STUDY ON REDUCTION OF VIBRATION CONTROL DEVICES FOR AKASHI-KAIKYO BRIDGE

T. FUJIWARA, K. OGIHARA & S. FUKUNAGA
Long-span Bridge Engineering Center,
Honshu-Shikoku Bridge Expressway Co., Ltd., Japan
toru-fujiwara@jb-honshi.co.jp
katsuya-ogihara@jb-honshi.co.jp
susumu-fukunaga@jb-honshi.co.jp

ABSTRACT

The Akashi-Kaikyo Bridge is a suspension bridge with a main span of 1,991m. Since the towers have the height of 287m, they could suffer out-of-plane deformation due to vortex induced oscillation even after completion of the bridge. Therefore, cruciform cross-section which had excellent aerodynamic stability was selected for the tower shafts and tuned mass dampers were installed in each tower shaft as vibration control devices. Furthermore, additional dampers were installed between the tower and the girder for double safety, because the towers are the most important structure of the suspension bridge. According to the monitoring data, it has been found that recorded vibration amplitudes of the tower shafts were very small compared with design values. It indicated that some of the vibration control devices may be unnecessary. Although the vibration control devices have to be maintained in good condition, it has been demanded to reduce expenses for their maintenance. Therefore we started to study on damping performance and allowable vibration amplitude of the tower, in order to make some of the devices unnecessary. After safety of the bridge will be confirmed without some devices, maintenance cost will be reduced.

1. INTRODUCTION

The Honshu-Shikoku Bridges (herein after referred to as HSB) connect Japanese two major islands, Honshu and Shikoku, with three routes, as shown in figure 1. The eastern route is the Kobe-Awaji-Naruto Expressway. The central route is the Seto-Chuo Expressway. This route is popularly known as the Seto Ohashi Bridges, which accommodate both a highway and a railway. The western route is the Nishi-Seto Expressway. The long-span bridges on this route accommodate passages for pedestrians, bicycles and motorcycles in addition to a highway.

The HSB consist of ten suspension bridges, five cable-stayed bridges, three truss bridges and an arch bridge, including the Akashi-Kaikyo Bridge; the world's longest suspension bridge with the main span of 1991m, and the Tatara Bridge; the Japanese longest cable-stayed bridge with the main span of 890m. The HSB are important parts of the national trunk roads in Japan, and no alternative route exists. Therefore, all bridges are required to be in sound condition for a long time.

Since typhoons have attacked Japan frequently, aerodynamic stability was one of the most important issues in the design for the HSB. Various vibration control devices have been installed on many bridges to ensure aerodynamic stability. In order to verify the aerodynamic stability, structural health monitoring has been conducted on major bridges. Meanwhile, as the vibration control devices are thin steel members or mechanical devices, it is necessary to maintain them carefully during the service period of the bridges. Maintenance cost of the devices has been becoming larger problem due to budget restriction in maintenance stage. As more than ten years have passed since completion of the HSB, it has become the time to evaluate effect of the devices by utilizing maintenance data. And some of the devices have been already removed with confirmation of safety of the bridge [1].

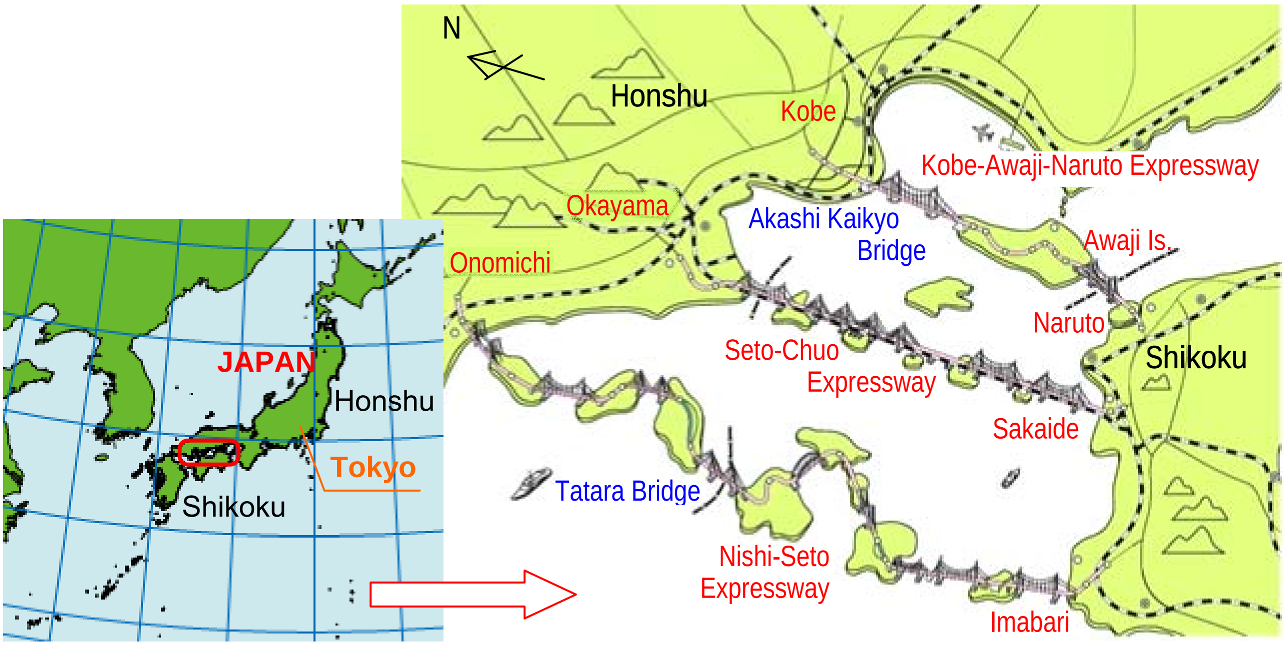


Figure 1 - Three routes of Honshu-Shikoku Bridges

The Akashi-Kaikyo Bridge is the world's longest suspension bridge, linking the Kobe City and the Awaji Island on the Kobe-Awaji-Naruto Expressway. Since the towers are steel towers with the height of 287m and are more flexible than other existing towers in the world, the towers could suffer out-of-plane deformation due to vortex induced oscillation under the wind not only during construction but also after completion of the bridge. Therefore, cruciform cross-section which had excellent aerodynamic stability was selected for the tower shafts and tuned mass dampers were installed in each tower shaft as vibration control devices in order to suppress the vibration amplitude of the tower shafts. Moreover additional dampers were installed between the tower and the girder for double safety, because the towers were the most important structure of the suspension bridge and should not get damage by any possibility.

According to results of the structural health monitoring, it has been revealed that recorded vibration amplitudes of the tower shafts of the Akashi-Kaikyo Bridge were very small compared with design values. It indicated that some of the vibration control devices were unnecessary. Therefore, we started to investigate the condition of the bridge, such as damping performance and allowable vibration amplitude of the towers, and damping performance of the vibration control devices, in order to make some of the devices unnecessary. After safety of the bridge will be confirmed without some devices, maintenance cost will be able to reduce. This paper presents study on reduction of the vibration control devices installed on the Akashi-Kaikyo Bridge.

2. OUTLINE OF AKASHI KAIKYO BRIDGE

The Akashi Kaikyo Bridge is the world's longest suspension bridge with the total length of 3,991m including the main span of 1,991m, as shown in figure 2. This bridge was opened to traffic in April l998. As Japan is located in typhoons and earthquakes prone area, many typhoons have attacked the bridge site and the Hyogoken-nanbu Earthquake (the Kobe Earthquake) occurred near the site in January 1995.

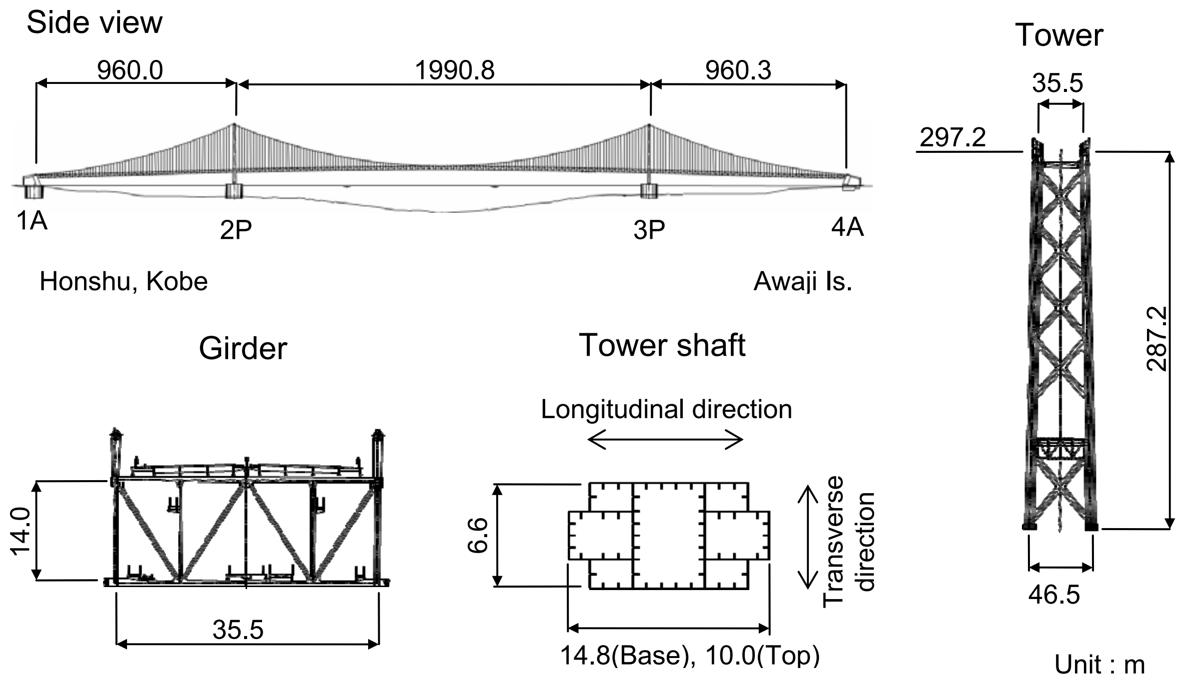


Figure 2 - General view of Akashi-Kaikyo Bridge.

In addition, the Akashi Strait has the deepest seabed of 110m, the maximum tidal current of 9 knots, and the daily congestion of about 1400 navigating ships. In order to overcome severe natural conditions and the social requirements, advanced technologies were introduced in design for this bridge.

Especially the following technical challenges were required in wind-resistant design;

- Aerodynamic stability of the tower after completion as well as under construction
- Aerodynamic stability of the stiffening girder against the required wind velocity of 78 m/s, which was estimated from wind observation records for more than 20 years
- Dynamic behaviours under large three-dimensional deformation of the whole bridge and fluctuation of wind velocity due to its long span.

Therefore, aerodynamic stability of the stiffening girder was verified by wind tunnel tests using a 1/100-scaled full bridge model of 40m length and the truss-type stiffening girder which had large torsional stiffness was selected [2].

3. OUTLINE OF DESIGN FOR TOWERS OF AKASHI-KAIKYO BRIDGE

3.1. Design of tower

The towers of the Akashi-Kaikyo Bridge have the height of 287m and were about 100m taller than those of existing suspension bridges at the time of its design. The towers are made of steel and support the cables reaction of about 1,000 MN. The dimensions of the tower shafts are 6.6 m x 10 m at the top and 6.6 m x 14.8m at the bottom, as shown in figure 2. Main steel grade is SM570 (steel for welded structure having breaking strength of 570N/m^2), and the total weight of one tower reaches about 24,700ton.

Since the towers are made of steel and their height is 287m, the towers are flexible considerably. And their natural frequency is relatively low during construction and even after completion of the bridge. Due to this low natural frequency, the towers vibrate easily by vortex induced oscillation, not only during construction but also after completion of the bridge. Control of the vortex induced oscillation, therefore, was one of the most important issues in design of the towers.

3.2. Shape of cross-section for vibration control

Various wind tunnel tests had been conducted by an elastic 1/86th three-dimensional model with height of 3.3m. Based on the results, the cruciform cross-section was selected for the tower shafts. Size of the cut-off at the corners varies according to the height of the towers. Although the amplitude of out-of-plane vibration became lower with this cross-section, vibration amplitude was still large.

3.3. Vibration control device [3]

3.3.1. Outline of design of vibration control device

As the amplitude of out-of-plane vibration was not enough small, wind tunnel tests were conducted for the towers both during construction and after completion of the bridge. Resonance wind velocity of the vibration, and the relationship between the damping of the tower and the vibration amplitude were obtained. Considering stress in the tower members and the safety of construction work, condition of the towers such as allowable vibration amplitude or allowable acceleration, was decided. And the design damping of the towers were calculated by considering the results of the wind tunnel tests and the allowable vibration amplitude. As the results, it turned out that out-of-plane displacement due to the vortex induced oscillation exceeded the allowable displacement without any vibration control devices in slower wind velocity than the design wind velocity. Therefore, some vibration control devices were needed to reduce vibration amplitude.

As the vibration control device, Tuned Mass Dampers (hereinafter referred to as TMD), Tuned Liquid Dampers, and Friction Dampers, etc. were examined, and Tuned Mass Dampers were chosen from the reliability and cost.

3.3.2. Design procedure of vibration control device

The TMD consists of a hanging pendulum, two springs and two oil dampers as shown in figure 3. To design the TMD, the tower and the TMD were assumed as Two-Degree-of-Freedom (hereinafter referred to as 2DOF) system. As external force by the vortex induced oscillation was harmonic loading, analysis of the 2DOF system was conducted as the system subjected to harmonic loading.

Figure 3 shows the procedure to design the TMD. At first, weight of pendulum, frequency, and damping were assumed. Then, analysis of 2DOF system was conducted and total damping of this 2DOF system was obtained. Frequency and damping of the TMD were adjusted until total damping of the 2DOF system satisfied the design damping of the towers and amplitude of the pendulum satisfied the design amplitude of 50 cm. This procedure was repeated until the minimum weight of pendulum was obtained. As adjusting error of the TMD, 5% change of the frequency and 10% change of the damping of the TMD were also considered.

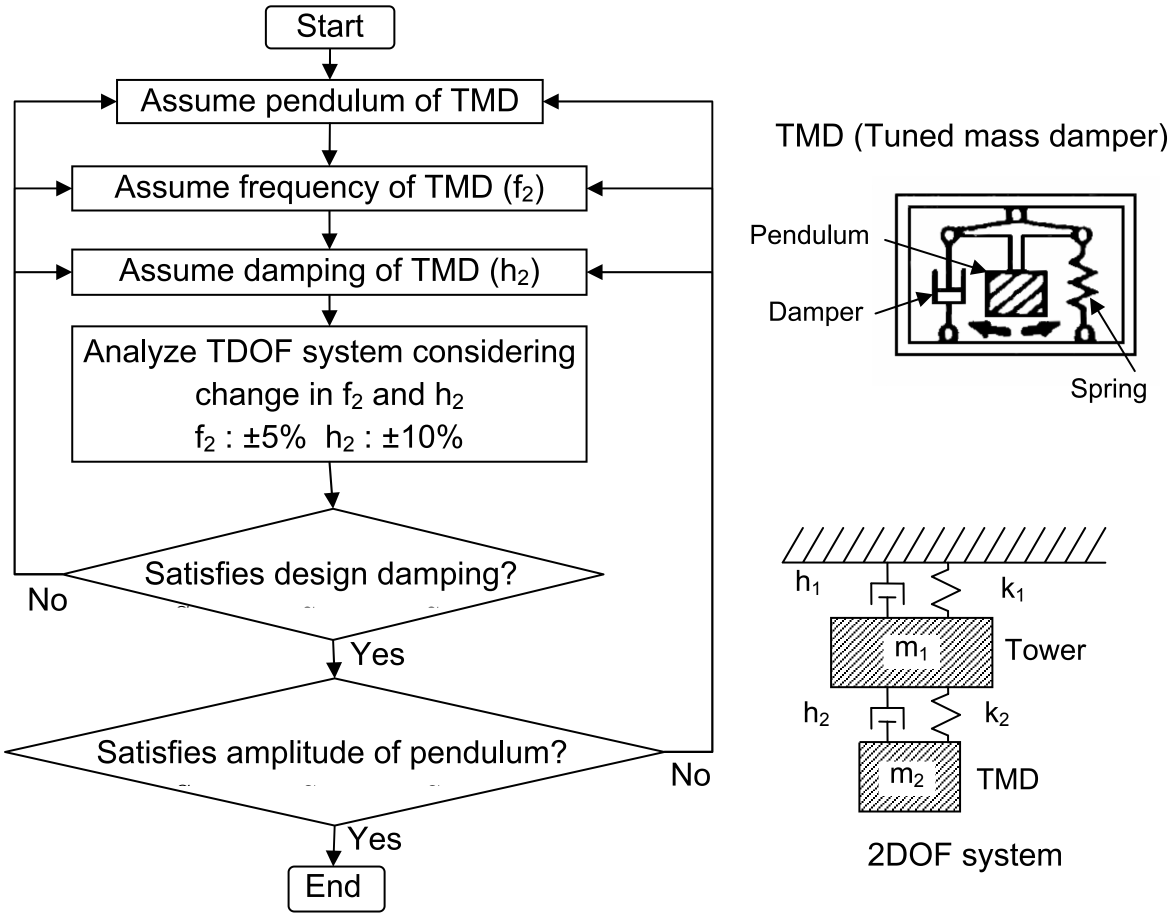


Figure 3 - Procedure to design TMD

3.3.3. Vibration control devices during construction of bridge

During construction of the bridge, three types of TMDs had been used for vibration control. Eight units of TMD-1 with the mass of 10.5 tons and twelve units of TMD-2 with the mass of 9.5 tons were installed in each tower shaft as shown in figure 4. These TMDs are also used for vibration control after completion of the bridge. As additional TMD during construction of the bridge, two units of TMD-3 with the mass of 10.5 tons were installed at the top of the tower.

When towers are free standing or are supported by a few strands of cable, towers are vulnerable to vibrate by very low wind velocity. The tower of the Akashi-Kaikyo Bridge would suffer vortex induced oscillation by very low wind velocity of 9–17 m/s. This vibration was the 1st flexural vibration and its frequency was continually changed from 0.13 Hz to 0.24 Hz by progress of cable erection work. As the vibration was harmful for erection work of the cables, Semi-Active Dampers (hereinafter referred to as SAD) were installed at the top of the towers. And the vibration was suppressed lower than 50 gal during early stage of construction of the cables. Table 1 shows the relationship between vibration frequency and vibration control devices during construction of the bridge.

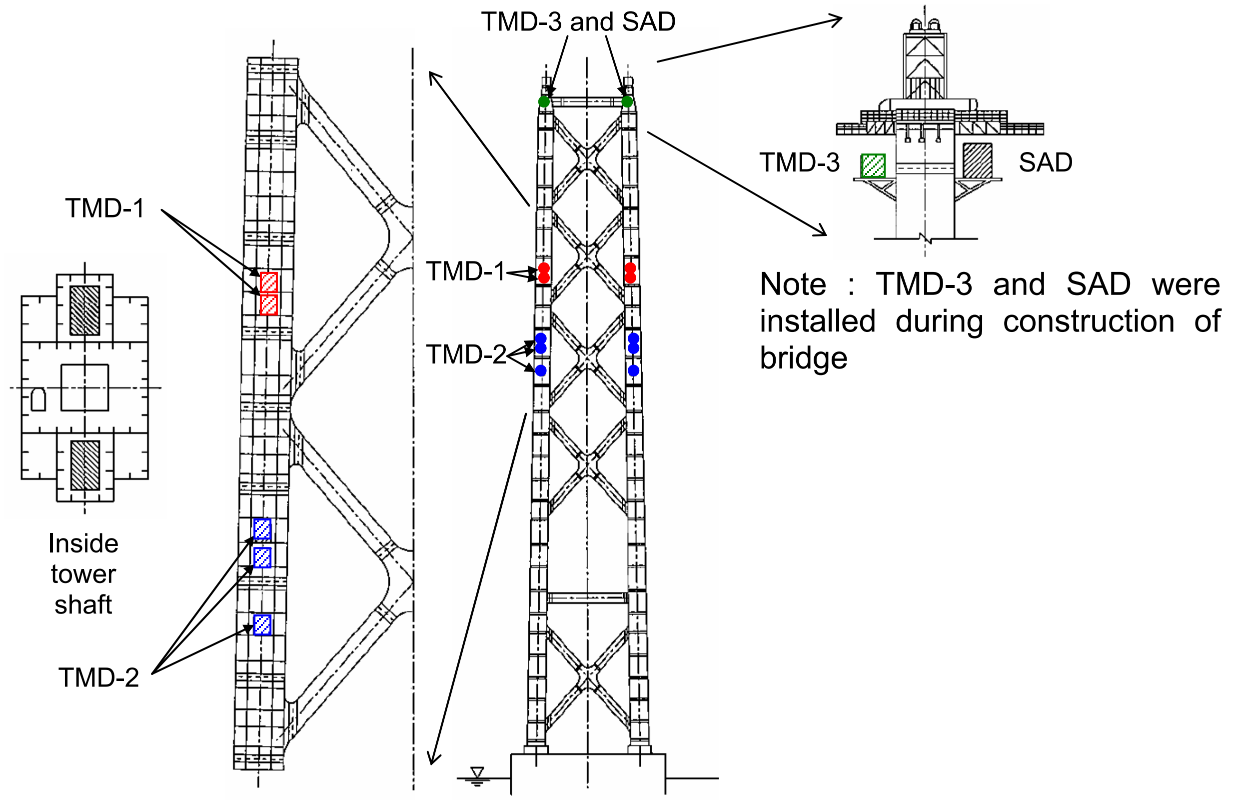


Figure 4 - Location of vibration control devices

Table 1 - Vibration control devices during construction of the bridge

Frequency (Hz)	Vibration control devices	Weight of pendulums
0.6 – 1.3	TMD-2 (inside tower)	114t
0.3 – 0.6	TMD-1 (inside tower) TMD-3 (top of tower)	84t 21t
0.3 or lower	SAD (top of tower)	-

3.3.4. Vibration control devices after completion of bridge

As the vibration control devices after completion of the bridge, TMD-1 and TMD-2 are used to suppress flexural vibration and torsional vibration, respectively. Necessary weight of the pendulum of the TMD-2 is 90t and is lighter than that during construction of the bridge. But the TMD-2 during construction were decided to use after completion of the bridge. Therefore, the weight of the pendulum is heavier than the necessary weight. Table 2 shows vibration control devices after completion of the bridge.

Table 2 - Vibration control devices after completion of the bridge

Vibration mode	Vibration control devices	Weight of pendulums
Torsional mode (0.75 Hz)	TMD-2 (inside tower)	114t
Flexural mode (0.44 Hz)	TMD-1 (inside tower)	84t

Although the tower shafts were designed with cruciform cross-section which had excellent aerodynamic stability, the towers would be excited by the vortex induced oscillation even after completion of the bridge. The vortex induced oscillation would occur at the wind velocity of about 36 m/s for the flexural vibration mode (0.44Hz) and at the wind velocity of about 67 m/s for the torsional vibration mode (0.75Hz). The maximum amplitude would be generated by wind with the azimuth angle of about 10 degrees from the transverse direction.

Considering the probability of occurrence of strong wind and the allowable stress, design of TMDs of the towers was conducted. Allowable amplitude of the tower shafts at two-third height for the flexural vibration was calculated by allowable stress (about 80% of yielding stress) of the tower shaft member and this amplitude was 407mm. But the resonance wind velocity was very low of about 36 m/s and this wind velocity was thought to occur frequently. Therefore, allowable amplitude for the flexural vibration was decided to be 300mm. According to the analysis result of the 2DOF system, it turned out that logarithmic decrement of the tower was necessary to be 0.051 or more in order to control the flexural vibration within the allowable amplitude. And considering errors of vibration frequency and of the results of the wind tunnel tests, the TMD-1 inside the tower were adjusted to gain 20% more than the necessary logarithmic decrement of the tower.

For the torsional vibration, allowable amplitude of the tower shafts at two-third height was calculated by yielding stress of the upper horizontal beam member and this amplitude was 150mm. The resonance wind velocity was the same as the design wind velocity of 67 m/s and such kind of wind was thought to hardly occur. Therefore, allowable amplitude for the torsional vibration was decided to be 150 mm. According to the analysis result of the 2DOF system, it turned out that logarithmic decrement of the tower was necessary to be 0.071 or more in order to control the torsional vibration within the allowable amplitude. And considering errors of vibration frequency and of the results of the wind tunnel tests, the TMD-2 inside the tower were also adjusted to gain 20% more than the necessary logarithmic decrement of the tower.

However, the tower would be damaged, if large torsional vibration occurred. Therefore additional dampers were installed between the towers and the side span girders for double safety, in case of when the TMDs would not work for some reason. As the TMDs and the additional dampers are work cooperatively, the maximum out-of-plane displacement due to the vortex induced oscillation at two-third height of the towers is estimated to be 100mm for the flexural vibration and 49mm for the torsional vibration, respectively.

3.3.5. Structural health monitoring system

Since typhoons have attacked Japan frequently, aerodynamic stability is one of the most important issues in the design for the long-span bridges. Various measurement equipments are installed on major bridges including the Akashi-Kaikyo Bridge. In order to verify validity of the wind-resistant design and the effect of vibration control devices of the towers, structural health monitoring system was also installed on this tower. The measurement equipments of the towers are as follows;

- Anemometer at the top of the tower
- Velocity gauges at the top and two-third height of the tower
- Displacement gauge for the pendulum of TMD-1 and TMD-2

4. VIBRATION TEST AND FIELD OBSERVATION

4.1. Vibration test of free standing tower

Vibration test was conducted to examine vibration characteristics of the free standing tower with and without the TMDs. The Semi-Active dampers at the top of the tower were used as oscillators.

Table 3 shows the frequency and mode shape of both measured and calculated values. And they have good agreement.

Table 3 - Frequency and mode shape of free standing tower

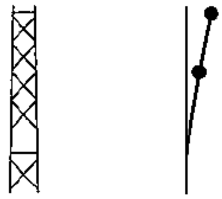
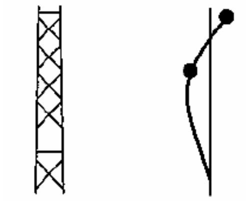
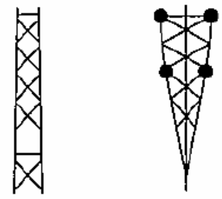
Vibration mode	Frequency (Hz)		Mode shape —— Calculated ● Measured
	Calculated	Measured	
1st Flexural vibration mode	0.127	0.126	
2nd Flexural vibration mode	0.677	0.673	
Torsional vibration	0.473	0.471	

Table 4 shows both measured and design damping (logarithmic decrement) of the tower for each vibration mode. The measured damping of the tower without vibration control devices for the 1st flexural vibration mode (Test No. 1) was a little smaller than the design damping specified in the design standard. But the dampings of other modes were bigger than the design values. And the damping of the tower with TMDs satisfied the design requirement for each mode.

Table 4 - Damping of free standing tower

Vibration mode	Test No.	SAD	TMD-1	TMD-2	TMD-3	Damping δ	
						Measured	Design
1st Flexural vibration mode	1	×	×	×	×	0.007	0.010
	2	P	×	×	×	0.028	0.024
	3	A	×	×	×	0.105	0.075
	4	P	○	○	○	0.036	0.024
	5	A	○	○	○	0.111	0.075
2nd Flexural vibration mode	6	×	×	×	×	0.038	0.010
	7	×	×	○	×	0.080	0.045
	8	P	○	○	○	0.096	0.045
Torsional vibration	9	×	×	×	×	0.028	0.010
	10	×	○	×	○	0.075	0.042
	11	P	○	○	○	0.075	0.042

Note : ○ is working, × is not working, P is passive working of SAD, A is active working of SAD

4.2. Field observation of free standing tower

The field observation of behavior of the free standing tower was conducted to confirm the vibration characteristics of the tower and effect of the vibration control devices. Figure 5 shows the relationship between average wind velocity and vibration amplitude of the top of the tower due to wind of around perpendicular direction. The results of the wind tunnel tests are also shown in figure 5. The data of the wind tunnel test are with logarithmic decrement (δ) of 0.025. Because the measured and design logarithmic decrement of the vibration test of the free standing tower was 0.028 – 0.036 and 0.024 for the 1st flexural vibration, respectively, as shown in table 4 (Test No.2 and No.4).

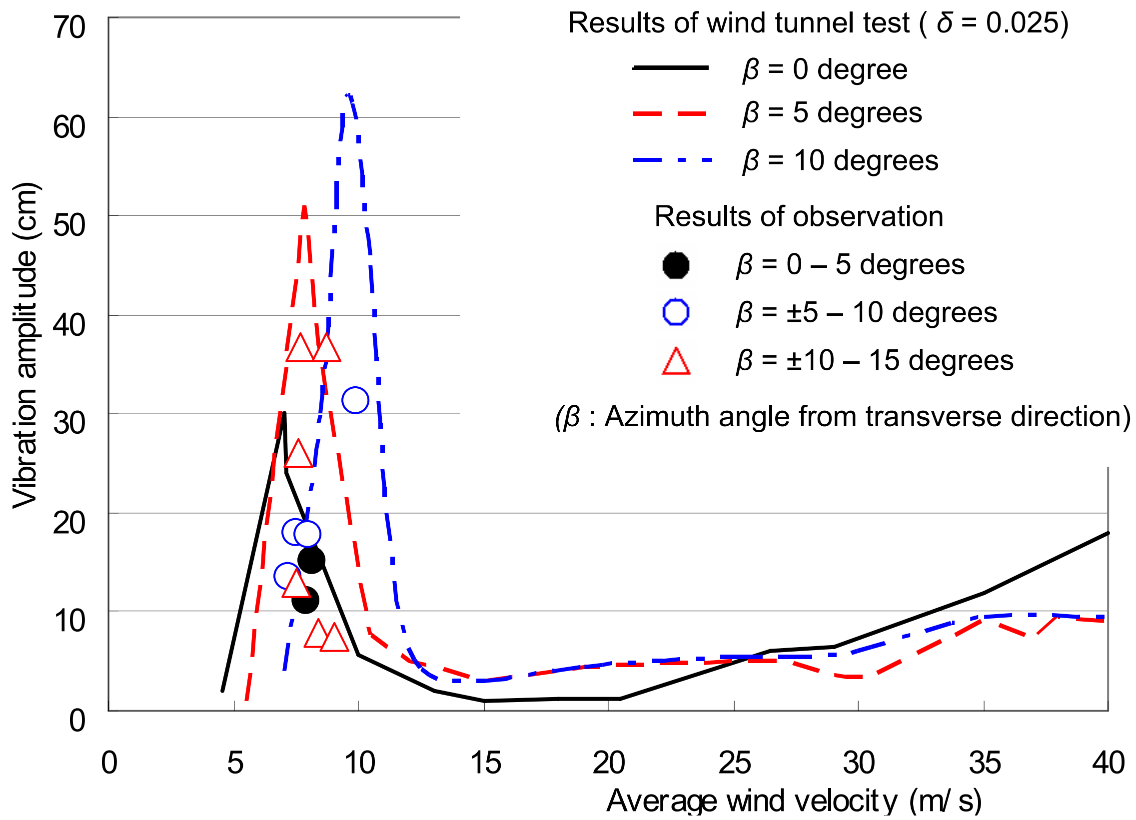


Figure 5 - Relationship between wind velocity and vibration amplitude

According to the results of the field observation, it was confirmed that the resonance wind velocity of the vortex induced oscillation agreed the results of the wind tunnel tests. However vibration amplitude was a little smaller than that of the wind tunnel tests. Therefore, effect of vibration control devices was thought to satisfy the design requirements.

4.3. Field observation after completion of bridge

After completion of the bridge, field observation has been conducted using structural health monitoring system. A strong wind was recorded by the typhoon 9807 as shown in figure 6. Average wind velocity was about 32 m/s and this wind velocity was nearly equivalent to the resonance wind velocity for the flexural vibration of about 36 m/s. The wind direction was 5 to 15 degrees from the transverse direction as shown in figure 7.

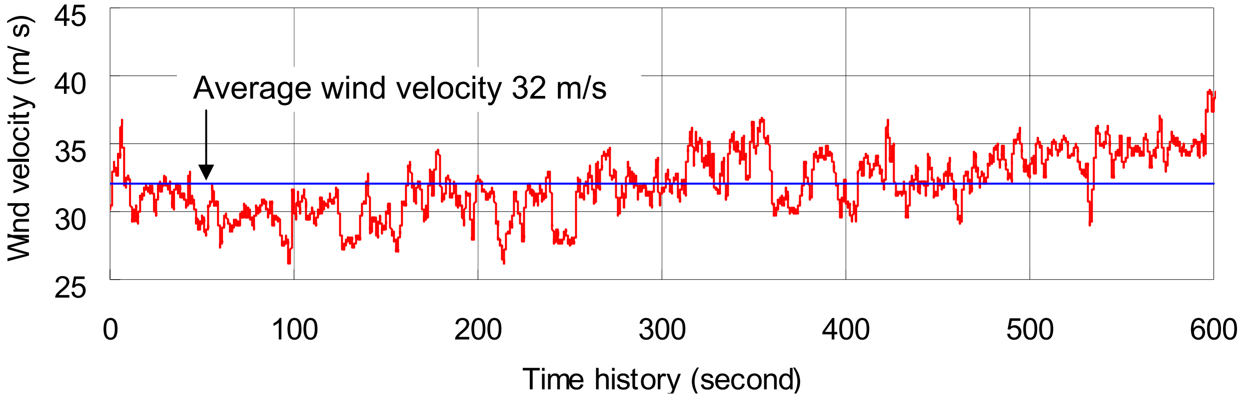


Figure 6 - Wind velocity by typhoon 9807

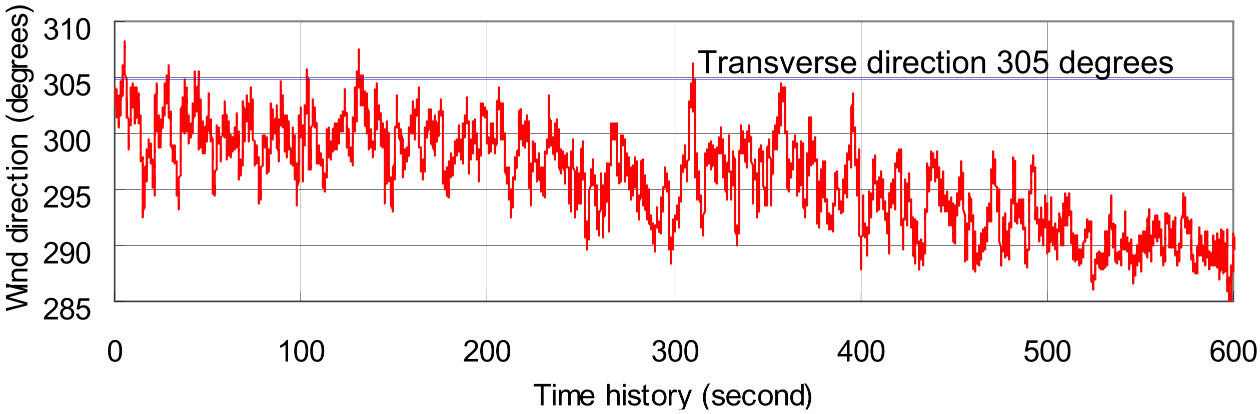


Figure 7 - Wind direction by typhoon 9807

Figure 8 shows out-of-plane displacement of the tower shaft at two-third height. Displacement data were calculated from velocity data measured by velocity gauges. The maximum out-of-plane displacement was about 8 cm. This displacement was thought to be generated by the 1st symmetrical and 1st asymmetrical vertical vibration mode of the girder, because their predominant frequencies were 0.065 Hz and 0.085 Hz, as shown in figure 9. According to the figure 9, there was another vibration mode whose predominant frequency was 0.45 – 0.46 Hz. This vibration mode was thought to be the flexural vibration mode of the tower whose design frequency was 0.44 Hz. Figure 10 shows out-of-plane displacement of the same position without long period vibration mode with its frequency of 0.2 Hz or lower. And this out-of-plane displacement was thought to be generated mainly by flexural vibration of the tower. The maximum displacement was about 2cm and very small compared with the design value of 10cm which was calculated by consideration of effect of the TMDs inside tower shaft and the additional dampers between the tower and the side span girder.

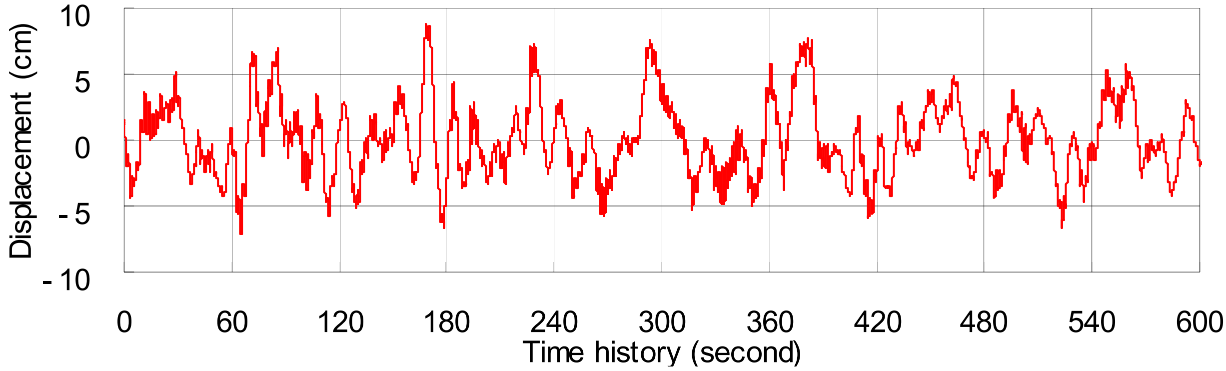


Figure 8 - Out-of-plane displacement of tower at two-third height

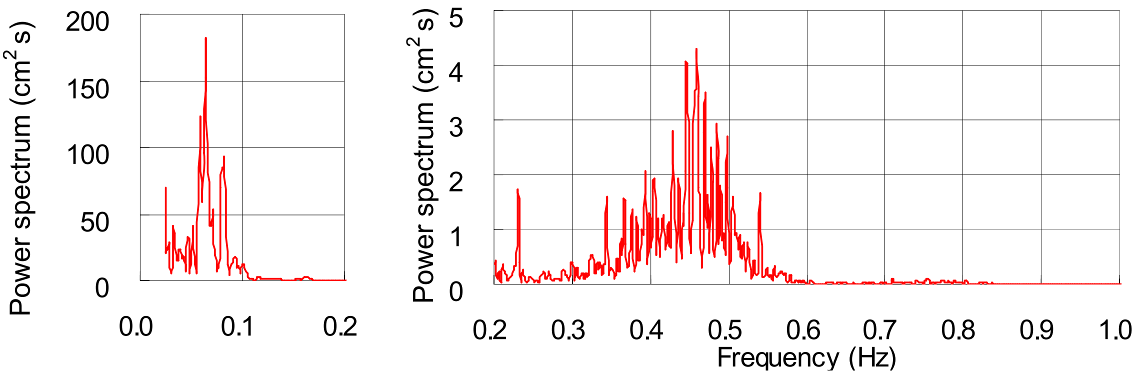


Figure 9 - Power spectrum of out-of-plane displacement of tower at two-third height

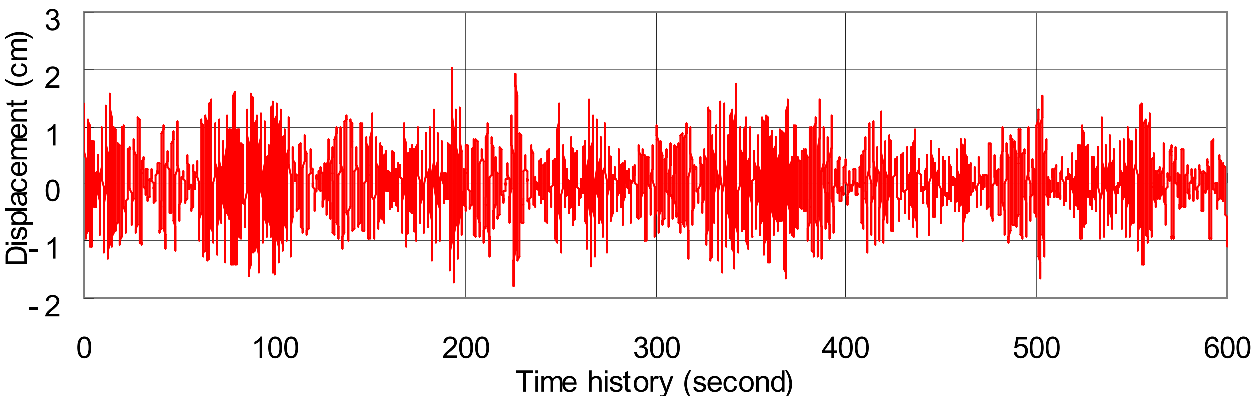


Figure 10 - Out-of-plane displacement of tower at two-third height without long period vibration mode

Figure 11 shows displacement of pendulum of the TMD-1. Although the displacement was very small of about 1cm, the predominant frequencies was 0.45 to 0.50 Hz, as shown in figure 12. The frequency of the displacement was almost the same as adjusted frequency of the TMD-1 (0.44 Hz). Therefore, the TMD-1 was adjusted with design frequency of the flexural vibration mode of the tower. However, it was clarified that the tower was hardly excited by wind from perpendicular direction.

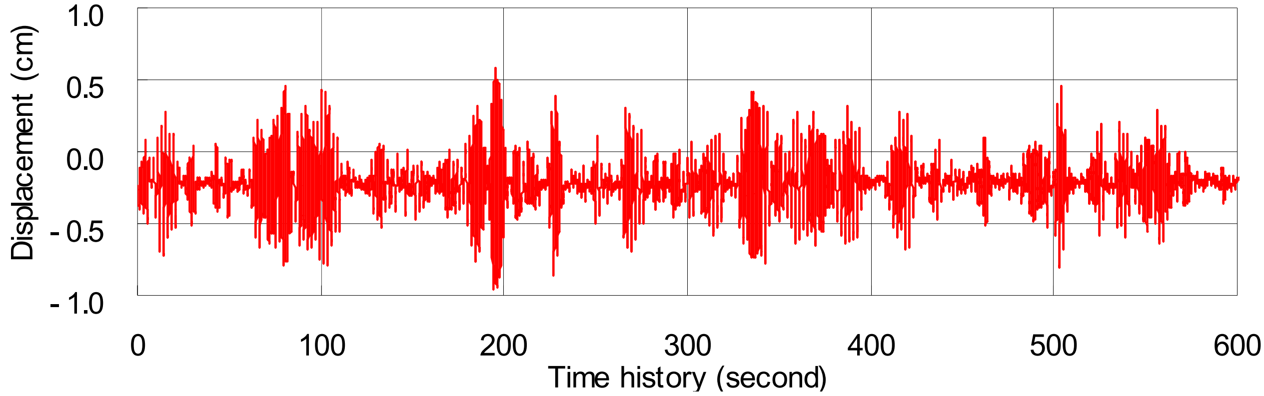


Figure 11 - Displacement of pendulum of TMD-1

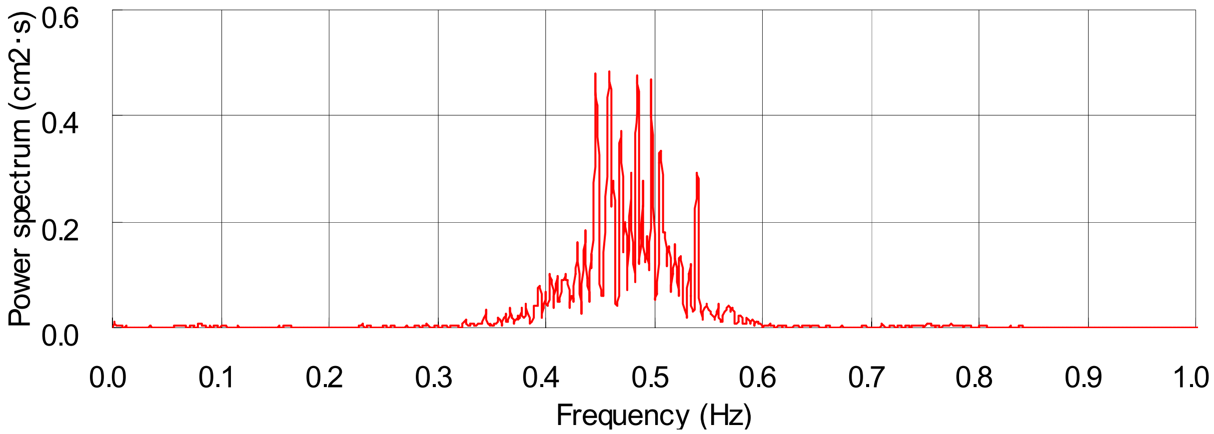


Figure 12 - Power spectrum of displacement of pendulum of TMD-1

5. STUDY ON REDUCTION OF VIBRATION CONTROL DEVICES

5.1. Background of study

As mentioned in the previous chapters, vibration characteristics of the tower are as follows;

- In vibration test for the free standing tower, measured shapes of vibration and their frequencies were almost the same as the design values. But most of measured dampings (logarithmic decrement) were bigger than the design values.
- In structural health monitoring for the free standing tower, resonance wind velocity were almost the same as the design values. But measured amplitudes of the vortex induced oscillation were a little smaller than the results of wind tunnel tests.
- In structural health monitoring after completion of the bridge, measured amplitudes of the vortex induced oscillation were smaller than the design values.

It was clarified that the out-of-plane amplitude by the vortex induced oscillation had been smaller than the design values. The reasons are thought to be as follows;

- Damping performance of the tower itself was possible to be underestimated or various margins of damping performance of the vibration control devices were possible to be overestimated in the original design.
- Exciting force for the tower by wind was possible to be smaller than the design assumption.

Therefore, there is a possibility to reduce vibration control measures for the towers of the Akashi-Kaikyo Bridge.

Meanwhile, aerodynamic stability of the towers is very important for safety of whole bridge and it is necessary to maintain all the vibration control devices sound condition in any time. Ten years have passed since completion of the Akashi-Kaikyo Bridge and it has been revealed that there are many problems in maintenance for the vibration control devices, especially the TMDs. Major problems are as follows;

- It is necessary to overhaul dampers of the vibration control devices periodically. In overhaul, seal and oil have to be changed at a damper manufacturing factory. Interval of overhaul is instructed to be every 5 years in the maintenance manual.
- When the dampers of the TMDs are transported to the damper manufacturing factory, they have to be taken out through narrow maintenance hatch from inside of the tower shafts. Length, diameter and weight of the dampers are about 150 cm, 32 cm and 250kg, respectively. But diameter of the maintenance hatch is only 55 cm. Therefore taking out of the dampers is difficult work and its cost is not cheap.
- As budget of maintenance of the bridge has been demanded to be reduced, maintenance cost of the vibration control devices has also been requested to be reduced.

According to above mentioned reasons, we started to study on reduction of maintenance cost of the vibration control devices of the towers.

5.2. Procedure of study

For reduction of maintenance cost of the vibration control devices, we decided to study on the damping performance of the tower. But study on the exciting force for the tower is excluded from the study issues, because it is very difficult and expensive to measure the exciting force for the tower. And we also decided to study on change of damping performance of the vibration control devices to extend their maintenance interval. Details of the studies are as follows;

5.2.1. Re-evaluation of damping performance of tower

As mentioned in the previous chapter, vibration frequencies of the towers and the TMDs were almost the same as the design values. However, out-of-plane amplitude due to the vortex induced oscillation was very small compared with the design values. One of the reasons was that design assumption was underestimated in the damping of the tower itself. Therefore, we are planning to examine damping performance of the tower itself by stopping some of the vibration control devices. And if the actual damping of the tower itself would be bigger than the design structural damping, it will be possible to make some of the vibration control devices unnecessary. Moreover, we are planning to conduct complex eigenvalue analysis to get precise damping performance of the tower including the vibration control devices. The results will be useful to evaluate necessary number of the TMDs and maintenance interval of the TMDs as mentioned in 5.2.3.

5.2.2. Re-evaluation of allowable vibration amplitude of tower

As for the flexural vibration, the allowable amplitude of the tower at two-third height was decided to be 300 mm based on the assumption of safety side. However, design allowable amplitude was 407 mm. If it would be confirmed that vibration of tower shaft with its amplitude of 407mm have no harmful effect on the bridge, reduction of necessary number of the TMD-1 may be possible.

As for the torsional vibration, the allowable amplitude of the tower was decided by yielding stress of the upper horizontal beam member. However, the upper horizontal beam does not get vertical reaction force from the cables. Therefore, the upper horizontal beam is thought to be repairable even if it would get damages. Furthermore, if stress of the upper horizontal beam would exceed yielding stress, damping of the tower would also increase and vibration amplitude would not increase. If allowable amplitude of the tower could be large, reduction of necessary number of the TMD-2 may be possible.

Therefore, we are planning to conduct pushover analysis using elastic-plastic finite deformation analysis to simulate deformation by the vortex induced oscillation and to study the possibility of changing the allowable amplitude of the vortex induced oscillation of the tower.

5.2.3. Re-evaluation of change of damping performance of vibration control devices

10% change of damping ratio was considered as adjusting error in design of the TMDs as mentioned in 3.3.2. Therefore, damping constant of the oil dampers of the TMDs can be allowable to change within a certain extent. Meanwhile, damping constant of oil damper generally increases due to increase of oil viscosity inside the damper. Figure 13 shows change of damping constant of dampers of the TMD-1. These data were measured before overhaul of the dampers at damper manufacturing factory. Design damping constant of dampers of the TMD-1 is 2.16 kN/ (cm/sec.) and average annual increase of damping constant is +15.2 N/ (cm/sec.)/year. If damping constant is allowable to change within ±10%, allowable change of damping constant of the TMD-1 is 216 N/(cm/sec.). As average annual increase of damping constant is +15.2 N/(cm/sec.)/year, it takes about 14 years for damping constant of the damper to reach the upper limit of allowable change. It indicates that interval of overhaul of the dampers of the TMDs may extend from 5 years which is instructed in the maintenance manual. Therefore, we are planning to evaluate change of damping performance of the vibration control devices to extend their maintenance interval.

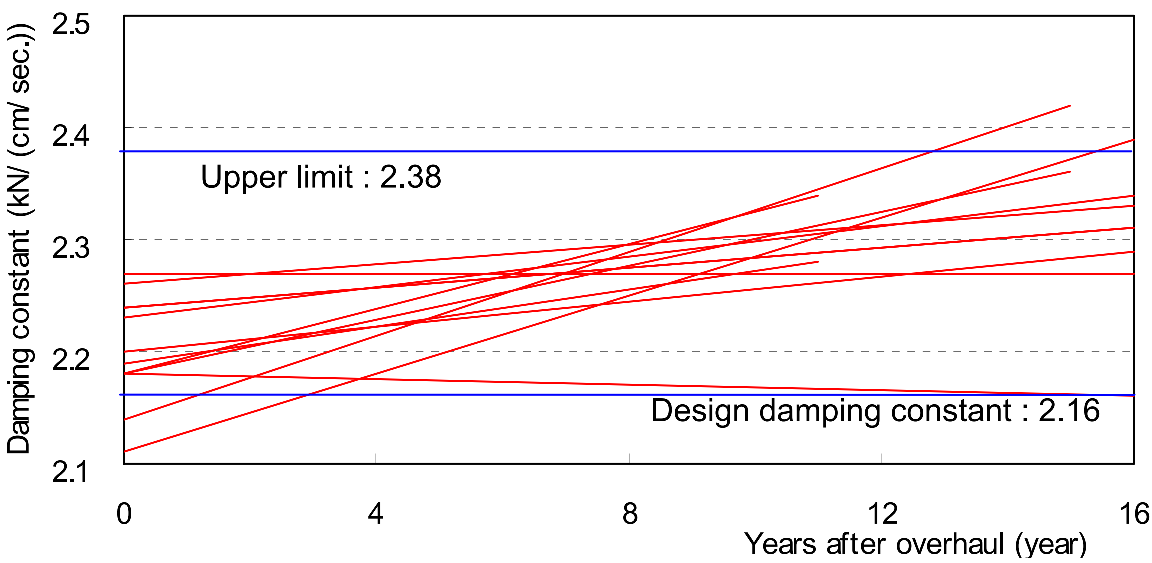


Figure 13 - Change of damping constant of TMD-1

6. CONCLUSION

Approximately ten years have passed since the Akashi-Kaikyo Bridge was completed, and there are various problems and subjects on maintenance of the vibration control devices of the tower. Major problem and subject are as follows;

- Vibration frequencies of the tower and the TMDs were almost same as the design values, but out-of-plane amplitudes of the vortex induced oscillation were small compared with the design values.
- Maintenance cost of the vibration control devices has been becoming larger problem due to budget restriction in maintenance stage.

Cause of small vibration amplitude was thought that damping performance of the tower might be underestimated. And it was thought that some design assumptions for the vibration control devices were possible to be estimated safety side too much. Therefore, we started to study on reduction of maintenance cost of the vibration control devices as follows:

- Examination of damping performance of the tower by stopping some of the vibration control devices, in order to get actual damping of the tower itself.
- Pushover analysis using elastic-plastic finite deformation analysis to simulate deformation by the vortex induced oscillation, in order to decide appropriate allowable vibration amplitude.
- Study on appropriate interval of overhaul of the vibration control devices, in order to extend the interval of overhaul of the vibration control devices.

After the above study will be finished and safety of the bridge will be confirmed without some vibration control devices, maintenance cost will also be able to reduce. As the bridges are designed under limited time and insufficient information at construction stage, design of bridge is not necessarily efficient or economical. However, more efficient maintenance is thought to be possible by re-evaluation of bridge performance based on maintenance data which are obtained after completion of bridge.

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