

SEISMIC RETROFIT STUDY OF CABLE-STAYED BRIDGE ON TOKYO-GAIKAN EXPRESSWAY

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Abstract

This paper describes the seismic performance verification and retrofit method examination of a cable-stayed bridge in the Sakitama Bridge. First, the input earthquake motion was specified for use in both the target seismic performance and the bridge verification, while the parts and members of the latter were verified by seismic response analysis. The main tower and caisson foundation that were difficult to evaluate were verified by a nonlinear finite element analysis. Based on the verification results, regions requiring a seismic retrofit were identified, and the retrofit methods were examined.

Introduction

The imminence of large-scale earthquakes, namely the Tokai, Tonankai and Nankai Earthquakes has been pointed out in Japan in the near future. During the three-year period from FY 2005 to FY 2007, the Ministry of Land, Infrastructure, Transport and Tourism has focused on the seismic retrofit of bridges on emergency routes that would play important roles during earthquakes [1]. Under this Three-Year Program, in addition to bridges built before the 1980 edition of the Design Specifications for Highway Bridges was established, special structures and long-span bridges were also designated as targets of the seismic retrofit.

As the emergency transportation route, the Tokyo-Gaikan Expressway would assume vital roles in rescue and relief activities and emergency transportation during a large-scale earthquake (Fig. 1). The Sakitama Bridge on this Tokyo-Gaikan Expressway is located in Bijogi, Toda City, Saitama Prefecture, and crosses the Arakawa River (Fig. 2). The cable-stayed bridge in Sakitama Bridge has a special structure, and was therefore selected for the examination of a seismic retrofit under this Three-Year Program. This paper describes both the seismic performance verification and retrofit method of the cable-stayed bridge in Sakitama Bridge.

The Outline of the Studied Bridge

Figure 3 shows a general view of a cable-stayed bridge in the Sakitama Bridge. This cable-stayed bridge is carrying both the expressway and general road (National Highway Route 298) of the Tokyo-Gaikan Expressway. The two bridges are arranged in a parallel for inbound and outbound routes.

This cable-stayed bridge lays the cables in a multi-fan type in a single plane. It

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Fig.1 Location of Sakitama Bridge



Fig.2 Photo of Sakitama Bridge

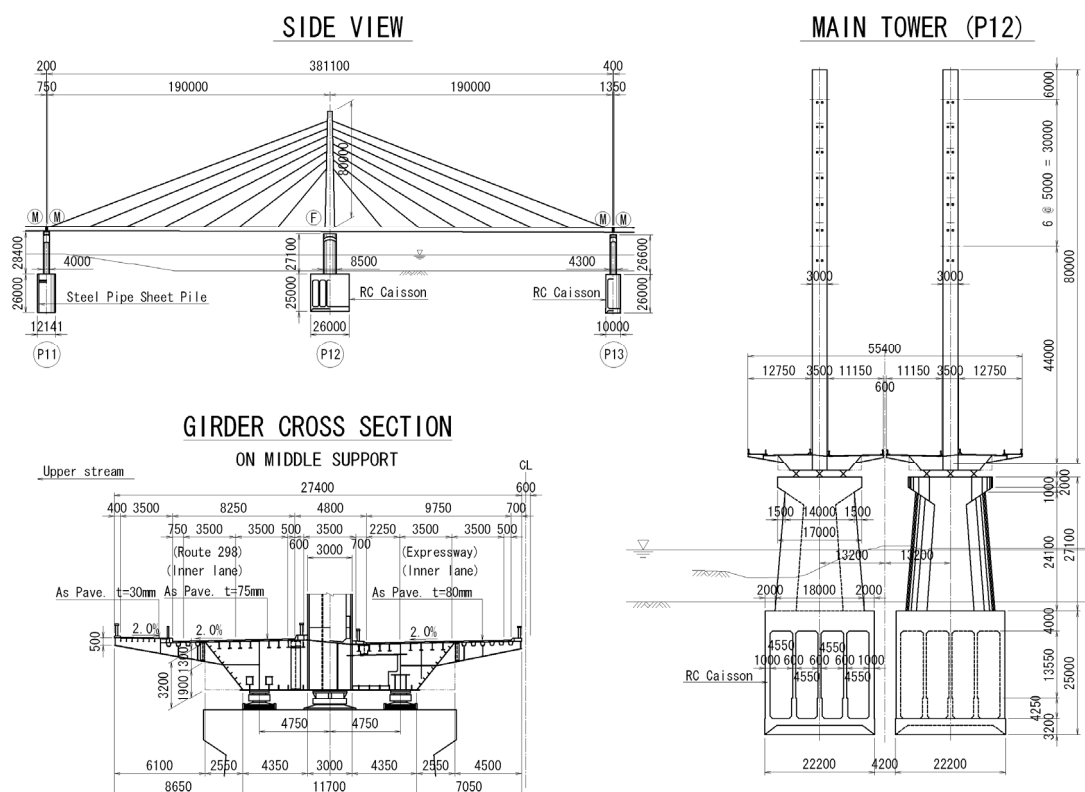


Fig.3 General view of Sakitama Bridge

is 2-span continuous steel cable-stayed bridge with a span length of 190m. The main tower is a single post type and made of steel. The main girder is a steel box girder with 3-cell, with a total width of 26m. The main tower and main girder are rigidly connected, and connected to the pier via the bearing support. The piers from P11 to P13 are all of the RC wall type. The bearing support condition in the longitudinal direction is fixed only on P12 pier, the middle support. A pivot bearing is installed on P12 and pivot roller bearings are installed on P11 and P13, which are the end supports. The bearing support condition in the transverse direction is all fixed, and wind bearings to be used

for fixing the transverse direction are installed on P11 and P13, which are the edge supports. Rocking bearings for controlling uplift are also installed on P11 and P13. The ground is predominantly soft to a depth of about 25m from the surface; mainly comprising a cohesive soil layer, the N value of which is around 3 to 11. The primary natural period of the surface ground is 0.68 to 1.08 sec., and the ground is specified as Ground Type III in the Design Specifications for Highway Bridges. The foundation type of P11 is a steel-pipe-sheet pile foundation, and those of P12 and P13 are pneumatic caisson foundations.

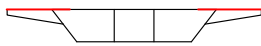
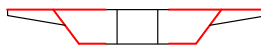
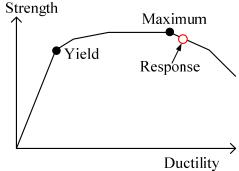
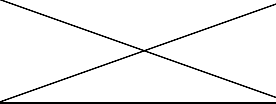
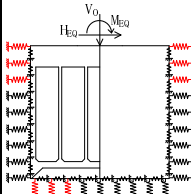
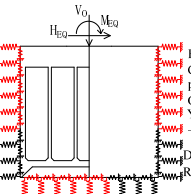
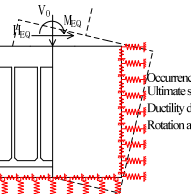
This bridge is based on the design standard specified in the Design Specifications for Highway Bridges of 1980 that was issued before the seismic design standards on highway bridges were revised in 1996, following the Hyogo-ken Nanbu Earthquake in 1995. Therefore, this bridge is designed to ensure seismic performance only for earthquakes (level 1 earthquake motion) highly likely to occur during the service life, and may not potentially satisfy the seismic performance for earthquakes (level 2 earthquake motion) not likely to occur but of high intensity. The cable-stayed bridge for the inbound route has been in service since 1992, and that for the outbound route since 1998.

Seismic Retrofit Basic Policy

Based on the Seismic Design Edition of the Design Specifications for Highway Bridges [2] and the Three-Year Program of Seismic Retrofit of Bridges on Emergency Routes [1], the target seismic performance for the level 2 earthquake motion incurred by the cable-stayed bridge in Sakitama Bridge requires that it should sustain only limited damage from level 2 earthquake motion, and be capable of swiftly reverting to its main function as an emergency road. The performance is intended to ensure safety against the risk of the bridge collapsing, and facilitate the traffic of emergency vehicles for disaster recovery activities after earthquakes. The performance is also intended to allow ordinary vehicles to use the road only with emergency repairs, and allow permanent repairs to be made while the bridge is in service.

To meet the above-mentioned requirements of the target seismic performance, the critical state of the structural elements comprising the bridge must be set. Table 1 shows the division of the damage level of each member and the required performance for level 2 earthquake motion. The main girder is intended to avoid local buckling on the deck plates in the driving lanes of emergency vehicles, and the floor system that supports the deck plates is intended to yield only slightly and remain strong enough to bear the load of the emergency traffic. The main tower is intended not to succumb to local buckling, but to yield slightly in its marginal section, and remain strong enough to bear the load of emergency traffic. The cables are intended neither to succumb to falling away due to loss of cable tension, nor break when subject to tensile force. The bearing is intended to maintain the force applied to the component parts below the maximum strength. The pier is intended to tolerate major ductility but to confine it to an extent facilitating prompt repairs. The foundation is intended to tolerate ductility provided the ductility limits deformation to a level that is not destructive to the overall safety of the bridge system.

Table.1 Classification of damage level of bridge members for level 2 earthquake

Bridge members		Degree of damage for structure		
		Slight damage ←————→ Severe damage		
		Damage level I	Damage level II	Damage level III
Main girder	In plane derrection	A state that occurrence stress degree slightly exceeded a yield stress degree	A state in the domain that was stable as for the strength and the ductility although the plasticity spread	A state that a strength as the main beam system has begun to deteriorate
	Out plane derrection	A state that the deck plate which stretched caused local buckling, and the box girder inside slightly yield	A state that local buckling and plasticity progressed to the box girder inside	A state that a strength as the main beam system has begun to deteriorate
				
Main tower		A state that occurrence stress degree slightly exceeded a yield stress degree, But local buckling at the elastic level does not occur	A state in the domain that was stable as for the strength and the ductility although the plasticity spread	A state that a strength as the main tower system has begun to deteriorate
stay cable		A state that tensile stress degree exceeded a yield stress degree (0.7% elongation strength) in one cable $\sigma > \sigma_y (= \sigma_u / 1.4)$	A state that tensile stress degree exceeded a yield stress degree (0.7% elongation strength) in plural cables $\sigma > \sigma_y (= \sigma_u / 1.4)$	A state that a cable was falling away by tension loss in plural cables, and tensile stress degree exceeded break strength $\sigma > \sigma_u$
Bearing support		A state that occurrence horizontal force exceeded the smallest yield strength in their component parts $P_y < P \leq P_u$		A state that occurrence horizontal force exceeded the smallest maximum strength in their component parts $P > P_u$
RC pier		A state that cracking of the concrete and yield of the reinforcement occur, but have still redundancy for maximum strength $1.0 < \mu_r \leq 1.2$	A state that cracking of the concrete and yield of the reinforcement spread, but have stability for the strength and the ductility $1.0 < \mu_r \leq \mu_a$	A state that a strength as the pier system has begun to deteriorate $\mu_r > \mu_a \quad S > P_s \quad \delta_R > \delta_{Ra}$
Foundation (P12,13 : caisson) (P11 : Steel pipe sheet pile)		A state that plasticity of the around ground and foundation slightly float 	A state that foundation yield, and foundation floating and plasticity of the around ground spread through 	A state that foundation strength deteriorated and lost stability of the superstructure 

* The bold frames show that it is a limit state to satisfy required performance.

Figure 4 shows an outline of the seismic retrofit examination on the cable-stayed bridge. After specifying the target seismic performance for level 2 earthquake motion, the seismic performance verification of the current structural system is performed via seismic response analysis. Regarding the input earthquake motion, we initially examined the need to prepare site waves taking regional characteristics into consideration. Consequently, it emerged that there was no information concerning the active fault near the Sakitama Bridge that required realistic consideration of an earthquake. Therefore, we decided to use the standard waves (type II of level 2 earthquake motion) in the Design Specifications for Highway Bridges established based on the Hyogo-ken Nanbu Earthquake of 1995. For plate-boundary earthquakes, we also decided to use the standard waves (type I of level 2 earthquake motion) in the Design Specifications for Highway Bridges because we considered that this bridge would be affected mainly by an inland direct strike type earthquake, and that no effect

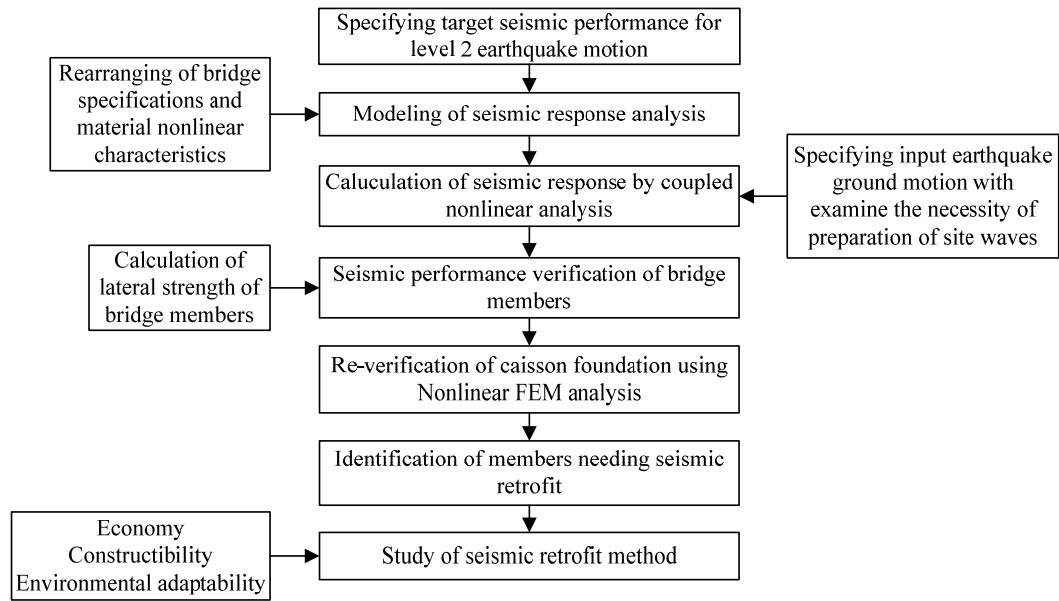


Fig.4 Flow of seismic retrofit design

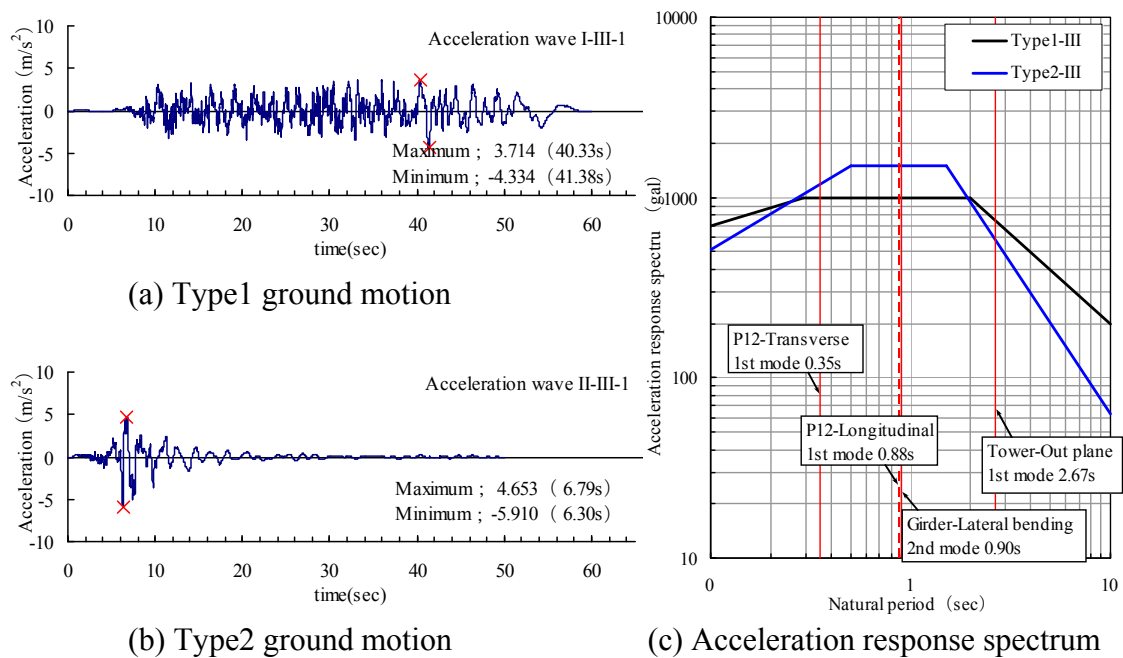


Fig.5 Design earthquake ground motion

could be expected from the preparation of site waves. Figure 5 shows the time history response waveform and acceleration response spectrum of the representative 1 wave of types I and II.

Seismic Response Analysis Model

Before performing a seismic response analysis, we calculated the lateral strength of the bearing support, pier and foundation, and estimated the damage sequence. As shown in Fig. 6, the lateral strength of the caisson foundation of P13 in

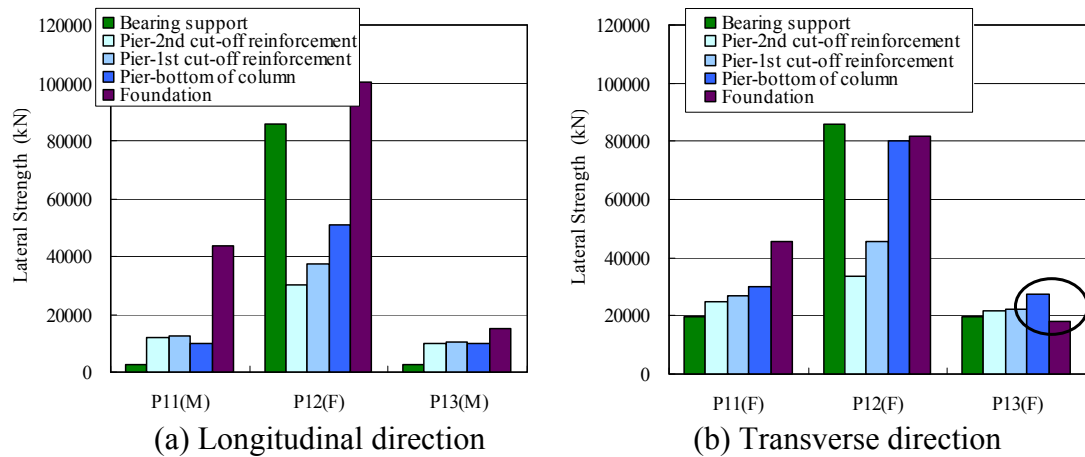


Fig.6 Comparison of lateral strength capacity of each members

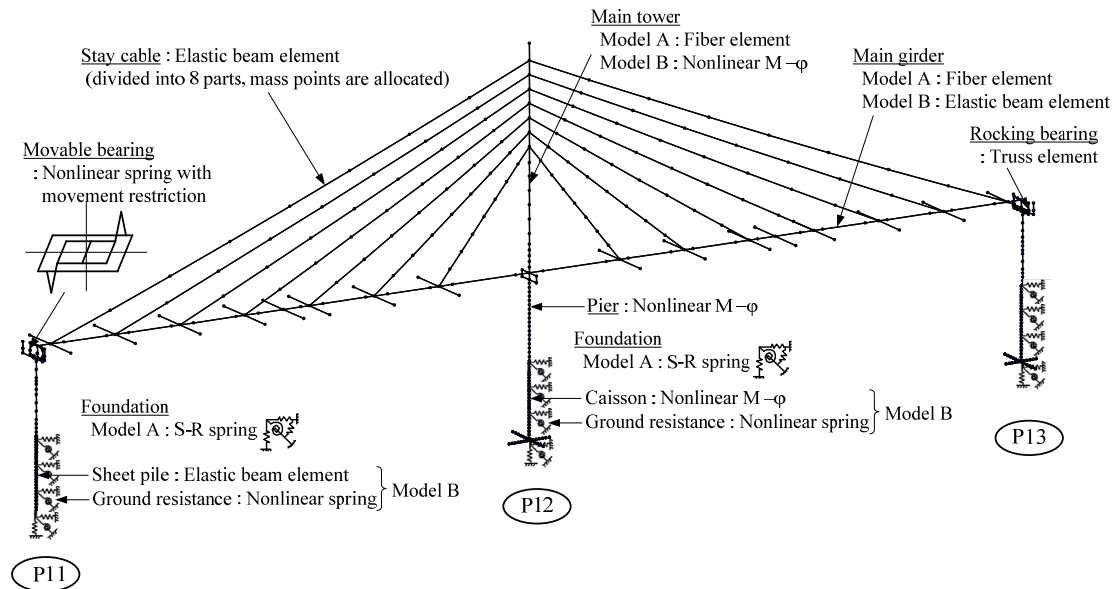


Fig.7 Analytical model

the transverse direction is lower than that of the pier, and the surface ground is soft. Accordingly, it is necessary to perform seismic performance verification of each part and member via seismic response analysis that takes account of the ductility of the foundation. As for the main girder made of orthotropic steel deck 1-box girder, the overhang of the deck is long, and number of ribs of the sidewalk is small. It is therefore highly probably that the main girder will buckle by an earthquake in the transverse direction, and it is advisable to use fiber elements that allow us to properly consider the nonlinearity of the material in complicated sectional form. However, the calculation load of the analysis that takes account of the plasticity of both the foundation and main girder is significant, and the calculation result is complicated. As shown in Fig. 7, we therefore decided to perform seismic response analyses on two cases; one for the case where the foundation is assumed to cause no ductility (Model A), and the other case where the ductility of the foundation is taken into consideration (Model B).

In Model A, the foundation is replaced with a sway-rocking spring, and the main girder is modeled with a fiber element; likewise the main tower and pier. Each

cable is subdivided into 8 parts, and mass allocated to take into account the vibration of cable itself. The rocking bearing is modeled with a truss element, and resistance in the longitudinal direction is taken into account. To take ductility of the foundation in Model B into consideration, the caisson foundation is a nonlinear girder element $M-\phi$ model, and ground resistance is modeled with a nonlinear spring. The main tower and pier are also $M-\phi$ models and thus the same as the foundation. To restore their force characteristics, the main tower is a movement hardening type tri-liner model, and the pier and caisson foundation are degrading type tri-liner models, while the main girder is an elastic beam element. The movable bearings on P11 and P13 are nonlinear springs that take account of both frictional resistance and movement restriction.

The material and geometrical nonlinear dynamic analysis of these Models A and B was performed using input seismic motion shown in Fig. 5. Rayleigh damping is used for viscous damping in a dynamic analysis. Rayleigh damping is set to include main modes by the natural vibration analysis performed in advance (Fig. 8). The Newmark- β ($\beta=0.25$) method is used as the numerical integration. The computation time interval is 0.002 seconds.

The main girder, and the cable significantly affected by the ductility of the main girder are verified using the result of Model A, while other parts are verified using that of Model B.

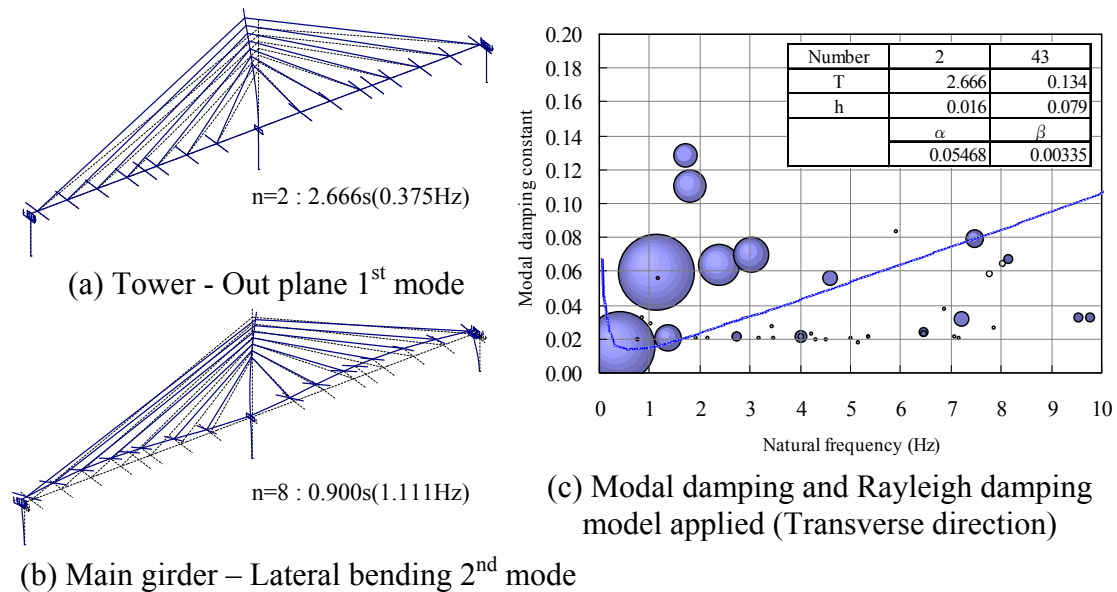


Fig.8 Natural vibration modes and modal damping (Model A)

Seismic Performance Verification of Members

This section describes the details of the seismic performance verification performed using the seismic response that was calculated by material and geometrical nonlinear analysis, and also using either the load-carrying capacity or deformation performance of the bridge members.

The main girder was significantly affected by the earthquake in the transverse direction. The result of Model A that used fiber elements showed local ductility in both the sidewalk deck plate and the box girder, up to about $4\epsilon_y$, and $2.5\epsilon_y$ respectively (Fig. 9). However, the nonlinear finite element analysis results suggested that the undulation

of the road surface after local buckling was so small that the serviceability of the road could be ensured after earthquake (Fig. 10). Consequently, the damage level was evaluated as I, and no retrofit was deemed necessary.

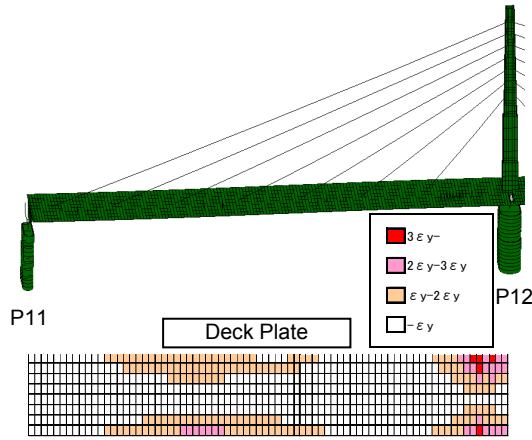


Fig.9 Strain distribution of deck plate (Dynamic analysis result by Model A)

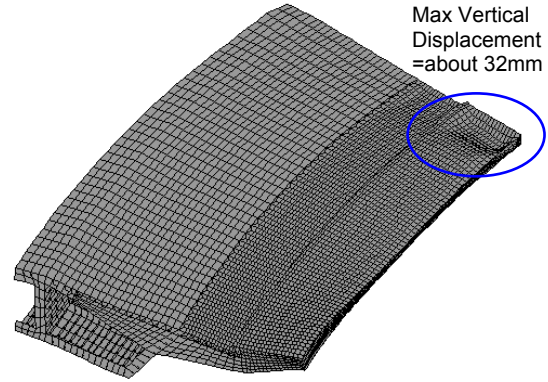


Fig.10 Deformation of deck plate (Nonlinear FEM analysis result)

The cables were affected predominantly by an earthquake in the longitudinal direction, however, the maximum tensile stress caused by earthquake was 773N/mm^2 , namely well below the 0.7% elongation strength $\sigma_y (=1160\text{N/mm}^2)$ (Fig. 11). The minimum tensile stress was 151N/mm^2 , and the cable has no tension loss. Accordingly, there is no damage.

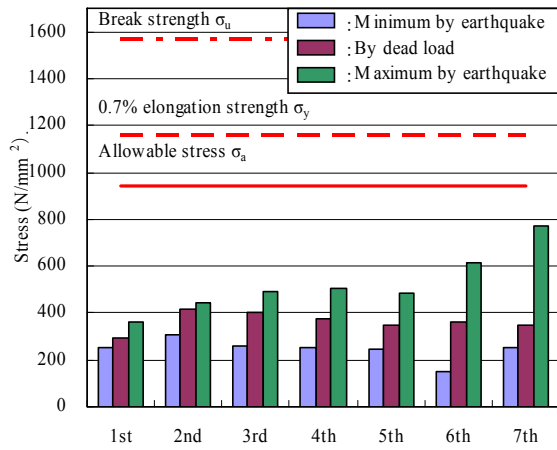


Fig.11 Performance verification result of stay cables (Longitudinal direction, Type2 earthquakes)

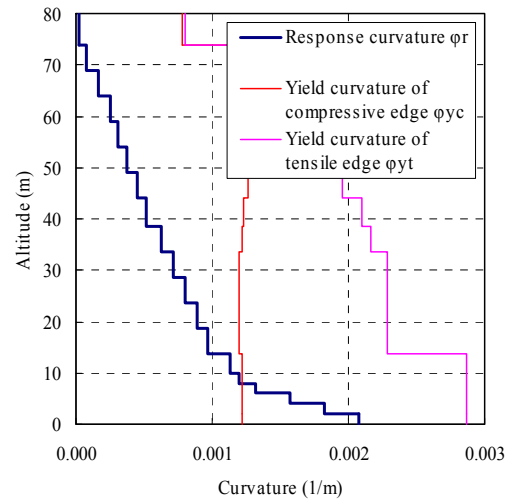


Fig.12 Curvature distribution of tower (Transverse direction, Type1 earthquakes)

The main tower was affected by a type I Earthquake in the transverse direction such that it caused local buckling in the elastic region within the range of about 26m from the foundation. For this reason, longitudinal ribs are added to control the width-thickness ratio parameter and keep it within the limiting value specified in the Design Specifications for Highway Bridges. Assuming that the parameter was

improved, the post-yielding behavior was observed, whereupon the tower was seen to show slight ductility within a range of about 8m from the foundation (response plasticity rate $\mu_r=1.38$) as shown in Fig. 12. The P- δ curve of the main tower was calculated by nonlinear finite element analysis. The results showed that the tower lacked toughness of the SM570 high strength steel, and that its ductility capacity was evaluated as around 1.79 (Fig. 13). It was so close to the response value and difficult to evaluate the damage as level I, consequently, ductility enhancement measures were deemed necessary to reduce the deformation.

Table 2 shows the result of verification of the bearing supports. The maximum response horizontal force of the pivot bearing on P12 exceeded the yield strength, however, it was lower than the maximum strength. Consequently, the damage level was evaluated as level I, and the pivot bearing was deemed to satisfy the required performance. The maximum horizontal displacement of the pivot roller bearings on P11 and P13 exceeded the movement capacity, and the upper bearing collided with the stopper. However, the maximum horizontal force generated on the stopper was less than the maximum strength, hence no retrofit was deemed necessary. The maximum tensile force generated on the rocking bearing was less than the yield strength, hence the rocking bearing was to cause no damage. The maximum horizontal force generated on the wind bearing exceeded the maximum strength, hence suggest the potential for

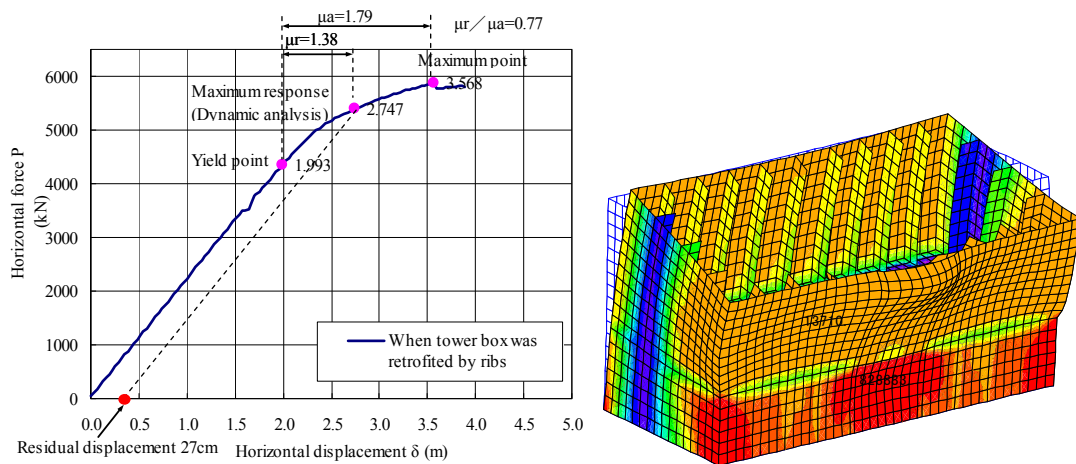


Fig.13 Nonlinear FEM analysis result of tower (Transverse direction, Type1 earthquakes)

Table.2 Performance verification result of bearing supports

Bearing support	Support No.	Response quantity	Seismic direction	Maximum response	Capacity		Damage level
					Yield strength	Maximum strength	
Pivot bearing	P12	Force (kN)	LG	56737	50460	85782	I
			TR	53935			I
Pivot roller bearing	P11	Displacement (mm)	LG	110	110	230	I
		Force (kN)	LG	2565	1512	2570	
	P13	Displacement (mm)	LG	106	110	230	0
		Force (kN)	LG	0	1512	2570	
Rocking bearing	P11	Tension (kN)	LG	14476	20937	27561	0
	P13		LG	6035			0
Wind bearing	P11	Force (kN)	TR	19822	11520	19584	III
	P13		TR	17621			III

the components to be broken and moved laterally. If the main girder slides laterally, loads carried by the rocking bearing sharply increases, thus causing the rocking bearing to break. Then, uplift of the main girder may be occurred, and the bridge would not be used for emergency vehicles. For these reasons, the damage level of the wind bearing was evaluated as level III, and a retrofit was deemed necessary.

Figure 14 shows the maximum response force distribution of the pier that corresponds to its failure mode. This figure shows the response distribution of both Models A and B together with that of the foundation. In an earthquake in the longitudinal direction, piers P 11 and P 13 caused ductility to their bases solely by the inertia force of their own weight. In an earthquake in the transverse direction, the maximum shear force of the entire pier, including the cut-off of longitudinal reinforcement at mid-height, exceeded the shear strength, hence the high potential for the piers to succumb to shear failure. Meanwhile, P12 experienced shear failure mode in both longitudinal and transverse directions, with the shear force exceeded its

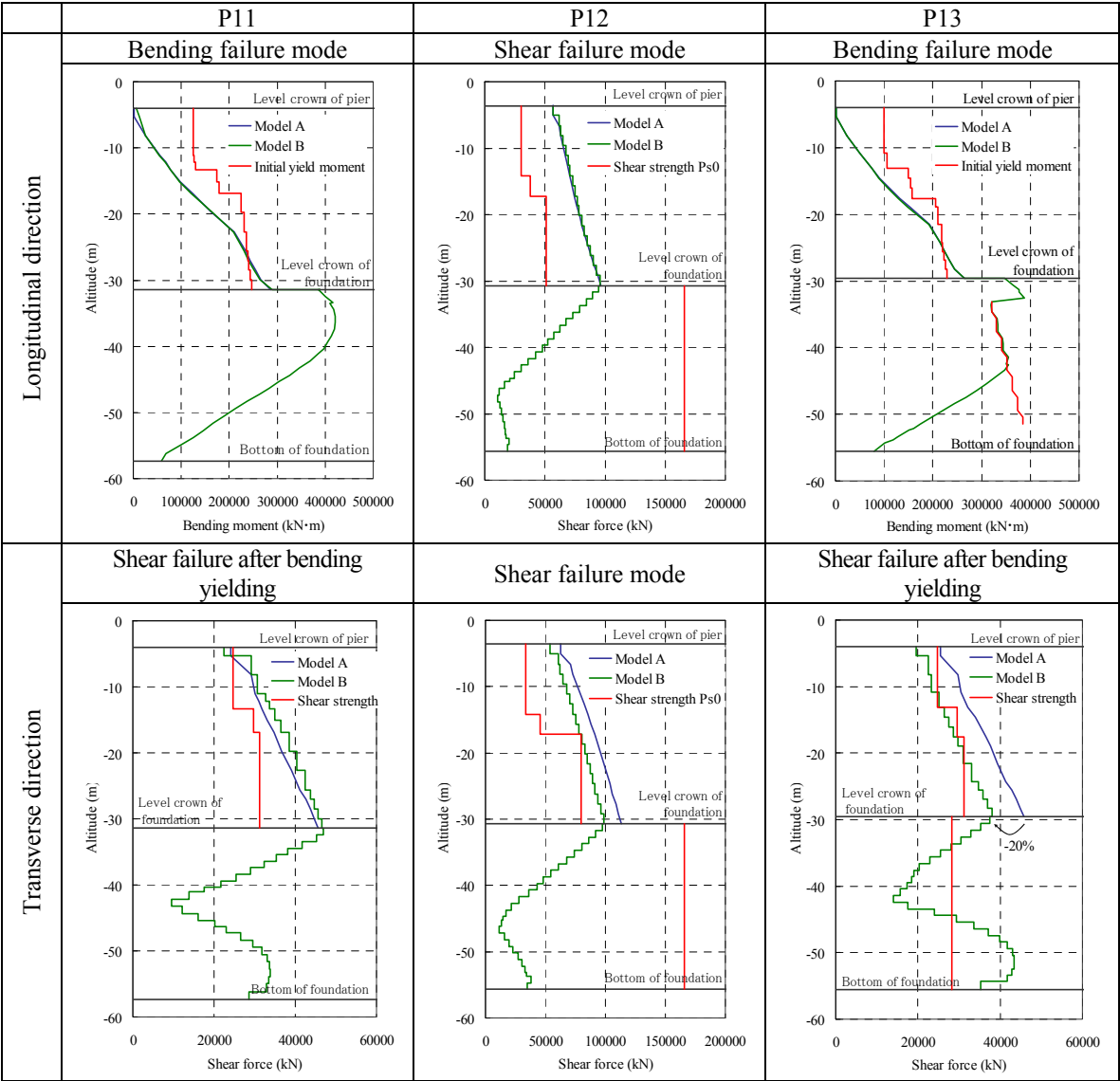


Fig.14 Maximum section force distribution of pier (Type2 earthquakes)

maximum shear capacity in both directions. Based on these results, all piers were evaluated as level III, and some retrofits were deemed necessary. As shown in the shear force distribution in the transverse direction of P13, the maximum horizontal force acting on the crest of the foundation diminished by around 20% in cases where the ductility of the foundation was taken into account. Consequently, such latter cases were considered to be closer to the actual behavior. It was decided to use the value of this case as the acting force to be used to verify the foundation.

Table 3 shows the verification result of the caisson foundation. The verification was performed by a lateral strength approach; using the acting force of the crest of the foundation obtained by a dynamic analysis. The verification of the caisson foundation of P12 showed that the shear force of its members exceeded the shear capacity, although satisfying the stability verification as the foundation. During the verification of the caisson foundation of P13, the shear force exceeded the shear capacity, and its ductility demand of about 27.0 substantially exceeded the ductility capacity of 5.9. The circumferential rebar of the sidewall of the caisson foundation of P13 is the main reinforcement, and bars are arranged using D19@300, while the reinforcement D16@300 in the longitudinal direction is very low. The verification of hollow RC section with low reinforcement based on the beam theory is simple, however, this verification may become over conservative. Therefore, the verification was repeated using nonlinear finite element analysis (see the subsequent section). Meanwhile, the steel-pipe-sheet-pile function of P11 satisfied both the stability and member verifications.

Table.3 Performance verification result of caisson foundations (Transverse direction, Type2 earthquakes)

Performance verification item				P12 caisson			P13 caisson		
				Response		Capacity	Response		Capacity
Stability verification	Ductility demand of foundation			3.022	<	6.000	26.956	>	5.900
	Plasticity domain rate of foundation front		%	54.0	<	60.0	55.8	<	60.0
	Float area rate of foundation bottom		%	35.8	<	60.0	0.0	<	60.0
	Rotation angle of level crown of foundation		rad	0.012	<	0.020	0.039	>	0.020
Verification of members	Vertical section of side wall	Shear	kN	77912	<	162837	52817	>	28727
	Horizontal section of side wall	Bending	kN·m	1277	>	1239	1709	>	1212
		Shear	kN	1153	<	1331	1153	>	411

Figure 15 shows a summary of the verification results showing both the evaluation results of the damage levels of this bridge, and the locations where the required performance were not met when exposed to an earthquake in the transverse direction

Seismic Performance Re-verification of the Caisson Foundation

Figure 16 (a) shows the FE model of the caisson foundation of P13 that was verified in detail using nonlinear finite element analysis. On the FE model, the caisson foundation was modeled with a solid element, and the reinforcement was modeled with a rebar element. The surrounding soil was modeled with a nonlinear spring element, which was arranged at each joint on the peripheral surface. To reduce the computation,

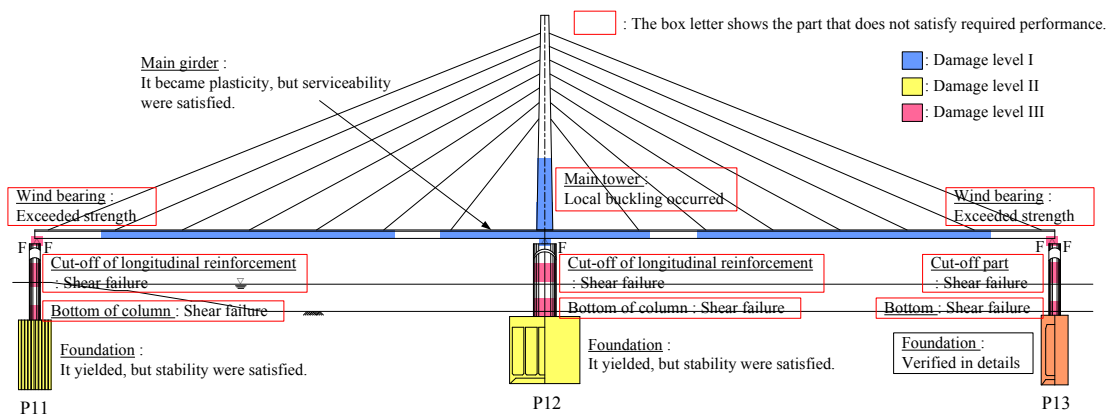


Fig.15 Summary of performance verification result (Transverse direction)

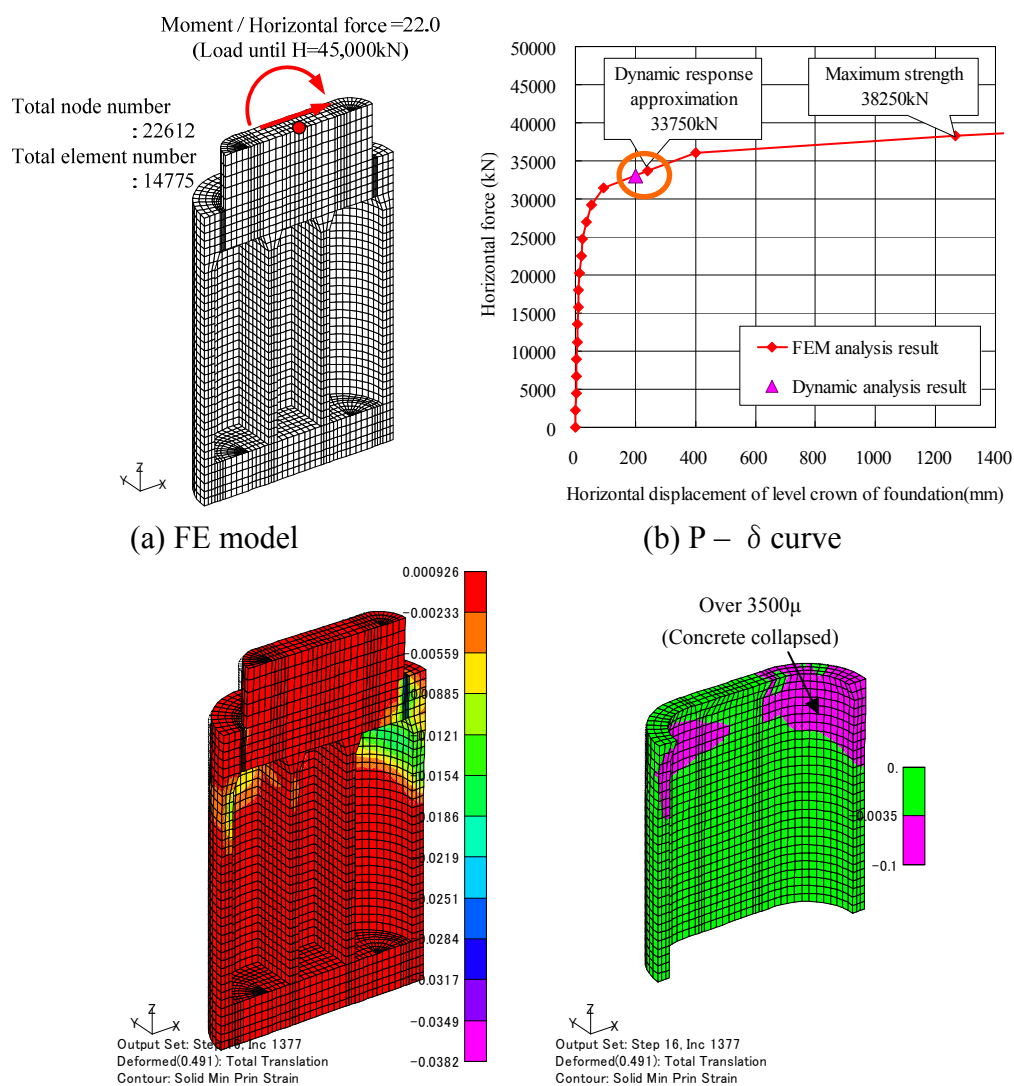


Fig.16 Nonlinear FEM analysis result of P13 caisson (Transverse direction, Type2 earthquakes)

a half model was used for the longitudinal direction. The model used in the analysis involved a 5m portion of the pier and the rigid surface on top of the pier, and the force obtained from the dynamic analysis was applied gradually to the center. The analysis software used for this analysis is the general-purpose finite element program ABAQUS Ver. 6.5.

Figure 16 (b) shows the P- δ curve obtained from the analysis. Stability verification was performed at the loading step corresponding to the dynamic analysis response of approximately 200mm, the horizontal displacement of the foundation crest. The result showed that the ductility demand of 6.4, although slightly exceeding the ductility capacity of 5.9 that included a safety factor of 1.8, did not reach the ultimate ductility factor of 9.8. The dynamic analysis response value was on the upward gradient region, and assuming that the horizontal strength of 38,250kN were considered, there is 13% margin remained until the strength. Therefore, although the foundation caused plasticity, it was considered the safety was maintained.

The members were checked for the loading steps that corresponded to the dynamic analysis response value. As shown in Fig. 16 (c), stress concentration associated with bending in the upper corner of the sidewall occurred. The right figure shows the minimum principal strain of the concrete expressed with the contour in two colors, when the ultimate strain was 3,500 $\mu\epsilon$. This stress concentration exceeded the ultimate strain, and there was a possibility of causing spalling of cover concrete, and buckling and expansion of reinforcement. However, the dead load analysis performed on a model with no members in the circular area of the sidewall revealed that the stresses on the linear section of the side and partition walls were approximately half the allowable stress. Based on this result, it was considered that the members were able to support the dead load caused by the level 2 earthquake motion.

As described above, the caisson foundation was reevaluated by nonlinear finite element analysis, and the result showed that it was able to satisfy the stability requirements. However, the validity of this analysis method has not yet been confirmed by experiments. PWRI is now conducting experimental research into the caisson foundation of low steel reinforcement.

Areas Requiring Retrofit and Retrofit Details

Based on the seismic performance verification results described above, Fig. 17 shows a summary of the areas requiring retrofit and the details.

As for the main tower, its width-thickness ratio parameter had to be improved, and increasing flexural strength at its tower base is necessary. For these reasons, we adopted a strengthening method involving installing stiffing ribs bolted on the exterior surface and these be continuous at cross ribs and diaphragms.

For the wind bearing, two methods were compared: 1) replacing the bearing, and 2) installing a structure limiting excessive displacement. Finally the method involving the replacement of only the upper bearing was adopted from an economic reason.

Regarding the piers, P11 and P13 incurred bending damage solely due to the inertia force due to their own weight, making it difficult to adopt retrofit methods by making use of dispersion of inertia force and seismic isolation. Consequently, the RC piling method was adopted for all three piers. For its construction method, a

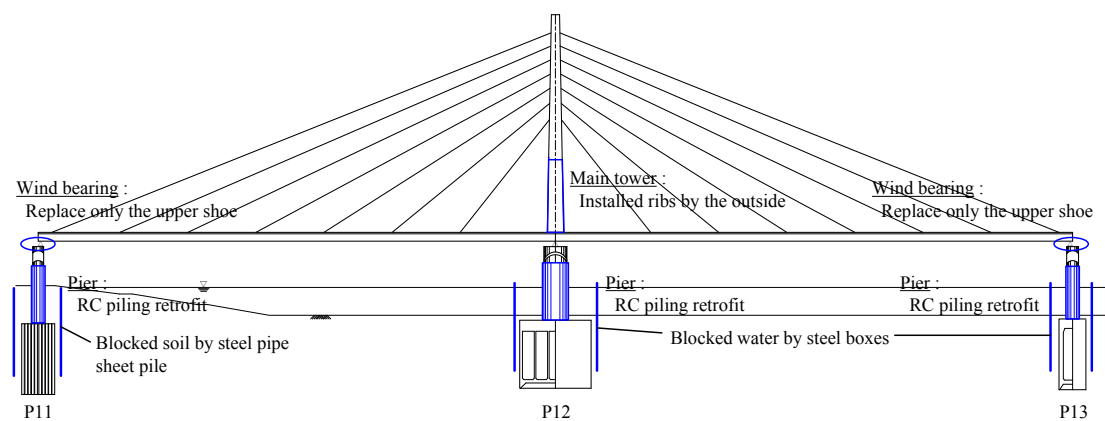


Fig.17 Seismic retrofit measures

comparative investigation was conducted between the PC confined method that allowed underwater placement, and the method involving blocking of water by steel casings to allow work to be performed in a dry condition. The latter method was adopted because of the significant advantages in terms of both construction period and cost and low impact on water quality.

Conclusion

The seismic performance of the cable-stayed bridge in Sakitama Bridge was examined. Retrofit was deemed necessary for the tower, wind bearing, and piers to ensure the target seismic performance. For the caisson foundation, however, the necessity of the retrofit will be re-examined after the results of experimental research now conducted by PWRI becomes available.

Acknowledgments

We highly appreciate "Sakitama Bridge Seismic Retrofit Examination Committee (Chair, Shigeki Unjou, Earthquake Disaster Researcher, NILIM)" for their valuable opinion and comments.

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