A STUDY ON SEISMIC RETROFIT OF OHSHIMA SUSPENSION BRIDGE WITH EMPHASIS ON TOWER-GIRDER COLLISION PROBLEM

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Abstract

The Ohshima Bridge is a long-span suspension bridge with a center span of 560 m. Seismic vulnerability assessments against site-specific large-scale earthquakes showed possibilities of tower-girder collision, as well as failure of center stay cables, RC piers and steel bearings. Presented in this paper is a study on the seismic retrofit with emphasis on the tower-girder collision problem. Pushover analyses with a shell model and nonlinear dynamic analyses with a 3D full bridge model were performed for damage evaluations of the existing bridge due to the collision effect. Furthermore, comparing investigations of the countermeasure, such as installation of viscous damper and rubber buffer, were implemented.

Introduction

The Ohshima Bridge, one of the Honshu-Shikoku Bridges connecting the Honshu and the Shikoku Island, is a long-span suspension bridge with a center span of 560 m, as shown in Figure 1. The stiffening girder is a steel box type with 23.7 m wide and 2.2 m deep. The tower is a steel two-story rigid-frame type with 88.35 m high. The bridge was designed based on specific design codes developed for the bridges and opened in 1988. In the original seismic design, although specific design seismic force was defined in consideration of earthquakes possibly occurring in plate-boundaries around the bridge sites, an inland near-field earthquake, such as the Hyogo-ken Nanbu Earthquake in 1995, was not considered in the design seismic force. The bridge is located in seismic-prone area, where plate boundaries and several inland active faults exist nearby. According to the latest findings, there is concern that a large-scale earthquake exceeding the original design seismic force would occur and seismic risk for the bridge would increase.

Above mentioned background motivated us to implement seismic performance upgrading for the bridge since the bridge is designated as a lifeline corridor for emergency transportations or restoration works immediately after a large-scale earthquake. By performing seismic vulnerability assessments against site-specific large-scale earthquakes, possibilities of tower-girder collision, as well as failure of center stay cables, RC piers and steel bearings were anticipated. This paper places an emphasis on a study of the tower-girder collision problem since damages of those elements might lead to critical situation of the entire bridge system. Pushover analyses with a shell model and nonlinear dynamic analyses with a 3D full bridge model were performed for damage evaluations of

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the existing bridge due to the collision effect. Furthermore, comparing investigations of the countermeasure, such as installation of viscous damper and rubber buffer, were implemented.

Seismic Vulnerability Assessment

Seismic ground motions used for seismic vulnerability assessments were indicated in Figure 2. These were defined as rock outcropping motions on the bedrock where S-wave velocity (Vs) was 2 km/s. Two types of scenario earthquakes were considered; one is the Tounankai-Nankai Earthquake which is an interplate earthquake, the other is the Geiyo Earthquake which is an intraslab earthquake. A fault model for the Tounankai-Nankai Earthquake is shown in Figure 3. In addition to the two types of scenario earthquakes, a seismic ground motion coming from an earthquake with a magnitude of M_J6.8 occurring just beneath the site was considered. This means a concern that an unknown inland active fault might exist near the site. All the seismic ground motions were estimated by a hybrid method (green's function method + 3D finite difference method).

Analytical conditions and an analytical model are shown in Table 1 and Figure 4, respectively. Nonlinear dynamic analyses with considering both material and geometric nonlinearities were performed to simulate a complicated seismic behavior. Besides, a breakage of a structural element, such as a center stay, was considered in the dynamic analyses. In other words, in cases where a response of a structural element exceeded its strength during dynamic analyses, the analyses were paused temporarily and resumed again after removing the corresponding element from the analytical model. As input motions into the analytical model, effective seismic motions considering soil-structure (kinematic) interaction were applied in three directions simultaneously.

As a result, although responses of main structural elements, such as the steel towers, stiffening girder, main cables and suspenders were within elastic range, responses of all center stays, some steel bearings and RC piers at side spans exceeded its strengths. Furthermore, the girder almost collided against the tower shaft; the maximum relative displacement between the tower and the girder was 595 mm, while the gap in a dead load condition was 600 mm. An overview of the girder end at the tower is shown in Figure 5.

A load combination in the analyses was "dead load (D) + effect of earthquake (EQ)" in accordance with seismic design of general Japanese bridges (Japan Road Association, 2002). To put it the other way around, live load (L) and effect of temperature change (T) was not taken into account the analyses, therefore the collision would surely happened in case of a load combination "D+EQ+L+T" since the gap narrows approximately 150 mm due to a load combination "L+T". Meanwhile, damages of the tower and the girder may affect the seismic safety and serviceability after events since those are critical elements composing the suspension bridge system. Consequently, to be on the safe side, it was decided that damage evaluations of the existing bridge in case of a load combination "D+EQ+L+T" was performed in order to grasp damage degree of the collision part and effects of the collision on other structural elements.

Damage Evaluation of Existing Bridge due to Collision

Figure 6 shows a flowchart of damage evaluations of the existing bridge due to the collision. Firstly, pushover analyses using a shell model were performed in order to define a property of a nonlinear spring element representing the collision. A gap of the nonlinear spring element was set to be 450 mm, which is subtraction of 150 mm, narrowing of gap due to "L+T", from 600 mm, gap in a dead load condition. Then, after installing the nonlinear spring elements into the 3D full bridge model, dynamic analyses were performed in order to see effects of the collision on other structural elements by using the Geiyo Earthquake ground motion, which was dominant for the collision problem among the three earthquakes. Finally, damage degree of the collision part was evaluated by feedback of the colliding force obtained by the dynamic analyses on the pushover analyses.

Figure 7 shows time histories of colliding forces, a girder displacement, a tower shaft displacement and a tower shaft strain at the base by the dynamic analyses. "No Collision Considered" in the legends means responses by dynamic analyses without installing the nonlinear springs in the model. As indicated in Figure 7 (a), (b), (c), the center stays failed at around 19 and 21 second, and then the girder collided against (right-hand side) tower shaft of 5P at around 21 second, (right-hand side) tower shaft of 6P at around 34 second, and then (left-hand side) tower shaft of 5P at around 37 second. The collision reduced the girder displacement by approximately 0.1 m. Table 2 shows effects of the collision on other structural elements. This shows that effects of the collision on other structural elements were small enough and, in particular, response of the tower shaft at the base, which was the largest value in the tower by dynamic analyses and had been a prominent concern, was not affected at all. This is because the maximum response of the tower shaft occurred at the different time from the collisions, as shown in Figure 7 (d), (e). Besides, time histories during 20 to 24 second in Figure 7 (d), (e) show that the collision made the response of the tower shaft lower. This seems to be a general phenomenon, as suggested in Figure 8, a collision pattern diagram, owing to oscillation characteristics of the tower and the girder. This diagram indicates that the girder likely collides against the tower shaft during a period when the tower shaft deforms into the main span direction. It is unlikely that the girder collides against the tower shaft during a period when the tower shaft deforms into the side span direction, which aggravates responses of the tower shaft. Therefore, it seems reasonable to say that the collision does not have much influence on other structural elements, although the influence depends on some other factors, such as amplitude of the colliding force, phase characteristic and intensity of the seismic ground motion, and so on.

Figure 9 shows damage degree of the collision part estimated by feedback of the colliding force by the dynamic analyses on the pushover analyses. These figures show that the maximum response of the steel deck was about eight times of yield strain (7.9 ε_y) and resulted in local plastic deformation, whereas the maximum response of the tower shaft remained within elastic range (0.42 ε_y). Furthermore, even though the response of global deformation as a column by the dynamic analyses (0.31 ε_y) added the response of local deformation as a plate (0.42 ε_y), the sum (0.73 ε_y) did not exceed a yield point.

As stated above, although some minor damage was generated at the steel deck, it was evaluated that no damage was generated at the tower shaft and the collision between the girder and the tower shaft did not have much influence on other structural elements by damage evaluations of the existing bridge, which took a load combination with low probability of occurrence, "D+EQ+L+T", into account in the analyses. The tower, however, was considered as one of critical elements composing the suspension bridge system. Especially, damage of the tower shaft seemed to be hardly repairable since it carries huge dead load. Therefore, it could be stated that there was still concern about seismic safety and serviceability after events in an unanticipated situation. Accordingly, it was decided to investigate a countermeasure for avoiding the collision between the girder and the tower shaft or decreasing the colliding force.

Comparing Investigation of Countermeasures

Three types of the countermeasure were compared in terms of structural characteristic, workability, durability and cost, as tabulated in Table 3. Type A, B were to install viscous dampers or high damping rubbers (HDR), respectively, for avoiding the collision and decreasing the loading on the tower. In a force vs. displacement relationship of Type B, a gap of 350 mm was provided not to restrain movement of the girder in a normal load condition. Type C was to install some steel members on the girder and the tower horizontal with rubber buffers for making the girder collide against the tower horizontal beam, which carries less dead load, ahead of the tower shaft and decreasing the colliding force. After replacing by nonlinear spring elements with the three types of properties, three cases of dynamic analyses were performed using the 3D full bridge model. As a result, in terms of structural characteristic, reacting (colliding) force and the maximum response of the tower shaft, Type A, B were superior to Type C since the collision was avoided in cases of Type A, B. In contrast, in terms of workability and cost, Type C was superior to Type A, B, whereas the three types were equivalent in terms of durability. Consequently, Type C was assessed to be the best countermeasure considering following aspects;

- ✓ In terms of workability and cost, Type C was superior to Type A, B.
- ✓ Type C could avoid the collision between the girder and the tower shaft, which was considered as the most undesirable situation.
- ✓ The collision between the girder and the tower horizontal beam in Type C did not have much influence on other structural elements; the maximum responses at the tower horizontal beam, $\varepsilon_{max}/\alpha\varepsilon_y$ = 0.71, $\gamma_{max}/\alpha\gamma_y$ = 0.30, hardly changed as compared with Table 2.
- ✓ This countermeasure was for fail-safe consideration, as mentioned in the previous chapter.

Conclusions

The concluding remarks are summarized as follows;

- ✓ Since seismic vulnerability assessments of the Ohshima Bridge, a long-span suspension bridge with a center span of 560 m, showed a possibility of tower-girder collision, pushover analyses with a shell model and nonlinear dynamic analyses with a 3D full bridge model were performed for damage evaluations of the existing bridge due to the collision effect.
- ✓ As a result, although some minor damage was generated at the steel deck, it was evaluated that no damage was generated at the tower shaft and the collision between the girder and the tower shaft did not have much influence on other structural elements.
- ✓ However, it was decided to investigate a countermeasure for avoiding the collision between the girder and the tower shaft, which was considered as the most undesirable situation, or decreasing the colliding force, since there was still concern about seismic safety and serviceability after events in an unanticipated situation.
- ✓ After comparing three countermeasures in terms of structural characteristic, workability, durability and cost, installing some steel members with rubber buffers for making the girder collide against the tower horizontal beam ahead of the tower shaft was assessed as the best one.

Acknowledgments

The authors wish to acknowledge useful supports from the Honshi Seismic Reinforcement Study Committee participants (chairman: Dr. Iemura, the professor emeritus of the Kyoto University).

References

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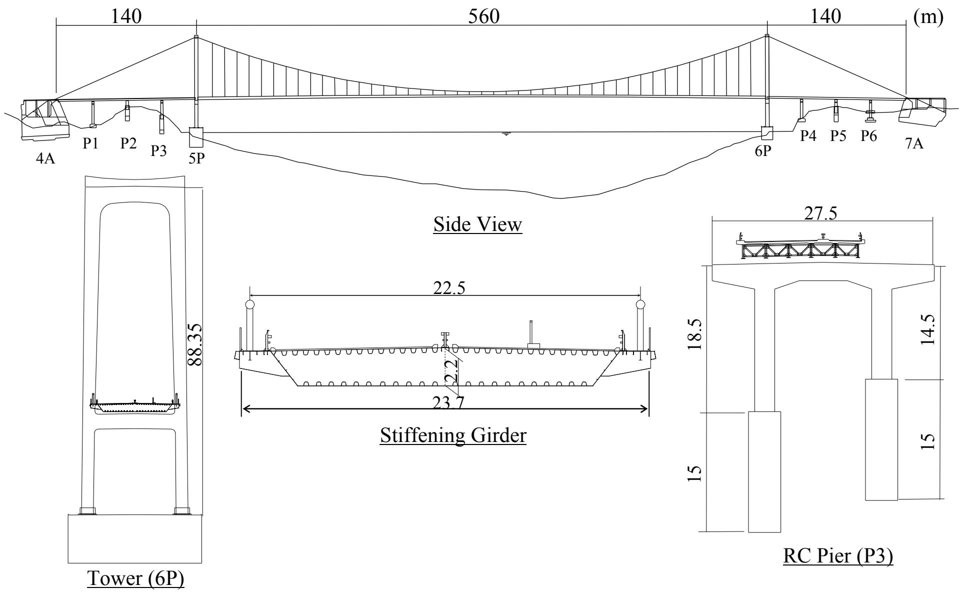
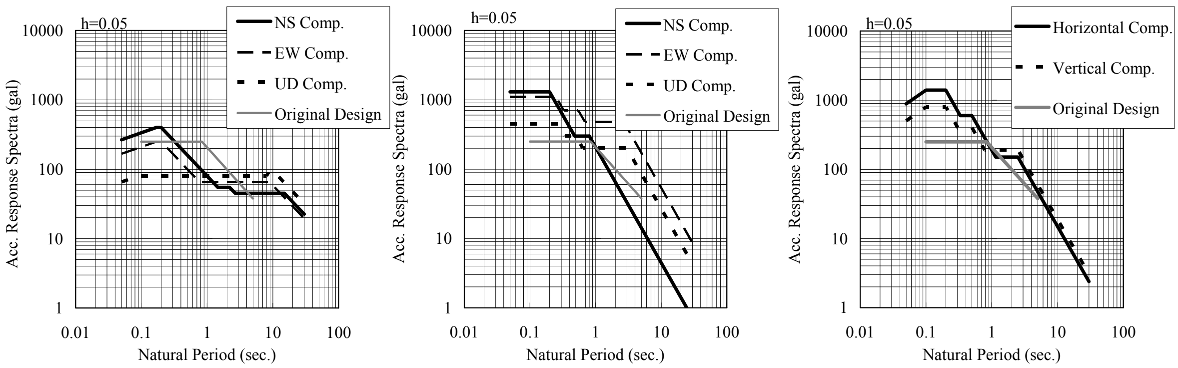


Fig.1 Overview, Ohshima Bridge



(a) Tounankai-Nankai EQ.　(b) Geiyo EQ.　(c) Unknown Inland Active Fault EQ.

Fig.2 Large-scale Earthquakes

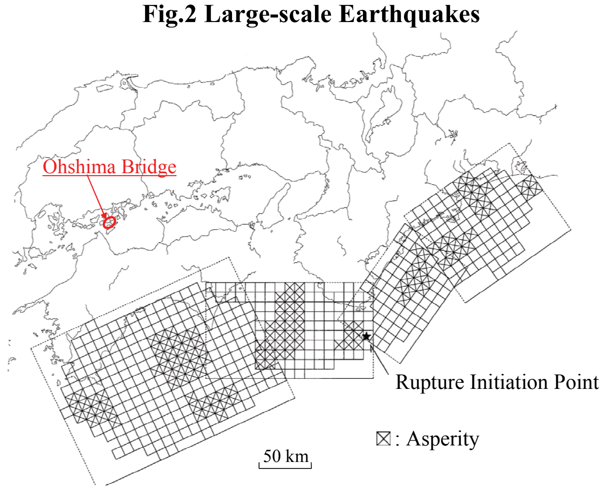


Fig.3 Fault Model of Tounankai-Nankai EQ.

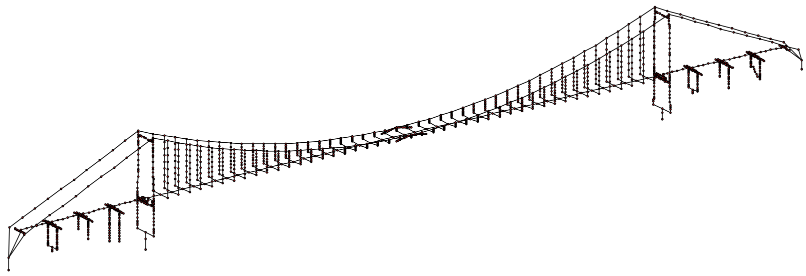


Fig.4 3D Full Bridge Model

Fig.5 Overview, Girder End at Tower

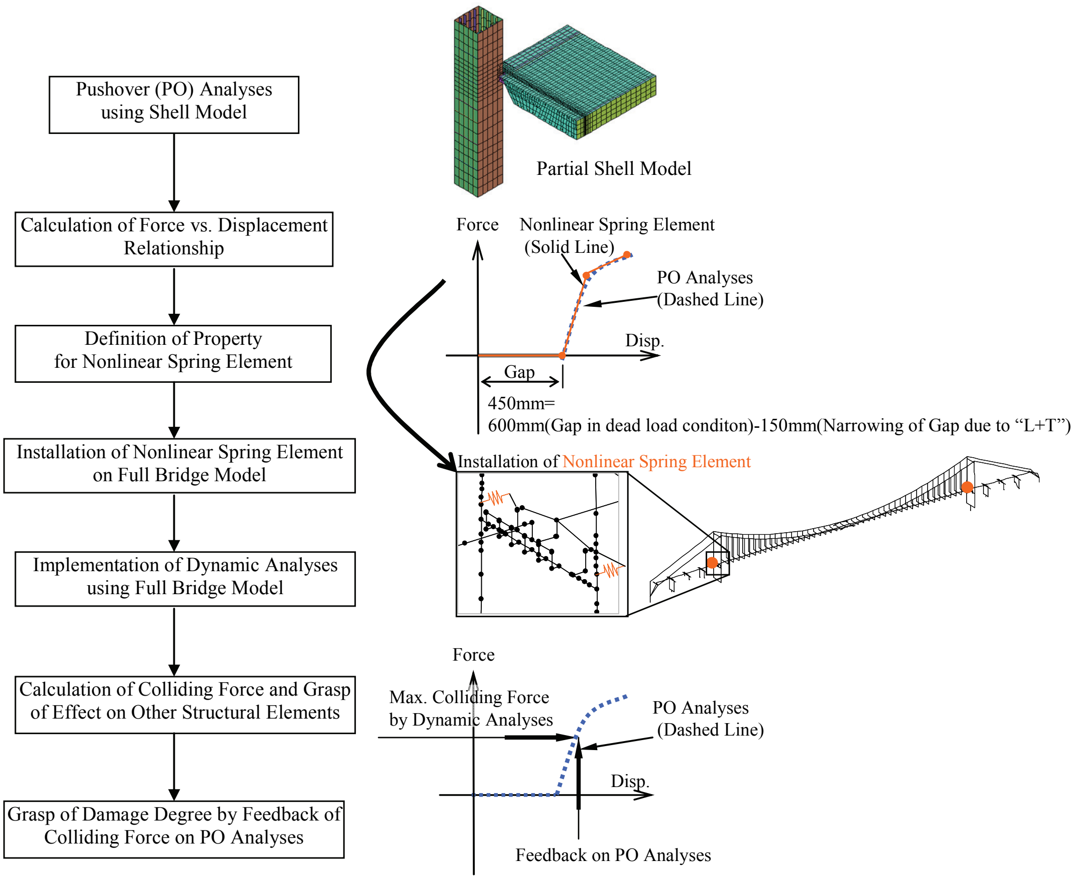
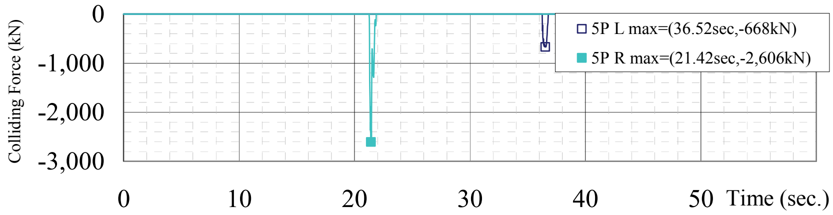
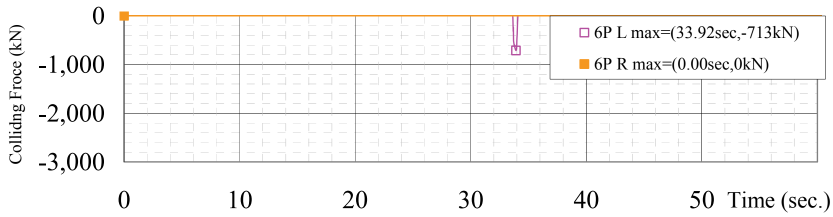


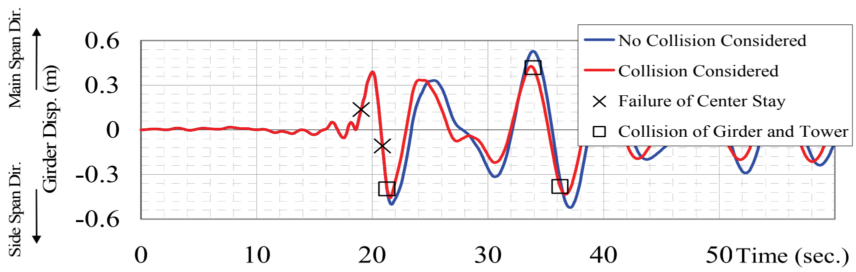
Fig.6 Flowchart of Damage Evaluation of Existing Bridge



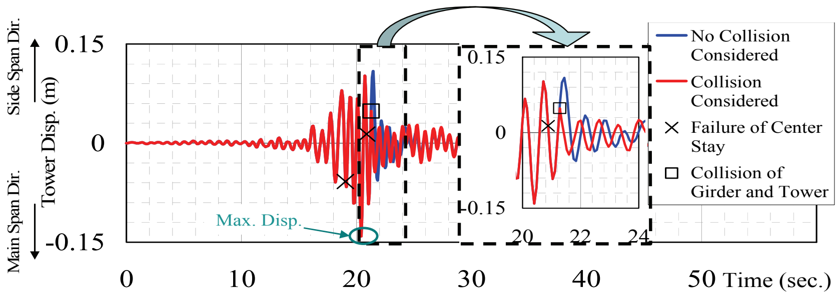
(a) Colliding Force at 5P



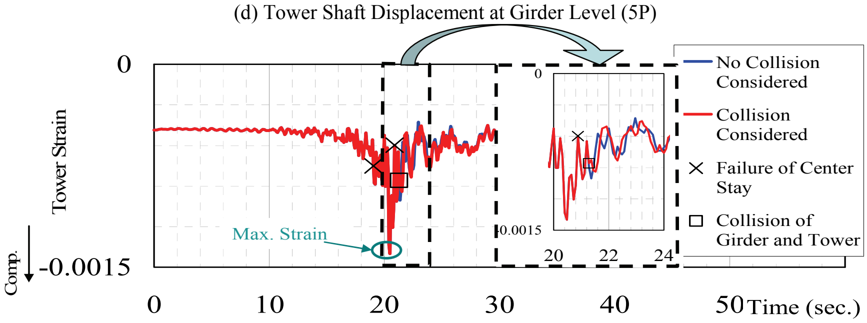
(b) Colliding Force at 6P



(c) Girder End Displacement (5P)



(d) Tower Shaft Displacement at Girder Level (5P)



(e) Tower Shaft Strain at Base (5P_R)

Fig.7 Time Histories by Dynamic Analyses

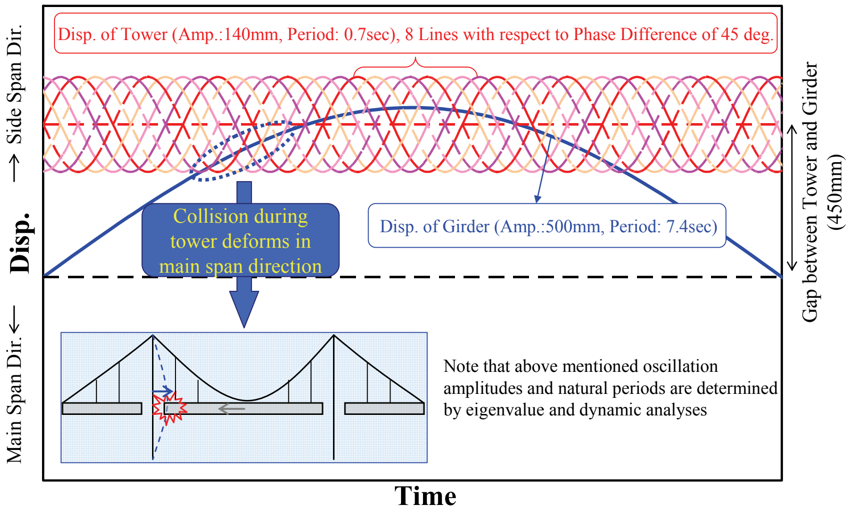
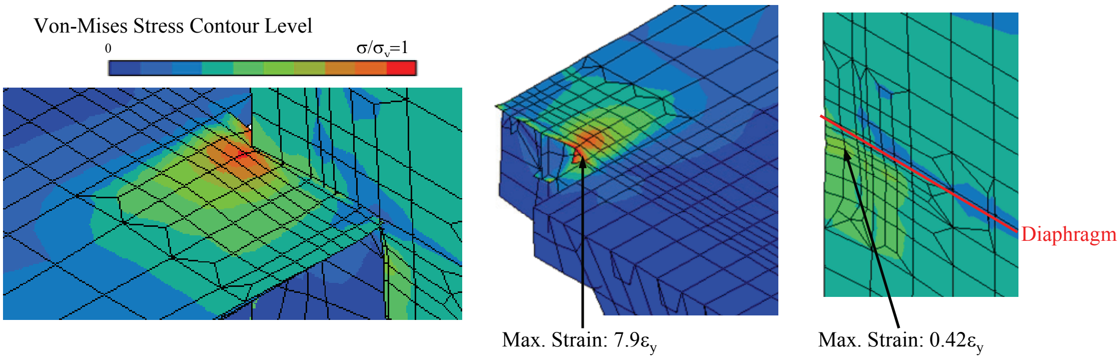


Fig.8 Collision Pattern between Tower and Girder



(a) Collision Part (b) Girder (c) Tower Shaft

Fig.9 Damage Degree of Collision Part

Table 1 Analytical Conditions

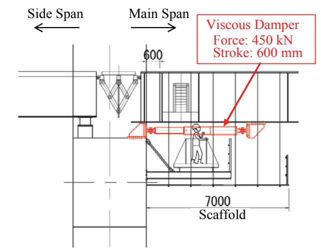
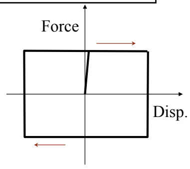
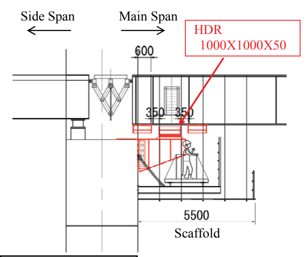
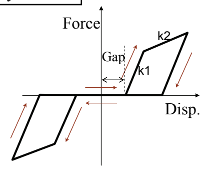
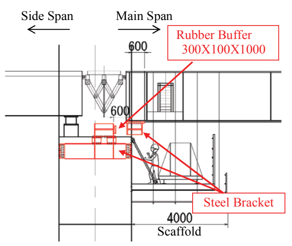
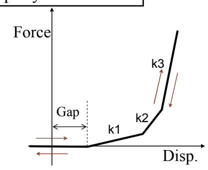
Analytical Method		Time history response analyses with material and geometric nonlinearities
Numerical Integration Method		Newmark's β method (β =0.25)
Damping Type		Member based stiffness-proportional damping
Time Interval		0.0025 sec.
Damping Ratios of Members	Tower, Girder	0.01
	Cable	0.01
	Foundation	0.1
Input Motion		Effective seismic motion
Loading Direction		3 directions (Longitudinal, Transverse and Vertical)

Table 2 Effects of Collision on Other Structural Elements

Quantity			Unit	Allowable Values		Max. Responses		Response Ratios
						(a) No Collision Considered	(b) Collision Considered	(b)/(a)
Tower (5P)	Shaft (strain at base)		$\varepsilon_{max}/\alpha\varepsilon_y$	Yield or Buckling	1.00	0.79	0.79	100%
	Horizontal (strain)	Lower	$\varepsilon_{max}/\alpha\varepsilon_y$	Yield or Buckling	1.00	0.71	0.71	100%
			$\gamma_{max}/\alpha\gamma_y$	Yield or Buckling	1.00	0.29	**0.31**	**107%**
		Upper	$\varepsilon_{max}/\alpha\varepsilon_y$	Yield or Buckling	1.00	0.93	0.93	100%
			$\gamma_{max}/\alpha\gamma_y$	Yield or Buckling	1.00	0.22	0.22	100%
Main Cable (tension force)			kN	Ultimate Strength	211,523	69,160	**69,790**	**101%**
Suspender (tension force)			kN	Ultimate Strength	2,920	1,257	1,257	100%
Sttiferning Girder (strain)			$\varepsilon_{max}/\alpha\varepsilon_y$	Yield or Buckling	1.00	0.86	0.86	100%
Bearing (5P)	(horizontal force)		kN	Yield Strength	3,180	2,671	2,671	100%
	(vertical force)		kN	Yield Strength	3,670	991	991	100%
	(displacement)		m	Movable Range	±0.620	0.596	0.604	**101%**

α: reduction coefficient due to buckling
ε_y: normal yield strain
γ_y: shear yield strain

Table 3 Comparison of Countermeasures

	Type A (Viscous Damper)	Type B (HDR)	Type C (Rubber Buffer)
Outline	* Installing 2 viscous dampers between girder and tower horizontal * Avoiding collision between girder and tower shaft by dissipating oscillation energies * Decreasing loading on tower  Property of Viscous 	* Installing 2 High Damping Rubbers (HDR) between girder and tower horizontal * Avoiding collision between girder and tower shaft by dissipating oscillation energies * Decreasing loading on tower  Property of HDR 	* Installing steel members on girder and tower horizontal * Making girder collide against tower horizontal ahead of tower shaft * Decreasing collision force by rubber buffers  Property of Rubber 
Structural Characteristic	Superior to Type C due to avoidance of collision	Superior to Type C due to avoidance of collision Larger bracket due to large HDR size	Inferior to Type A, B due to occurrence of collision
Reacting (Colliding) Force	1,500 kN	1,968 kN	2,303 kN
Max. Strain at Tower Shaft	$\varepsilon_{max}/\alpha\varepsilon_y$=0.74	$\varepsilon_{max}/\alpha\varepsilon_y$=0.77	$\varepsilon_{max}/\alpha\varepsilon_y$=0.79
Workability	Inferior to Type C due to distantly-positioned work from tower and larger scaffold	Inferior to Type C due to distantly-positioned work from tower and larger scaffold	Superior to Type A, B due to work close to tower and smaller scaffold
Durability	Almost same because durability of each devise has been verified by a long-term durability test, such as an accelerating degradation test.		
Cost (ratio to Type C)	1.81	2.19	1.00