fig.1. Among six piers, 2P, 3P and 4P are constructed at the sea. The ground condition of the target bridge is classified as G4 (normal to soft condition) according to the Japanese seismic standard [1]. All the piers are wall-type piers, and they are on cast-in-place piles or continuous diaphragm wall foundations. The PC box girder is a double-track girder, whose height is 9 m and the total weight is approximately 24,0000 kN. The girder is supported by steel bearings, one fixed bearing at 4P, and the others are movable bearings. In addition, the bearings come with damper stoppers and steel angle stoppers to secure running safety of trains.

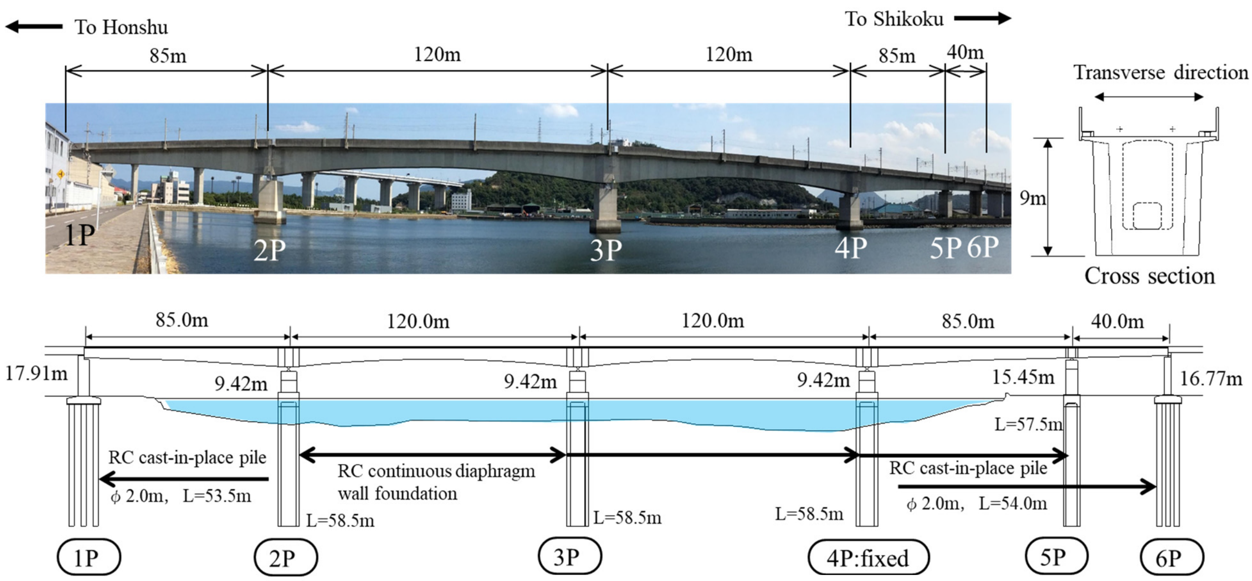
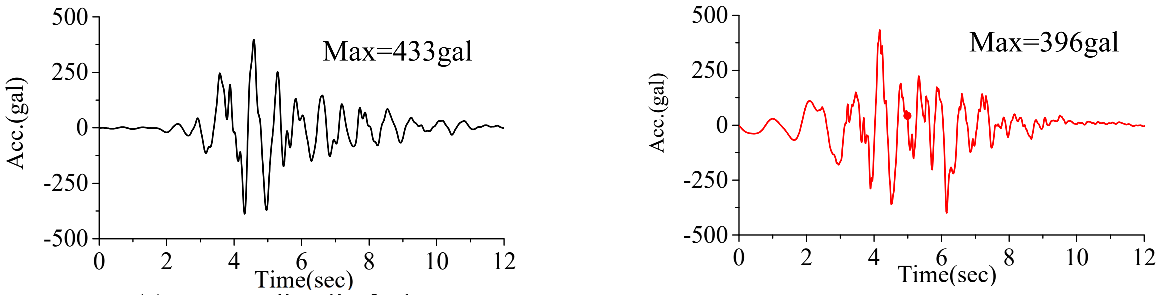


Fig. 1– Overview of the Kitaura Port Bridge

3. Seismic Performance Inspection and Retrofit Plan

3.1 Seismic Performance Inspection for Bridge without Retrofit

The first step of the retrofit was to comprehend the current seismic performance of the bridge under design earthquakes. According to the survey by government, the site would be affected by several possible earthquake, such as motions due to Nankai-Trough, Median Tectonic Line and undetected active faults. The estimation would begin from calculating a set of waveforms at the seismic bedrock caused by these all scenarios. The



(a) Due to dip-slip fault (b) Due to strike-slip fault

Fig.2 – Estimated motions for seismic design

Table 1 – Summary of seismic inspection

Pier	1P					2P, 3P, 4P				
Verification	Damage Level		Failure mode		Cut-off section	Damage Level		Failure mode		Cut-off section
	Pier	Foundation	Pier	Foundation		Pier	Foundation	Pier	Foundation	
Longitudinal direc.	2	1	M	S	NG	1	4	M	M	---
Transverse direc.	2	1	S	S	---	1	4	M	M	---

Pier	5P					6P				
Verification	Damage Level		Failure mode		Cut-off section	Damage Level		Failure mode		Cut-off section
	Pier	Foundation	Pier	Foundation		Pier	Foundation	Pier	Foundation	
Longitudinal direc.	2	1	M	S	---	2	1	M	S	NG
Transverse direc.	2	1	S	S	---	2	1	S	S	---

Failure mode : M = Flexual mode S=Shear (brittle) mode

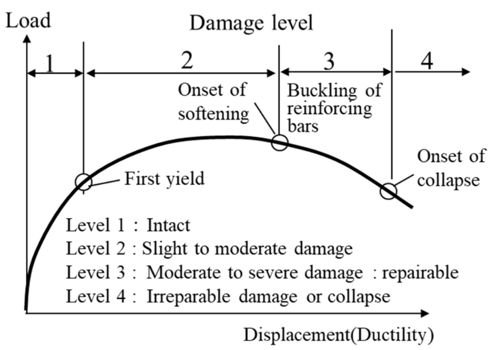


Fig.3 – Definition of damage level

Table 2 – Constraints to be considered for retrofit design

Pier No.	Ductility demand	Max. shear strain of rubber bearing (%)	Clearance between adjacent viaduct(mm)	Note
1P	μ<2∼3	250	310	Clearance between girder and pier top is quite small
2P	μ<1 (intact)		—	at the sea
3P			—	at the sea
4P			—	at the sea
5P	μ<2∼3		—	
6P			105	Clearance between girder and pier top is quite small

corresponding motions at the surface were then calculated considering soil condition nearby the bridge. According to the response spectra with respect to the surface ground motion and a dominant natural period of the bride, it was found that earthquakes due to undetected active faults would cause most significant effect in dynamic response. The active faults were classified as a dip-slip and a strike-slip faults, corresponding synthesized waveforms are shown in Fig.2. These waveforms were assumed as input motions in the following discussions.

Given design earthquakes, a static nonlinear analysis was performed for all piers, and seismic inspections were carried out for responses in both longitudinal and transverse directions. Table 1 summarizes results of the inspection with regard to each pier and foundation. The amount of the damage was expressed by

Table 3 – Specification of lead rubber bearing

Property			2P,3P,4P	5P
Number of bearings			4	2
Shear module of rubber		N/mm^2	1.0	1.0
Dimension	in longitudinal	mm	1900	1400
	in transverse	mm	1200	1400
Thickness of rubber	thickness per layer	mm	28	27
	number of layer	–	5	4
	total thickness	mm	140	108
Lead	diameter	mm	210	205
	quantity	-	4	4
Shape factor S1		-	12.336	12.090
Design stiffness	in vertical	kN/m	8430627	9059225
	in horizontal	kN/m	15296	16926

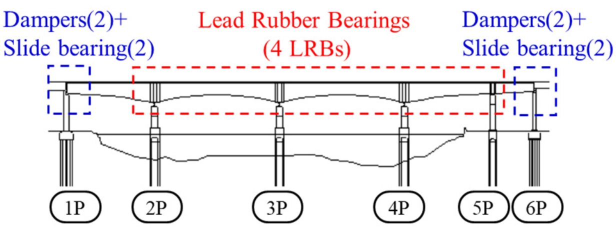


Fig.4 – Final retrofit plan

a damage level specified in the design standard [1]. For reference, typical relation between a damage level and a possible state of the structure is shown in Fig.3. The failure mode, flexural or brittle shear failure, was also evaluated.

According to the inspection results with respect to 1P, 5P, and 6P piers located on the land, damage level 2 was obtained for both longitudinal and transverse directions. However, failure mode of some piers were identified as shear failure that should be avoided to prevent the devastating brittle collapse. To make the matter worse, in 1P and 6P RC piers, severe damages would be expected at the cut-off section. Furthermore, damage level of foundations with respect to 2P, 3P and 4P located at the sea was 4, which would cause significant damage.

On the basis of the preliminary inspections, countermeasures were planned obtained as follows; (1) reinforcements against shear failure for superstructure and foundation in 1P, 5P, and 6P, (2) retrofit at cut-off sections in 1P and 6P, (3) upgrading flexural strength for foundations in 2P, 3P and 4P. Among them, reinforcements for 1P, 5P, and 6P located on the land are feasible, whereas one can hardly carry out retrofit for 2P, 3P and 4P located at the sea due to the workability, economical reason and environmental regulations. Another feasible design should be needed.

3.2 Alternative Retrofit Plan using Isolators and Dampers

The required performance for the bridge after seismic retrofitting as well as constraints for reinforcement works are summarized in Table 2. It was first decided that maximum allowable ductilies for 2P, 3P and 4P in the sea were surpressed below yieldings, by which avoiding the reinforcement of the foundation and ensuring early recovery even after strong earthquake. On the other hand, it was permitted to allow ductility response up to 3 for the 1P, 5P, and 6P on the land, because their restorations after earthquake would be relatively easy compared to works in the sea.

Based on the preliminary inspections and required performances described,it was planned that the existing steel bearings woule be replaced with seismic isolation bearings. It was intended to suppress the interaction force between the girder and the piers, by which avoiding reinforcements of pier and foundation in the sea. It was planned that the existing steel bearings were all replaced by a lead rubber bearings, or LRBs. The geometries and properties of the LRB was designed to meet the performance demands under general conditions such as durability against dead and live loads. The obtained specifications of LRBs for 2P through 5P is shown in Table 3 [2].

In 1P and 6P, however, clearance between the girder and the pier top was too small to employ an general rubber bearing. In order to overcome this difficulty, a thin slide frriction bearing consisting of the PTFE and Teflon was employed as an alternative to isolation rubber bearing.. It should be noted that response displacement of such a slide bearing would tend to be large because only a small amount of energy dissipation is expected. It followed that the bearing response would exceed the clearance between adjacent viaducts in 1P (310mm) and 6P (105mm), and unexpected poundings and damage of girders would be expected. Therefore, for 1P and 6P, supplemental viscous dampers for absorbing displacement were employed together with slide bearings.

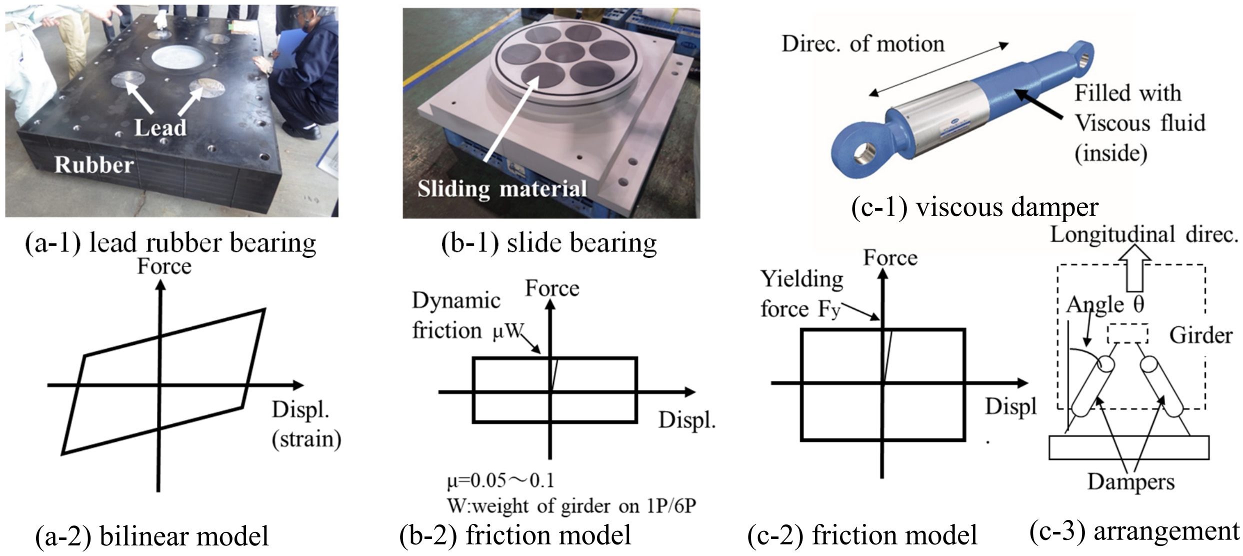


Fig.5 – Isolation bearings and damper for retrofit

Finnaly, countermeasures to be taken for seismic reinforcement was modified as illustrated in Fig.4. The number of the bearings after reinforcement were identical to the original bridge because it was planned to reuse the concrete bed of the existing bearing. Fig.5 shows the devices and their corresponding nonlinear behaviors.

3.3 Outline of Design Procedure

In the following discussion, the specifications of the sliding friction bearing used for 1P and 6P, and the LRB used for 2P to 5P were first designed to meet inspections under general conditions. The parameters of the seismic damper that satisfies the constraints shown in Table 2 were then derived by a trail-and-error basis employing dynamic analysis. Since the nonlinear behavior of the viscous damper used was expressed as shown in Fig.5 (c-2), yielding force (=F_y) was chosen as design parameter. Given required damping force, number of dampers and geometry were then determined referring to the production lineup.

Table 4 shows combination of seismic actions and design parameters to be considered in the design. Among them, coefficient ρ_m is multiplied by the yielding strength of reinforcing bars to consider variation of strength. Similarly, coefficient α_f is multiplied by the strength of horizontal soil strength to consider the uncertainty of soil condition. These factors are designated in the Japanese design standards. The performance demands shown in Table 2 should be met under total 16 conditions shown in Table 4.

Table 4 – Combination of design conditions

Design parameters	Notation	Condition
A: Direction of motion	L	longitudinal
	C	transverse
B: Seismic action	H	dip-slip fault
	V	strike-slip fault
C: Variation of rebar strength(ρm) and soil strength(αf)	1	αf=1.0, ρm=1.0
	2	αf=1.0, ρm=2.0
	3	αf=2.0, ρm=1.0
	4	αf=2.0, ρm=2.0
Total	A×B×C=16 cases	

In order to carry out large number of analyses and find feasible solutions in an economical manner, nonlinear behavior of the total bridge was expressed by a simplified distributed mass-spring model. An optimal damper yielding force that meets demands under 16 design conditions was searched in a trial-and-error basis, described in Chaper 4. The resulting parameter was then employed to a detail space frame model and dynamic analysis was performed. Finally, it was verified that given parameter can meet the performance demands shown in Table 2 using this detail analysis (Chapter 5).

4. Preliminary Design of Isolators and Dampers

4.1 Simulation Model

Fig.6 shows a simplified nonlinear analysis model in which the target bridge was characterized by a multi degrees of freedom system. Each pier-foundation system was expressed by a single degree of freedom model conisiting mass and nonliner spring. All six pier models were then connected to the single rigid body girder model via nonliner springs, expressing LRBs, slide bearings, and viscous dampers.

The partial modeling of the unit pier-foundation system is shown in Fig.7. The mass of the girder included the dead load and live train load (=35kN/m). The pier-foundation weight was given by means of the effective mass, 0.3 times of the total mass. The nonlinar behavior of the pier-foundation system was expressed by nonliear spering. Fig.8 shows nonlinear skeletons with respect to each pier-foundation according to the direction of motion. These curves were obtained via pushover analysis carried out for preliminarly inspections, and given to the nonlinear springs. Since natural period of each pier was obtained by a nonlinear pushover analysis, corresponding 1st and 2nd stiffnesses were given. In addition, hysteretic behavior was set according to the dominant location of damage. The bilinear-Clough model was used if the location of the plastic hinge was in the superstructure (1P, 5P and 6P), whereas biliear model was employed if the damage of concentrated on the foundation(2P, 3P and 4P). In addition, the frequency dependent damping that is inversely proportional to the natural period was given to the pier-foundation system to express the energy dissipation of the soil. These models were designated in the Japanese seismic standard [1].

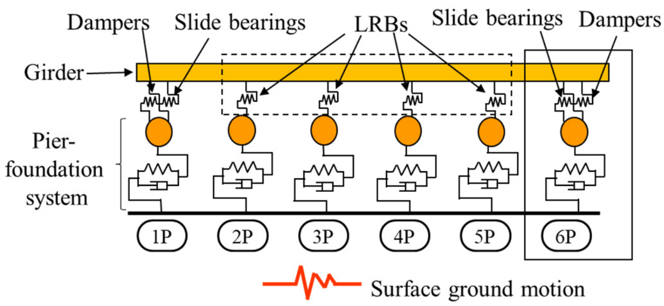


Fig.6 – Generalized model for preliminary design

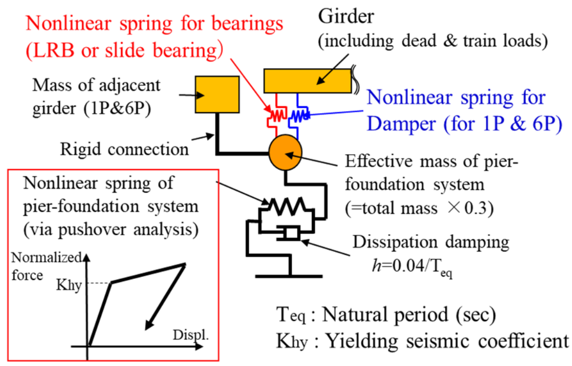
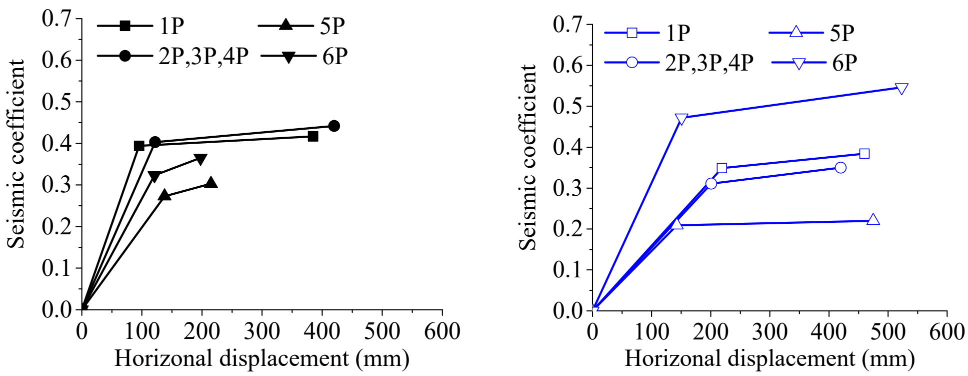


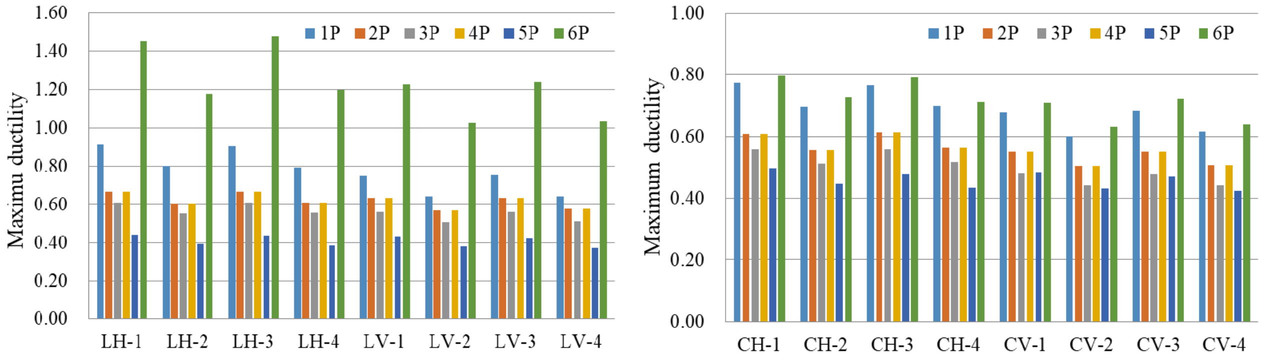
Fig.7 – Detail of each pier-foundation system



(a) longitudinal direction (b) transverse direction

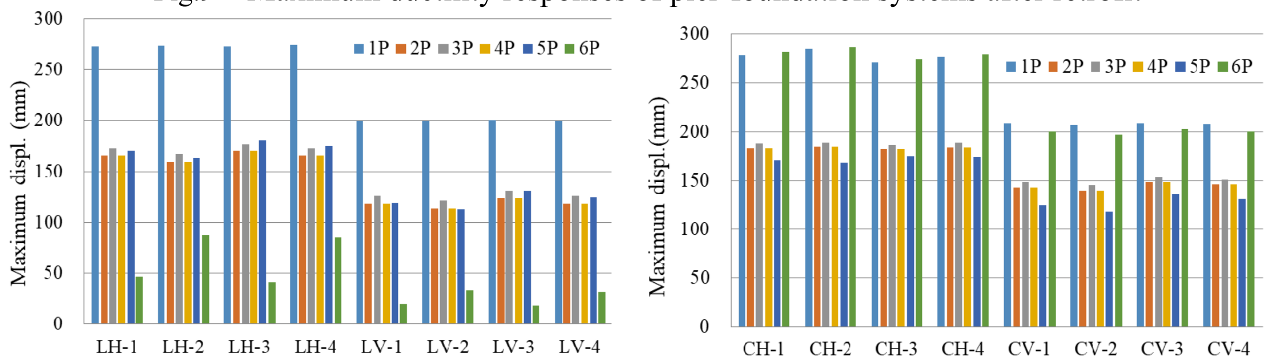
Fig.8 – Nonlinear behavior of piers-foundation system by pushover analysis

The behavior of the LRB was expressed by a bilinear model shown in Fig.5(a), whose parameters were determined from specifications shown in Table 3 [2]. The isolators and dampers were assumed to respond to both longitudinal and transverse directions under strong earthquakes. The nonlinear springs with respect to slide bearings and viscous dampers were expressed by a friciton model as previously shown in Fig.5(b) and



(a)longitudinal direction (b) transverse direction

Fig.9 – Maximum ductility responses of pier-foundation systems after retrofit



(a)longitudinal direction (b) transverse direction

Fig.10 – Maximum displacements of isolation bearings

Fig. 5(c). The friction coefficient of the sliding friction bearing (=μ) was set to 0.1. As for arrangement of dampers, they were inclined from the longitudinal direction with an angle of θ to make them work against planar motions, as illustrated in Fig. 5(c-3). In the following calculations, a large number of dynamic analysis was conducted to find the optimal arrangement angle (=θ) and yielding force (=F_y) of the damper that met demands shown in Table 2.

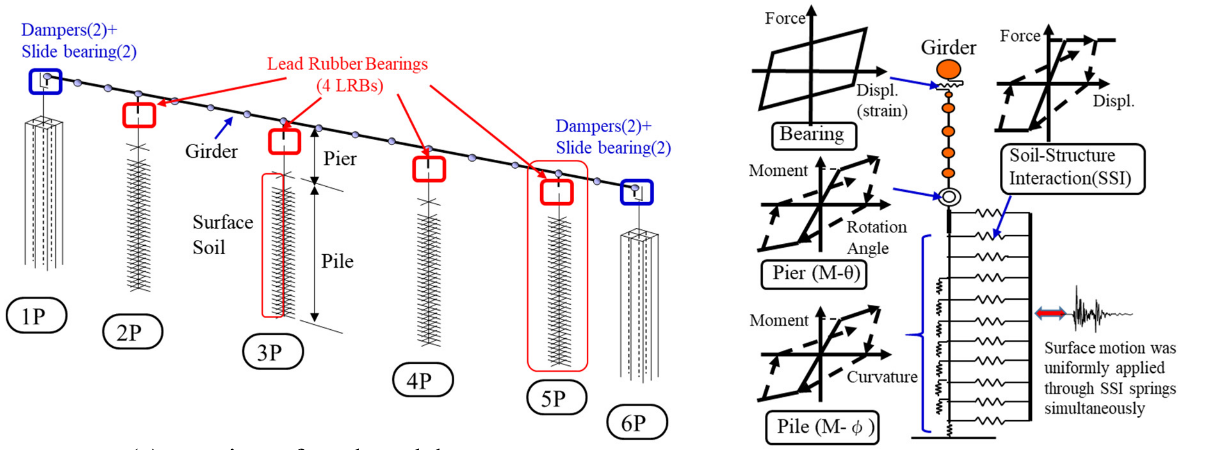
4.2 Results and Discussions

Fig. 9 shows the maximum ductility responses of all piers in case the yielding force of dampers for 1P and 6P was 1000kN and arrangement angles was 30 degree. Fig. 10 shows corresponding maximum displacements of bearings. The horizonal axes in these figures represent the analysis case. Notation of each case is explained in Table 4. For example, LH-1 represents a case that analysis was conducted in a longitudinal direction under strike-slip fault motion, and material variations $\alpha_f = 1.0$ and $\rho_m = 1.0$.

From Fig. 9, it can be seen that the designed damper successfully meet the ductility demands shown in Table 2 in all cases, except that the response in 6P slightly exceeds an elastic limit in longitudinal direction. In addition, it is found from Fig.10 that maximum displacement of the LRBs was below the shear strain limits, and response of slide bearings in 1P and 6P were also surpressed to the clearance between adjacent structures. It was finally concluded that designed isolation bearings and dampers could successfully meet the performance demands under given constraints.

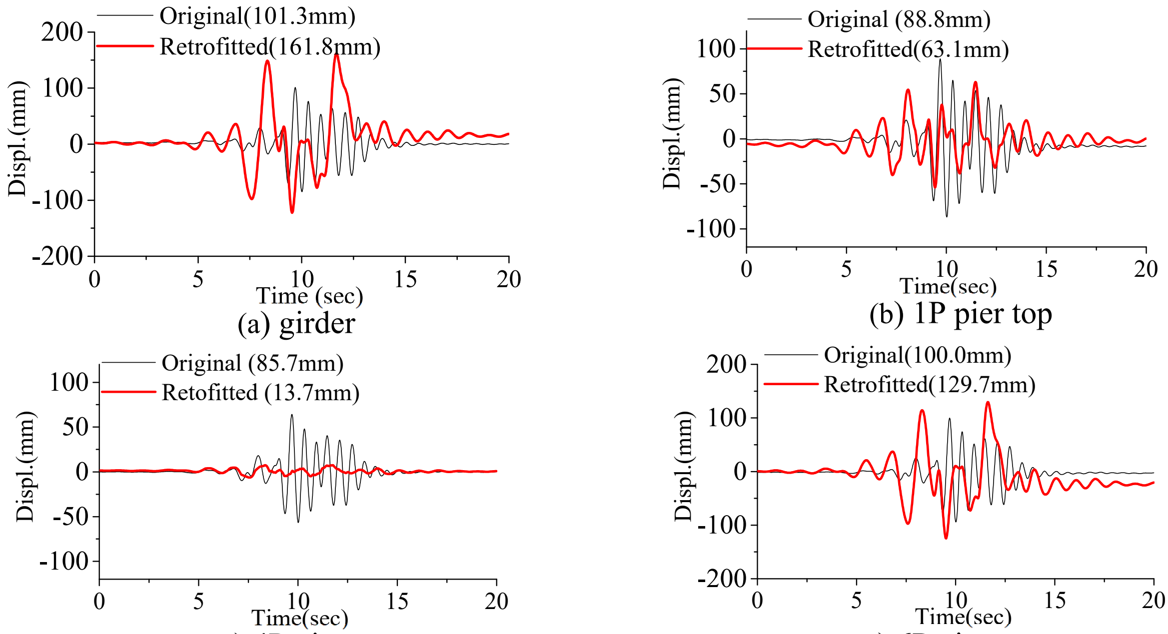
5. Performance Verification using Space-Frame model

On the basis of the preliminary designs mentioned in Chapter 4, more detail analysis and verifications were performed using a three-dimensional nonlinear space-frame analysis. In the following, analysis and



(a)overview of total model (b) mass-beam-spring model of each pier

Fig.11 – Space-frame model for detail analysis and verification



(a) girder (b) 1P pier top

(c) 4P pier top (d) 6P pier top

Fig.12 – Comparison of displacement time history with and without retrofit

verification under $\alpha_f = 2.0$ and $\rho_m = 1.2$ are described as a representative case, that the largest reaction forces would take place among 16 design conditions.

Fig.11 shows overview of the constructed space frame model. In composing the frame model, nonlinear behaviours with respect to pier and pile were expressed by a moment-rotation angle (M-θ) and a moment-curvature (M- ϕ) relations. The girders were rigid elements. The surface ground characters were modeled by nonlinear springs. The dead, supplemental, and live train loads were considered for mass of girder and piers. The structural damping was given by assuming a Rayleigh proportional damping. The list of the natural period and corresponding structural damping were calculated by an eigenvalue analysis and strain proportional damping method, and dominant two modes were selected for fitting to a Rayleigh damping model. According to the eigenvalue analysis, dominant natural period and corresponding total damping in a longitudinal direction were 0.47 seconds and 5.0%, whereas 0.72 seconds and 5.7% in a transverse direction.

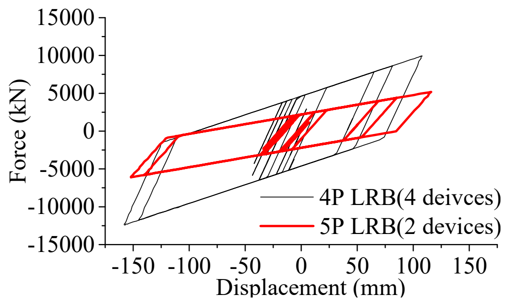


Fig.13 – LRB responses

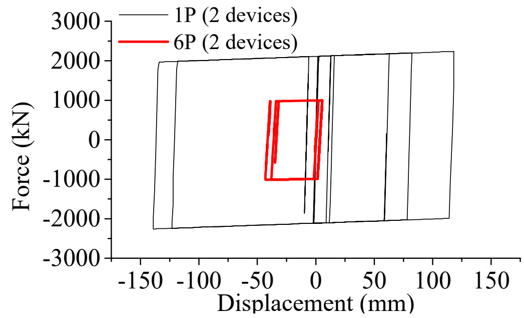


Fig.14 – Slide bearing responses

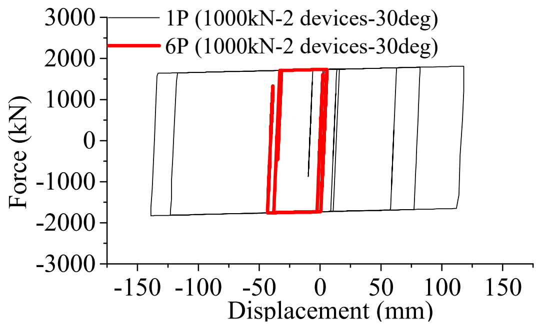


Fig.15 – Damper responses (1P and 6P)

Table5 – Summary of the verification

(a) original (without retrofit)

		Longitudinal direc.		Transverse direc.	
		Damage level	Safety against shear failure	Damage level	Safety against shear failure
1P	Pier	2	0.85	2	**2.04**
	Foundation	1	**1.13**	1	**1.08**
2P-5P	Pier	2	0.96	2	**1.23**
	Foundation	**4**	**1.51**	**4**	**1.03**
6P	Pier	2	0.38	2	0.68
	Foundation	1	0.77	1	**1.91**

(b) after retrofit

		Longitudinal direc.		Transverse direc.	
		Damage level	Safety against shear failure	Damage level	Safety against shear failure
1P	Pier	1	0.54	1	**1.83**
	Foundation	1	0.63	1	0.71
2P-5P	Pier	1	0.20	1	0.36
	Foundation	1	0.26	1	0.23
6P	Pier	1	0.58	1	0.41
	Foundation	1	0.12	1	0.38

The dynamic analysis was then performed by applying input ground acceleration through soil springs simultaneously. Fig.12 illustrate displacements of the girder and piers 1P, 4P, and 6P in longitudinal direction with and without seismic retrofit, under strike-slip fault motion.

From Fig.12(b) and Fig.12(d), it is found that responses of piers 1P and 6P increased after retrofit. It is observed from comparison of Fig.12(a), Fig.12(b) and Fig.12(d) that time histories of piers are similar to that of girder. Since the inertial force of the girder was transmitted to 1P and 6P piers through slide bearings and dampers, it is estimated that the increase of the girder response enlarged those pier displacements. On the other hand, it is observed from Fig.12(c) that a response of 4P pier located at the sea was greatly reduced by using isolation bearings. Similarly, it was confirmed that responses of 2P, 3P and 5P piers in both longitudinal and transverse directions were also decreased.

Fig. 13 shows the load-displacement relation of the LRB introduced into 4P and 5P piers. Fig. 14 and Fig.15 show the hysteretic responses with respect to the sliding friction bearing and dampers employed into both ends of bridge, 1P and 6P piers. It should be noted that reacting force in these relations are expressed as a sum of all devices at each pier. From these figures, it is considered that the decrease in displacement response, especially at 4P pier, was caused by decreasing interacting force at bearings and energy dissipations by means of a seismic isolator. In addition, it is confirmed from Fig. 14 and Fig. 15 that the relative displacement between girder and adjacent viaducts were reduced small enough to meet clearance demands in Table 2. The maximum shear strains with respect to all LRBs were also within the limit value of 250%.

Fig.16 – Overview of retrofit

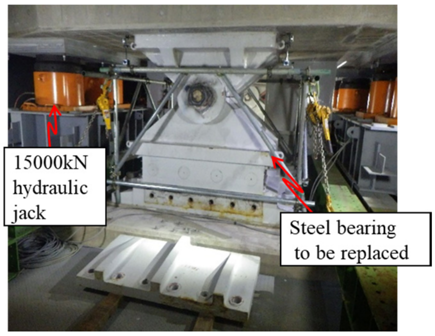


Fig.17 – Hydraulic jacks

Fig.18 – Removal of existing steel bearing

Table 5 summarizes the results of seismic inspections with and without retrofit in both longitudinal and transverse directions. In these tables, amount of possible damages with respect to superstructure and foundation are expressed by a damage level shown in Fig. 3. In addition, safety against shear failure was verified by calculating a verification index that is a ratio of acting shear force to shear failure capacity. The possibility of shear failure would be relatively small if the index is lower than unity.

It is confirmed from Table 5 that the damage level of 2P to 5P foundations, exceeded damage levels 4 in the original bridge, became almost intact by introducing the aforementioned retrofit. In addition, the reduction of the damage of piers also contributed to reduce acting shear force small enough to avoid a brittle shear failure. However, the shear failure 1P pier might still take place in a transverse direction even after introducing isolators and dampers. It was finally determined that the carbon-fiber sheet jacketing was to be employed into 1P pier to avoid shear failure.

6. Retrofitting Works

Since the detail verifications were completed, retrofit for installing isolators and dampers was carried out. One of the significant works was to replace existing steel bearings with new isolation bearings. The outline of the retrofit is described as follows.

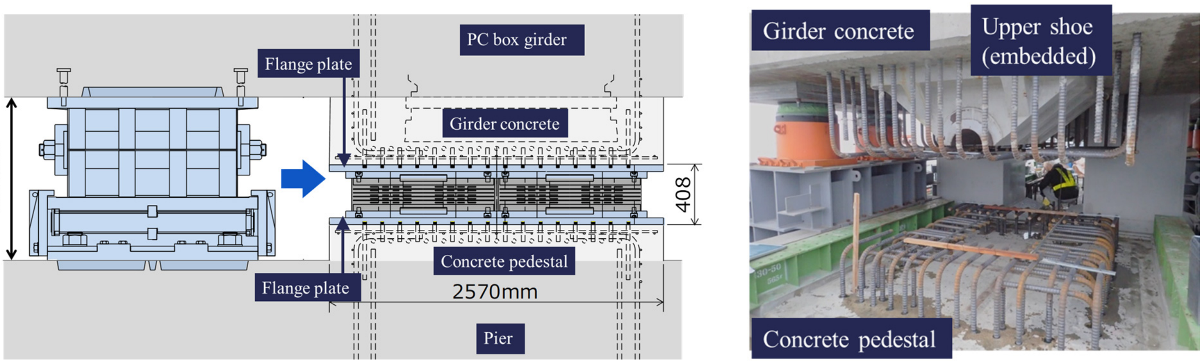
6.1 Preparing Working Environment and Installation of Jacks

Fig.16 shows overview of construction for piers at the sea. The works have done using a barge equipped with 120t crane, since constructing a dock was not permissible so as not to interfere the traffic of ships cruising neaby. Regarding 1P and 6P, there was no enough space to place equipments and materials for construction. Instead, supplemental frames were then constructed in fromt of the pier to support hydraulic jacks.The design maximum load on were approximately 80,591kN, Total eight 15,000kN jacks were installed on top of 2P, 3P and 4P piers, that affords the maximum vertical load of approximately 80,591kN at each pier.

6.2 Construction Management

Fig.17 illustrates hydraulic jacks that were lifting a girder at 2P pier. Since the replacement of the bearings was done without suspending daily train operation, manipulation of hydraulic jacks was performed during the night after train operation ended. The jack’s introduction force was managed in 10% increments with respect to the design load on each pier. The status of operation was monitored by measuring the reacting force and the displacement of the jack.

The amount of lifting a girder at piers 2P to 5P was 3mm, including 2mm for estimated vertical settlement after rubber bearing was installed and 1mm for gap between jack and girder. Actually, the total vertical displacement after all jacks were installed was as small as 0.5mm when a train was passing. On the other hand, sliding bearings at 1P and 6P piers would not deform in vertical direction, so the jack was fixed



(a)plain view of bearings (b)constuction of pedestal and girder

Fig.19 – Replacement of existing steel bearing to lead rubber bearing

(a) existing bearing (removed) (b) lead rubber bearing (c) dampers

Fig.20 – Installation of LRB and dampers

when the vertical load on the existing bearing was totally released. Finally, track measurement was performed after jacks were installed. It was confirmed that no track deformation was found.

6.3 Removal of Existing Steel Bearings

Since the jack-up was completed, the existing steel bearing was removed. Using a wire saw, the lower part of the existing bearing was cut off and removed as shown in Fig.18. The plain view of the replacement of bearings is illustrated in Fig.19. The new LRBs were then installed at the same position where the existing steel bearings had been placed. As shown in the figure, upper part of the steel bearings was left and later embedded in a concrete that was reused for girder concrete for LRB.

6.4 Installation of Bearings and Dampers

After installing girder and pedestal concrete, seismic isolation rubber bearings were installed and released the jacks. In constructing a girder and a pedestal concrete, it was necessary to embed anchor bars on existing girder and piers as shown in Fig.19(b). This anchoring was really a time-consuming work because existing reinforcing bars were not necessarily be precisely placed at locations shown in the design drawings. The place of drilling was then carefully checked every time at the construction site using a rebar probe.

Fig.20 (a) and (b) shows the final installation of the LRB. The vertical settlement of the rubber bearing was almost identical to the estimated design value (=2mm), and the level of the track was almost the same before and after replacement of the bearing. The dampers were also placed to 1P and 6P piers, as shown in Fig.20(c).

As of January 2020, all construction works including manufacturing bearings and dampers, replacement of bearings and supplemental seismic retrofit have been completed. It should be emphasized that such a massive retrofit, for the first time for Japanese railway bridges, was successfully accomoplished without suspending daily train operations.

7. Conclusions

In this paper, the design and construction works of a seismic retrofit for large-span railway PC bridge for the first time in Japan is presented. According to the preliminary investigation with regard to the seismic performance, it was clarified that some foundations were to be strengthened in order to avoid the devastating damage and secure rapid resume of train operation under expected motions. However, it was found almost impossible to carry out construction works for the foundation in the sea from the viewpoint of both workability, surrounding environment and cost. In order to overcome this problem, application of isolation bearings and dampers that replace the existing steel bearings were investigated, by which the inertial force of the girder will be drastically reduced and damage of pier and foundation are reduced accordingly.

In the retrofitting design, types and specifications of isolation bearing as well as seismic damper were numerically determined so as to meet given strict performance requirements under estimated earthquake at the site. In a numerical simulation, however, large number of response analyses should be conducted, considering combinations with regard to several seismic motions and structural conditions. A reduced mass-spring-damping model expressing a dominant behavior of the target bride as a whole was proposed in order to carry out numerous number of calculations in an economical manner. The possible specifications of isolation bearing and seismic damper for all piers that met the aforementioned performance requirements were then found by employing the model. It was found that the lead rubber bearing was suitable for all intermediate piers. However, a height between girder and pier top at the abutment is too small to install normal rubber bearings at both ends of bridge. It followed that a slide bearing was alternatively employed that is deployable to narrow space, A viscous damper was also deployed together with the slide bearing to accumulate large displacements and avoid pounding between adjacent viaducts. In addition, seismic dampers were also used for reducing the response of girder and avoid pounding between girder and adjacent abutment. Given specifications of isolators and dampers that met performance demands, detail analysis and verifications were carried out using nonlinear space frame models. It consequently followed that the designed isolators and dampers could successfully meet the performance demands under several severe constraints.

The retrofit works have successfully carried out accroding to the series of designs. The exisiting steel bearings were removed and new isolation bearings were installed to the same position. As of January 2020, all construction works including manufacturing bearings and dampers, replacement of bearings and supplemental seismic retrofit have been completed. It should be emphasized that such a massive retrofit, for the first time for Japanese railway bridges, was successfully accomoplished without suspending daily train operations.

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