Structural Monitoring and Maintenance, *Vol. 3, No. 3 (2016) 233-257*
DOI: http://dx.doi.org/10.12989/smm.2016.3.3.233

Japan's experience on long-span bridges monitoring

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(Received February 19, 2016, Revised May 8, 2016, Accepted May 12, 2016)

Abstract. This paper provides an overview on development of long-span bridges monitoring in Japan, with emphasis on monitoring strategies, types of monitoring system, and effective utilization of monitoring data. Because of severe environment condition such as high seismic activity and strong wind, bridge monitoring systems in Japan historically put more emphasis on structural evaluation against extreme events. Monitoring data were used to verify design assumptions, update specifications, and facilitate the efficacy of vibration control system. These were among the first objectives of instrumentation of long-span bridges in a framework of monitoring system in Japan. Later, monitoring systems were also utilized to evaluate structural performance under various environment and loading conditions, and to detect the possible structural deterioration over the age of structures. Monitoring systems are also employed as the basis of investigation and decision making for structural repair and/or retrofit when required. More recent interest has been to further extend application of monitoring to facilitate operation and maintenance, through rationalization of risk and asset management by utilizing monitoring data. The paper describes strategies and several examples of monitoring system and lessons learned from structural monitoring of long-span bridges in Japan.

Keywords: structural monitoring in Japan; long-span bridges; monitoring strategy; wind-induced vibration monitoring; seismic monitoring

1. Introduction

Construction of modern bridge in Japan started by introduction of Western technology around the Meiji Restoration in 1868, after lifting more than two centuries rigorous trade restriction enforced by Tokugawa Shogunate. The use of metal as a structural material began with cast and/or wrought iron used on bridges after the 1870s. The first modern bridge was the Kurogane Bridge in Nagasaki, a riveted cast iron bridge whose span was 27 m completed in 1868. Since 1895, steel has replaced wrought iron as the principal material for metallic bridge. High tensile steels were later adopted for bridge structures after the great Kanto earthquake disaster in 1923. Kiyosu Bridge (length 183 m), an eyebar-chain self-anchored suspension bridge over the Sumida

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http://www.techno-press.org/?journal=smm&subpage=7 ISSN: 2288-6605 (Print), 2288-6613 (Online)

River in Tokyo, is a typical example and a masterpiece among riveted bridges from this era. Construction of reinforced concrete bridges began in the 1900s. Since the 1950s, the use of prestressing technology has spread to nearly every type of simple structural element, and as a result, spans of concrete bridges became long and longer.

Long-span bridges have long been considered important since they constitute critical links connecting islands and circumventing bays in an archipelago country such as Japan. Construction of long-span bridges started after the end of WWII with completion of Saikai Bridge, a steel arch bridge with center span of 243.7 m, in 1955 in Nagasaki. Afterwards, construction of long-span bridges increased exponentially and culminated during the Honshu–Shikoku Bridge Project, a national project to link the Honshu and Shikoku Islands that was started in 1975 and completed in 1999.

The Honshu–Shikoku link has three routes, namely the Kobe–Naruto Route (89 km), Kojima–Sakaide Route (37 km) and Onomichi–Imabara Route (60 km) (Fig. 1). The first route is now part of Kobe-Awaji-Naruto Expressway consisting of two suspension bridges; Akashi Kaikyo Bridge and Ohnaruto Bridge. The Akashi-Kaikyo Bridge, opened to traffic in 1998, has total span length of 3911 m (960 – 1991- 960 m) and currently is the world's longest span bridge. This bridge was originally planned to carry both rail and road traffic, however, in 1985, the plan was changed so that it carries highway traffic only. The other bridge in this route is the Ohnaruto Bridge, completed in 1985 with center span of 876 m. The second route is the Kojima–Sakaide Route that constitutes of three suspension bridges and two cable-stayed bridges. This route carries both roadways (upper-deck), which now is part of Seto-Chuo Expressway and railway traffic (lower deck) that is part of Japan Railway JR Seto-Ohashi Line. Major suspension bridges in this route are: Shimotsui-Seto Bridge (main span 940 m) completed in 1988, Kita Bisan-Seto Bridge (main span 990 m) and Minamai Bisan-Seto Bridge (main span 1100 m). While the other major bridges Iwakurojima Bridge and Hitsuishijima Bridge, both are cable-stayed bridges with center spans of 420 m and side spans of 185 m and were constructed in 1988. The third route is the Onomichi–Imabara Route that carries only the roadway (part of Nishiseto Expressway). The route consists of five suspension bridges and four cable-stayed bridges. Most notable bridge in this route is the Tatara cable-stayed bridge with the main span 890 which was the world's longest span cable-stayed bridge when opened in 1999. Other major bridges are the Kurishima Kaikyo Bridges, three consecutive suspension bridges opened to traffic in 1999 with center spans of 600, 1020, and 1030 m. Other suspension bridges are Innoshima bridge (main span 770 m) and Ohsima Bridge (main span 560 m) completed in 1988. Fig. 1(b) illustrates the development of span length over the years in Japan. Currently there are 15 suspension bridges, 3 cable stayed bridges and a truss bridge with spans over 500 m as listed in Table 1.

Developments in design and construction of long-span bridges have always involved sophisticated models, analysis and cutting edge technologies. Instrumentation for measurement and performance monitoring of uncertainties associated with these new models, analysis and technologies are necessary to ensure the correctness and efficacy of their applications. Moreover many long-span bridges in Japan have been constructed in the seismically active and strong wind areas, consequently structural behaviors during seismic and wind loadings become major concerns. For these reasons, many of long-span bridges have been instrumented and continuously monitored since their construction completions.

Data accumulated from long-span bridges monitoring systems have been used in various ways. In the beginning, they have been used to verify design assumptions, to monitor structure behavior during extreme events and to provide feedbacks for development of design and construction.

These were the original purposes of monitoring in Japan. As bridge spans advance in the 1980s and the 1990s, monitoring systems were supplemented into construction process and control of vibration became common for long span bridge construction. Furthermore, as the structures become more advanced and more monitoring systems are implemented, data of structure behavior during various environment and loading conditions accumulate. The data are used to evaluate possible damage and facilitate decision making on repair and retrofit. Also, monitoring data are used to support maintenance and management of long span bridge stock.

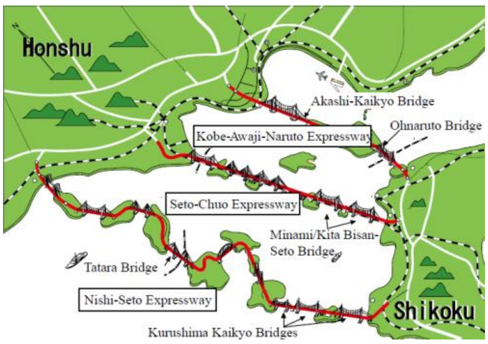
In this paper we describe several strategies and case studies of long-span bridges monitoring in Japan. The paper consists of four parts divided based on monitoring strategy and purpose, namely: (1) monitoring for extreme events and/or retrofit (2) monitoring for design verification and control, (3) long-term monitoring for structural performance evaluation, and (4) monitoring for maintenance and management.

2. Monitoring for extreme events and retrofit purpose

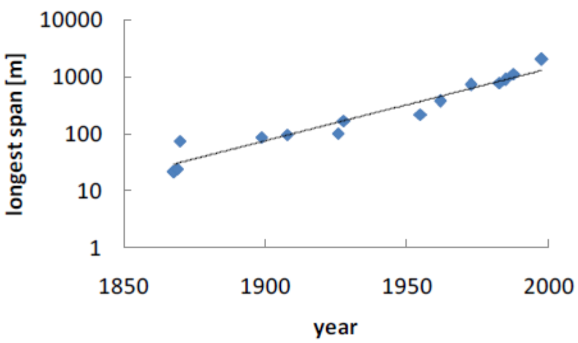
As a consequence of the high intensity of seismic activities in Japan and due to the fact that most part of Japan is under the passage of seasonal strong winds and typhoon, bridge monitoring system in Japan put more emphasis on structural evaluation against these extreme events. For this purpose, bridge instrumentations were focused on environmental and load effects on structures. Typically there are two main extreme environmental events that become the main concern for long-span bridge monitoring, namely, strong winds or typhoons and large earthquakes.

2.1 Monitoring during extreme wind events

In this section we describe several case studies of monitoring during strong wind and typhoon events on long-span bridges. The monitoring results not only provide assurance on bridge performance but also give insight on real structural behavior and unexpected phenomena that require special treatment as the feedbacks for design in the future.



(a) Bridges linking Honshu and Shikoku island



(b) Long-span bridge lengths in Japan over the years

Fig. 1 Development of long-span bridge in Japan over the years

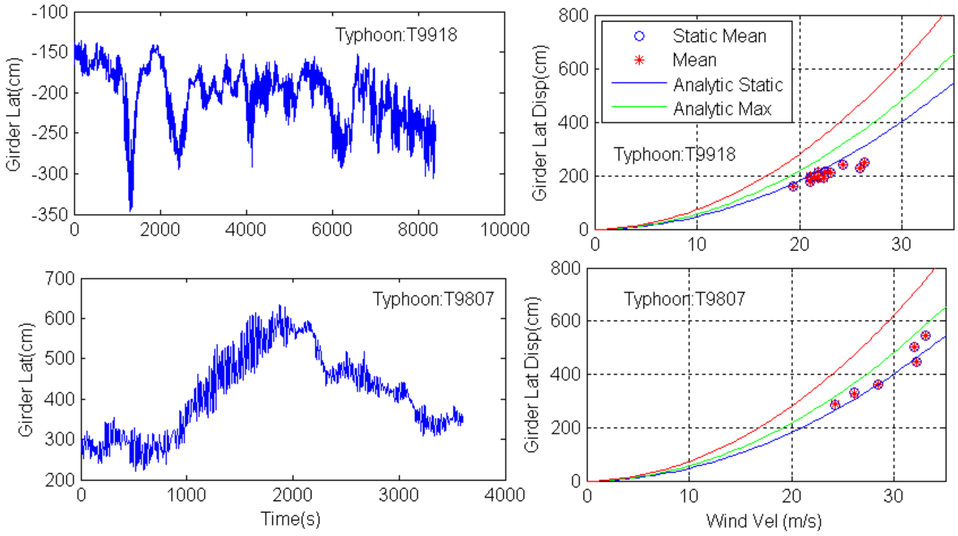


Fig. 2 Lateral displacement measured at the middle of girder of Akashi-Kaikyo Bridge during Typhoon 7 in 1998 (T9807) and Typhoon 18 in 1999 (T9918)

Two and a half year after its opening, a couple of strong typhoons have passed through the Akashi–Kaikyo Bridge. Monitoring system collected data of the bridge response as well as strong winds during the typhoons. Analysis of the full-scale data measured on the Akashi–Kaikyo Bridge during Typhoon 7 in 1998 and Typhoon 18 in 1999, respectively was conducted in the study by (Miyata *et al.* 2002). Focus of the study was on the power spectral density (PSD), spatial correlation of wind-speed fluctuation and response of the deck. The study notes that static lateral deflections of the deck in the middle of the center span are in good agreement with the analytical value when the bridge excited by winds whose directions are nearly normal to the bridge axis (Fig. 2). Meanwhile, analysis of spatial correlation shows discrepancies between the measurement and estimated exponential formula used in design.

In addition to monitoring of deck and tower vibration, observation of parallel hanger ropes of Akashi-Kaikyo Bridge was also conducted. The parallel hanger ropes are employed to suspend the girder from the main cables. Excessive vibration was observed on the ropes of downstream side of the wind, indicating the occurrence of wake induced vibration. Significant vibration was recorded especially during typhoon Vicki and Wald in 1998 and as the result; high damping rubber dampers installed to suppress vortex induced vibration of these ropes were damaged. From video record and wind monitoring, conditions that lead to occurrence of wake induced vibration were investigated and hanger ropes were retrofitted by attaching helical wires to modify their aerodynamic properties (Furuya *et al*. 1998).

Vibration of stayed cables with larger amplitude was observed during construction of the Meiko West Bridge, a cable stayed bridge with central span of 405m. By studying the monitoring data of the bridge for five months, it was observed that the phenomenon appeared only when strong wind and rain occur simultaneously (Hikami 1988). The vibration occurs only on the inclined cables under certain wind velocity range during raining condition. The cable in-plane vibration is visible and its amplitude can reach up to ten times of cable diameter. Vibration frequency involves not only the first mode, but also higher modes. Characteristics of this so called "rain-and-wind-induced vibration are very unique that the classical cable vibration mechanisms such as vortex-induced vibration and wake interference have been disregarded as the sources of mechanism. Hence it was considered as a new type of cable vibration caused by rain and wind.

The occurrences of rain-and-wind induced vibration were observed in many long-span cable-stayed bridges such as Tempozan and Aratsu Bridge (Yoshimura *et al.* 1995) in Japan and many more around the world (see list of complete observation in Fujino and Siringoringo 2013). The vibration has a practical consequence that is fatigue of the cable and damage to the cable anchorage. Control of this vibration becomes serious concern in long-span cable-stayed bridges. Based on the finding, the vibration can now be effectively suppressed either by vibration control of cables or surface treatment of cables.

To investigate the cause of failure associated with wind, monitoring is of critical importance since fluid-structure interaction may trigger rare phenomena which cannot be easily recreated at wind tunnels. In the Hakucho Bridge, unexpected along-wind vibration of the bridge tower and associated girder lateral vibration were observed from monitoring data and the phenomenon was investigated by wind tunnel test (Siringoringo and Fujino 2012). The wind-induced responses of the bridge during six strong-wind events between 2008 and 2012 were studied with the emphasis on tower single frequency large vibration. Significant tower in-plane and the girder lateral vibration were observed in the moderate wind velocity (13-24 m/s), where the tower in-plane accelerations are characterized by single-frequency harmonic-like oscillation at 0.6 Hz and 0.8 Hz. The occurrences of the two oscillations depend upon wind velocity and direction. The tower single-frequency in-plane oscillations affect the girder lateral responses, which were confirmed by comparison of frequency component of tower and girder lateral vibration (Fig. 3). Finite element analyses have shown that the single-frequency oscillation at 0.6 Hz corresponds to the tower's local in-plane in-phase mode, while the oscillation at 0.8 Hz corresponds to the tower's local in-plane out-of-phase mode. The two modes are the tower dominant modes with small motion participation from other components such as girder and cable (Siringoringo and Fujino 2012).

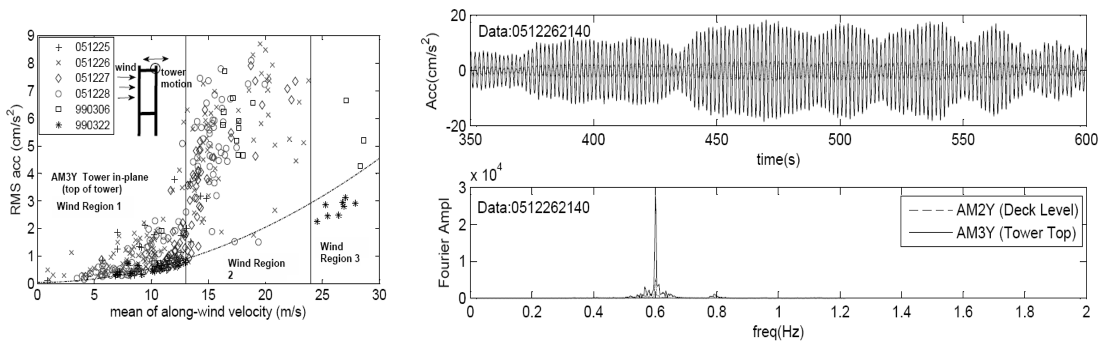
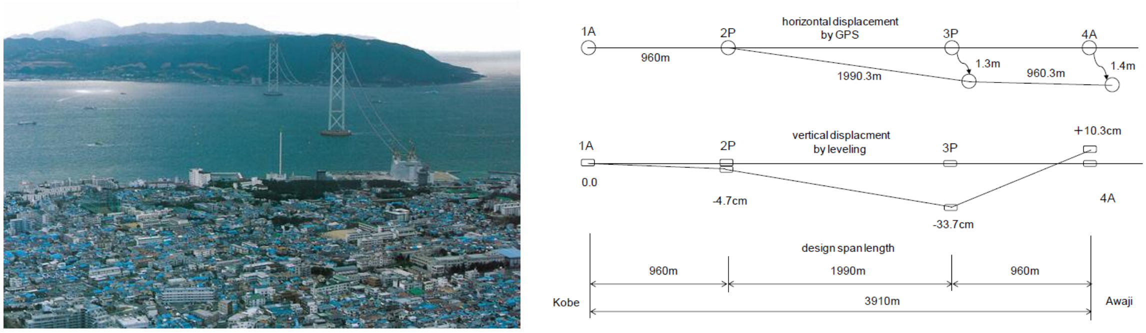


Fig. 3 Along-wind vortex-induced vibration on Hakucho Suspension Bridge tower (Siringoringo and Fujino 2012)



(a) Akashi-Kaikyo Bridge after 1995 Kobe Earthquake

(b) Observed movement after 1995 Kobe Earthquake by GPS and Leveling

Fig. 4 Observed movement at Akashi-Kaikyo Bridge after 1995 Kobe Earthquake

Wind tunnel experiment using 1:20 scale model was carried out to investigate the phenomenon under various wind velocities and angles of attack. The results show that the bluff body of the windward tower cause vortex shedding as indicated by the presence of single frequency dominant oscillation of wind in front of the leeward tower. The vortex shedding created a periodic force towards the leeward leg. In the wind velocity range of 13-17 m/s and 17-24 m/s, and under specific wind angle of attack, the vortex shedding frequency coincides with the tower natural frequency of 0.6 Hz and 0.8 Hz, respectively, causing it to resonate in along-wind direction.

While vortex-induced vibration of bridge tower is not uncommon during freestanding construction stage, the occurrence on a tower of a completed bridge, especially on its strong axis as reported in this study is very rare. Although displacement amplitude of tower vortex-induced vibration is relatively small to cause immediate significant problem, the use of monitoring data shows that this type of vibration still appears despite the presence of cross-sectional corner cut as the countermeasure.

2.2. Monitoring during large earthquakes

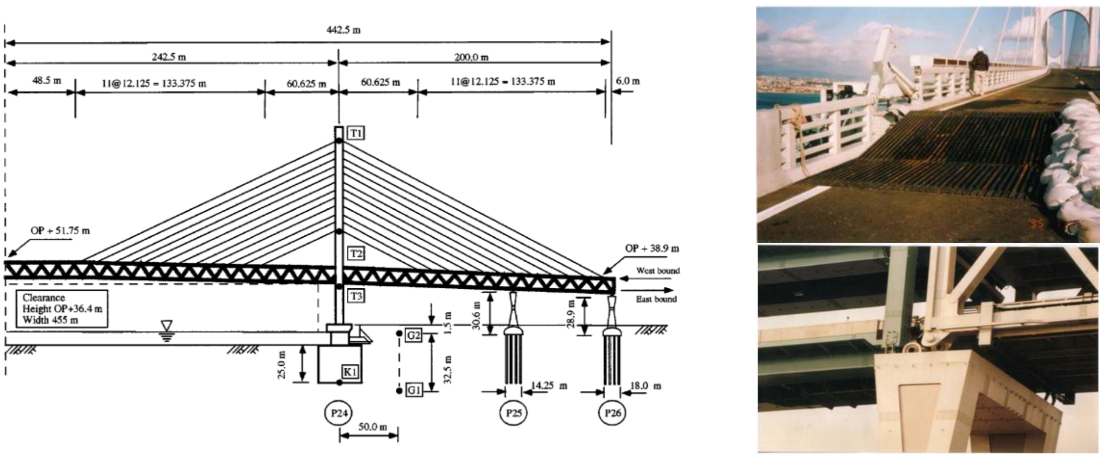
The Akashi-Kaikyo Bridge was still under construction when the 1995 Kobe earthquake occurred as shown in Fig. 4. Construction of abutments and towers had been completed, while part of superstructure was suspended by the main suspension cables. Fault movement occurred right below Akashi Kaikyo Bridge. The base movement of the abutments and pylons due to fault action are shown in Fig. 4(b) (Lin and Uda 1996). GPS monitoring was effective to identify this movement. There was no severe damage observed on the bridge but the planned center span was altered from 1990 m to 1991 m.

Several important records were obtained from the earthquake since the bridge had been partly instrumented with monitoring system. The most important record was the velocity responses recorded on the top of tower P2. The peak velocity was about 1.3m/s with predominant frequency of 0.40 Hz in transverse direction, meanwhile in longitudinal direction the peak velocity is 0.9 m/s with the predominant frequencies of 0.47 Hz (Saeki *et al.* 1997). Based on the measured responses, two analyses were conducted; first was to simulate response of the towers in the earthquake, and to evaluate seismic safety of the completed bridge subjected to the near-field ground motion. This experience shows the importance of monitoring against extreme event even at the early stage of construction.

Another case example of monitoring due to 1995 Kobe earthquake is the analyses conducted for the Minami-Bisan-Seto Bridge and Hitsuishi Bridge at the Kojima–Sakaide of the Honshu-Shikoku Bridge (Okuda *et al.* 1997) and (Yoshizawa *et al*. 2000). Seismic performance of those bridges was investigated based on the recorded seismic response data. Both bridges are of double deck steel trusses and had been in service for seven years when the earthquake occurred. For the Minami-Bisan-Seto Bridge, ground acceleration with the peak of 0.03 g was measured at the ground surface and large amplification of longitudinal acceleration with the peak of 0.3 g occurred at the top of tower (Okuda *et al*. 1997). Bridges dynamic responses were later evaluated in a numerical simulation using ground acceleration applied to all supports.

Higashi-Kobe Bridge (Fig. 5), a cable stayed bridge with central span of 485 m opened 1994, was also located near the strong motion area of 1995 Kobe Earthquake. The bridge's bearing link systems connecting end girders and piers failed as shown in the Fig. 5(b), losing the vertical support to prevent uplift. Failure of wind shoe, whose function was to prevent transverse motion of the girder due to excessive transverse motion of the girder-end, led to the failure of the bridge's bearing link. Fortunately the bridge was supported by intermediate piers at the side spans that provided additional redundancy, so that total collapse was prevented.

The bridge was instrumented and successfully monitored during the earthquake. Girder response contains spike-like wave forms in time history that reflects the collisions of girders and damage to the bearings and the links (Ganev *et al.* 1996). From analysis of recorded seismic responses, it was concluded that pounding occurred between tower and deck, and that response of the soil-structure system strongly influenced by pore-water pressure buildup in the saturated surface soil layers. After this event, uplift prevention cables are applied to similar structural details to add redundancy in case of link failure. In addition, the structure is built on reclaimed land, and liquefaction was observed at the ground motion record. This record, with modification, is used for current design code of highway bridges for evaluation of bridges on the softest ground condition.



(a) Higashi Kobe Bridge Monitoring System (T: Location of sensors) (b) Damage at end-bearing

Fig. 5 Higashi Kobe Bridge and Damage at end-bearing during 1995 Kobe Earthquake

(a) Minato Cantilever Truss Bridge

(b)Movement at the pin connection

Fig. 6 Minato Cantilever Truss Bridge and movement at the pin connection

In the study by Kanaji and Suzuki (2007), similar spiking response was reported at the Minato Bridge during 2004 Kii Peninsula Earthquake (Mw 7.4). The bridge, shown in Fig. 6(a), is a cantilever truss, and connections are placed at central span to suspend the center girder as described in Fig. 6(b). In this case, the spiking behavior is considered due to motion and contact of this connection.

2.3 Monitoring for retrofit purpose

Another important seismic monitoring experience is the monitoring for verification of structural retrofit. In this case, monitoring experience can help to provide insight into retrofit process and verify the efficiency of retrofit actions. This has been the case for Yokohama Bay Bridge. The bridge which is a cable-stayed with central span of 460 m and has been continuously monitored by densely distributed sensor system since 1990. In 2005, a seismic retrofit program was implemented on the bridge for safety assurance of Level 2 earthquake according to Japan's bridge seismic code. The retrofit program considered two types of maximum credible earthquakes: a magnitude 8 far-field or moderately far-field large earthquakes taking place in the subduction zone of the Pacific plate and a near-field inland earthquake occurring beneath the site or close to the site.

The retrofit program utilized previous monitoring results and simulations from identified potential damages for both types of ground motion and concluded that significant damage would occur on the towers and bearings under such excitations. Furthermore, the far-field ground motion would create more damage and induce 1.5 m longitudinal displacement of the girder. Accordingly, five retrofit strategies and fail-safe design concept were introduced (Fujino *et al*. 2005). From the previous seismic monitoring results, it was realized that there is a possibility that Link Bearing Connections (LBC) may not function properly during a large earthquake. In such a case, excessive moment at the bottom of end-pier could be resulted and the LBCs could fail and create uplift deformation at the girder. To prevent such conditions, seismic retrofit of the bridge has been conducted and a fail-safe scenario is provided. As the feedback of the monitoring system, the seismic retrofit of the bridge in 2005 employed a fail-safe design, in which the girder-ends are connected to the footing using prestressed cables to prevent uplift of the girder-end as shown schematically in Fig. 7 (Fujino and Siringoringo 2011).

Yokohama-Bay Bridge responses were also recorded during the 2011 Great East-Japan Tohoku Earthquake (Siringoringo *et al*. 2013). Ground motion lasted more than 3min and the maximum girder horizontal response of 60 cm was observed. Fig. 8 shows the tower transverse acceleration, where periodic spikes resembling impulses are observed. Periodic spikes indicate occurrence of transverse pounding between tower and girder, and the spikes appear on the girder vertical accelerations as well. By observing time interval between two successive spikes, the structural mode that triggers the impulses was estimated. It was concluded that the girder first transverse mode triggered the pounding, since the average time interval between two consecutive spikes was observed to be around 3.2sec (0.31~0.32 Hz).

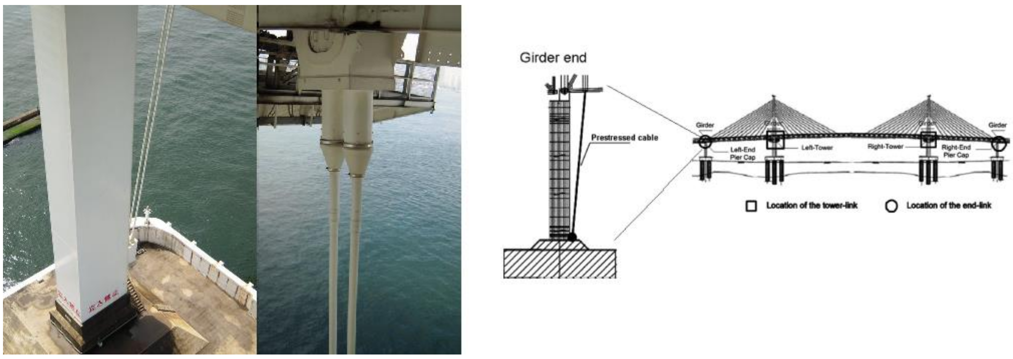


Fig. 7 Photos and schematic figure of a fail-safe design system using pre-stressed cable that connects girder-end and the ground to prevent uplift at Yokohama-Bay Bridge

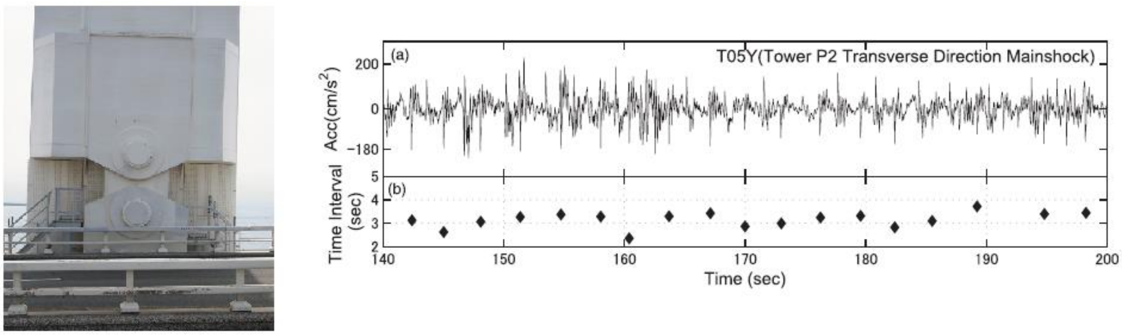


Fig. 8 Photo of LBC at tower of Yokohama-Bay Bridge and Monitoring system and observed seismic record at 2011 Tohoku Eartthquake. (a) accelerations of tower at deck level during the main shock showing spike indicating impulses (b) time interval between successive lateral poundings (Siringoringo *et al*. 2013)

Fig. 9 Photos Rollover incident of a cargo truck on the lower deck of Yokohama-Bay Bridge during the March 11, 2011 Great East Japan Earthquake

In the monitoring of long span bridges subjected to strong motion, the above mentioned spiky wave forms are often and repeatedly reported and these are not well-treated in conventional seismic design. Monitoring, especially densely distributed monitoring, would be required to further investigate this phenomenon and to develop appropriate modeling and countermeasures.

During the 2011 Great East-Japan Tohoku Earthquake, the Yokohama Bay Bridge was closed for about 30 hours due to an overturned cargo truck accident on the lower deck as shown in Fig. 9. The girder transverse vibration caused the truck to become unbalanced and overturned. Even if no major structural damage is observed, traffic disruption due to excessive vibration could cause human loss. As wind monitoring data is used to regulate traffic at strong wind conditions, earthquake monitoring and early warning data should also be incorporated into traffic control to reduce this risk.

3. Monitoring for design verification and structure control

Dynamic performance is an important consideration in long-span bridge design. Because of their flexibility and low damping, various types of vibration from different sources of excitation could occur during the lifetime of a long-span bridge. Aerodynamic stability and seismic response are special concerns in design, hence dynamic testing at completion was common at the early stage of development of long-span bridges.

3.1 Monitoring for design verification

In the beginning of development of long-span bridges in Japan, issues associated with quantification of external forces, especially wind loading was significant in design process. Limited experiences from the past and large uncertainties within design assumptions made verification in the scaled experimental testing an important design step. Fig. 10(a) shows an example of large scale one-tenth sectional bridge girder model constructed and operated from 1973 to 1975 to verify wind resistant design method for the Honshu-Shikoku Bridge Project that includes the Akashi-Kaikyo Bridge. The observed response is compared with estimation based upon wind tunnel testing as shown by Fig. 10(b) (Okauchi 1978).

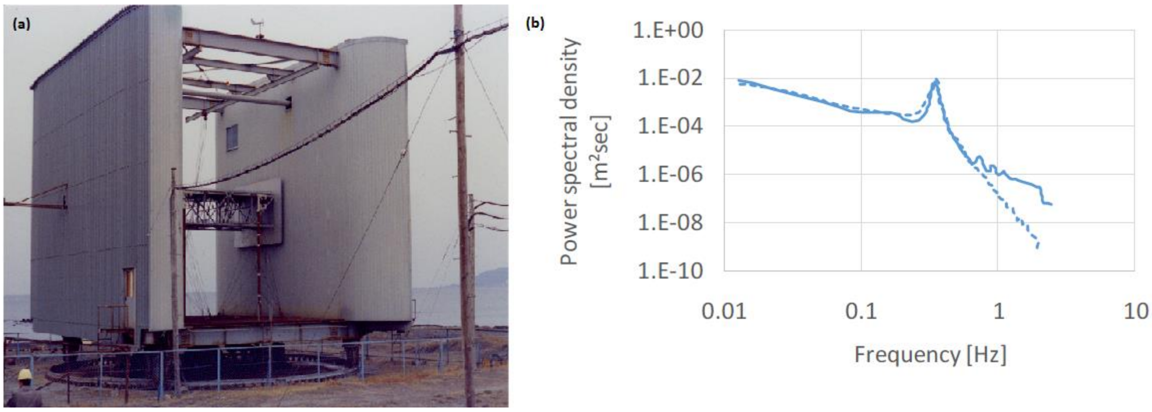


Fig. 10 (a) Large scale bridge model (courtesy of Honshu-Shikoku Bridge Authority) and (b) Comparison between in-situ model response and estimation based upon wind tunnel experiment (average wind speed 12.6 m/sec, elevation 0^{o}). Solid line: observation; dashed line: estimation

These attempts at earlier stage have developed to monitoring with permanent measurement installation during service life as sensor technology and information system advance. For example, instrumentation at Akashi-Kaikyo Bridge at the time of completion as shown in Fig. 11 (Abé and Fujino 2010). Fig. 12 shows an example of relationship between average wind speed and horizontal displacement measured by GPS. Displacement of the bridge can be measured with reasonable accuracy by GPS due to the prominent length of its span. The observed values are close to design average values and the maximum values are conservative with reasonable margin (Toyama *et al.* 2006). Additionally, measured data of power spectrum, turbulence intensity, and spatial correlation of natural wind at various long-span bridges in Honshu-Shikoku Bridge Project are studied and verified with design assumptions within reasonable conservative margins (Katsuchi *et al.* 1996, Miyata *et al.* 2002).

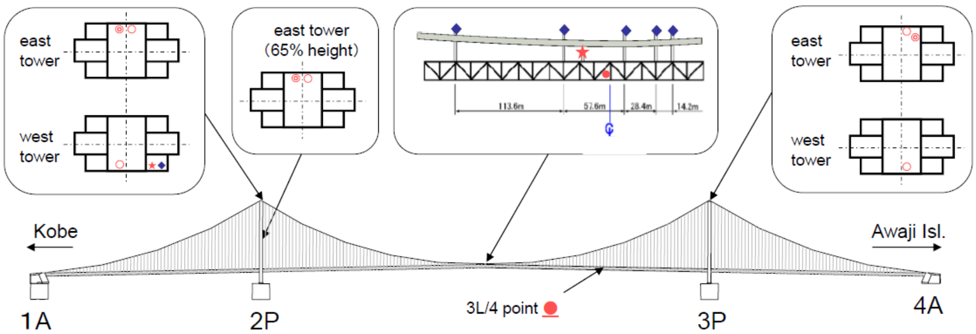
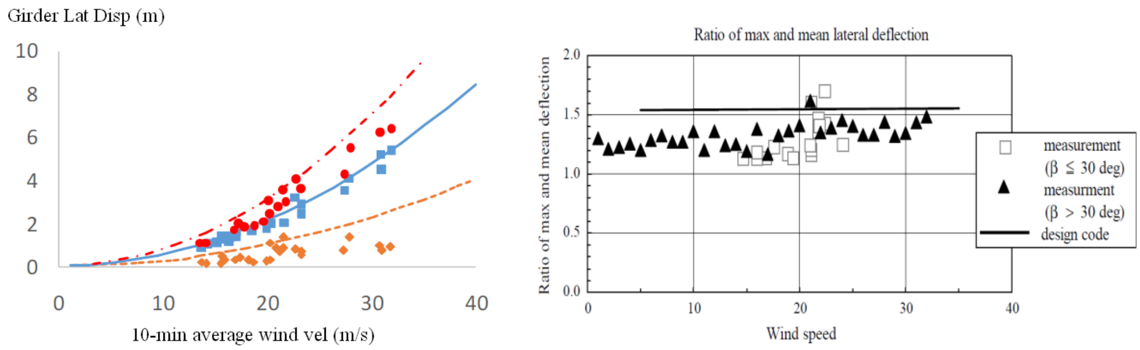


Fig. 11 Monitoring system for Akashi-Kaikyo Bridge (◆:anemometer, ★:GPS, ○:velocimeter(longitudinal); ◎:velocimeter(transverse);



(a) Relationship between girder Lateral displacement and wind velocity

(b) Comparison of measurement and design code

Fig. 12 Relationship between wind speed and girder lateral response at Akashi-Kaikyo Bridge

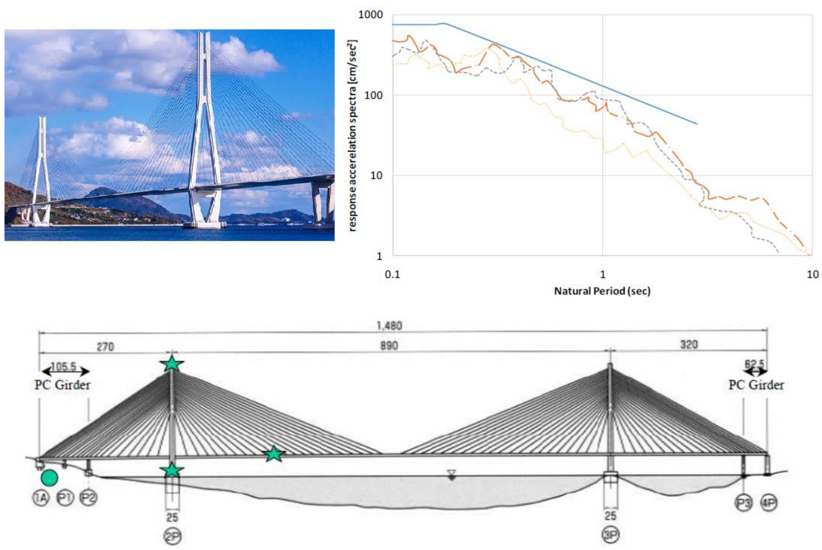
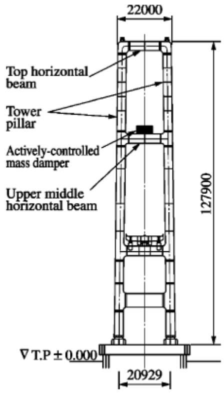


Fig. 13 Monitoring system for Tatara Bridge (●:seismometer; ★:accelerometer) and Comparison between design spectra (straight line) and observed response spectra in the 2001 Geiyo Earthquake

Observed seismic response was also used to verify seismic design. One example of such case is the Tatara Bridge (Fig. 13) the longest cable stayed bridge in Japan that was strongly excited by 2001 Geiyo Earthquake (Mw 6.7). The maximum ground acceleration at the bridge site was 144 cm/sec^2. From observation of seismic response it was revealed that actual seismic load in term of response spectra calculated from recorded ground motion was below the design specification as shown in Fig. 13. Seismic behavior of the bridge was studied by simulation analysis to verify the structural model and assumptions using the observed ground motions. The simulation results were found to be in good agreement with observed responses (Kawato *et al*. 2005).

(a) Pendulum type control device



(b) Location of installation at Hakucho Bridge tower

Fig. 14 Vibration control system of Hakucho Bridge tower (Fujino *et al.* 2012)

Another important problem of engineering interest in seismic analysis of long span bridges is the spatial variation of seismic ground motions. The variation is a result of time lag of seismic wave propagation since foundations of long span bridges are significantly apart by the long span. One such analysis was performed on Onaruto Bridge using observed response during the 1995 Kobe Earthquake (Mw 6.9). The study revealed that spatial variation of ground motions increased vertical response of the girder (Yoshida 1999). Similar tendency of increase of vertical girder response was also observed at other long-span bridges, for instance, the Akinada Bridge during the 2001 Geiyo Earthquake (Ogiwara *et al*. 2001).

Also observed at the 2001 Geiyo Earthquake was the center stay rod failure of the First Kurushima Kaikyo Bridge (Abé and Fujino 2009). Observed seismic ground motion was applied to dynamic three dimensional finite element dynamic analysis, and it was verified that the failed center stay rods performed as they had been designed for. Lesson learned from this experience is that reanalysis of observed data obtained from extreme events can provide valuable information to verify and update the design.

Another important case of design verification is estimation of appropriate damping values and mechanisms. Estimation of damping value and mechanism has been quite difficult because of complexity of mechanism involved and sensitivity of the estimated values to excitation condition. Nevertheless, there have been some studies that use seismic records of instrumented long-span bridges to clarify damping mechanisms and estimate the value. For example, Kawashima et al. (1991, 1992) utilized seismic records from over 33 earthquakes on the Suigo Bridge, a 290.45 m-long steel box girder two-span continuous cable stayed bridge, to clarify the damping characteristics of tower and deck. It was found that damping ratio correlates with the measured accelerations depending on the structural components and the direction of excitations.

At Tsurumi Tsubasa Bridge, strong motion observation has been performed from the opening of the bridge and several records of significant earthquakes have been obtained. During the October 23, 2004 Niigata Chuetsu earthquake, the records show that vibration continued over a long time that meant the damping in the response displacement amplitude was small (Yamamoto *et al.* 2009). Utilizing seismic records from ten earthquakes on Yokohama-Bay Bridge it was found (Siringoringo and Fujino 2006) that damping ratios for lower modes in both vertical and lateral direction has an increasing trend with the increase of earthquake magnitude. For small magnitude, average damping ratios are found to be 2% and increase significantly up to 4-5% as the earthquake magnitude increase, which is larger than previously suggested 2%.

3.2 Monitoring for structure control

Vibration control of long span bridges is commonly applied to suppress wind induced vibration. Conventional methods are passive vibration control, such as oil dampers for girder motion or tuned mass dampers for tower oscillation. As structures become larger and more flexible, larger capacity is required for control device, and active control, which introduces artificial external force to vibration suppression, becomes an attractive option. Because active control naturally requires measurement to modulate control force, monitoring forms the basis of this new technology.

Table 1 List of bridges in Japan with longest span over 500 m (Fujino *et al*. 2012)

Bridge Name	Year Completion	Longest Span (m)	Bridge Type	Remarks
Akashi Kaikyo	1998	1991	Suspension	Active control during construction/wake galloping of hangers, dry air injection for cable, permanent monitoring system
Minami Bisan Seto	1988	1100	Suspension	Dry air injection for cable
Third Kurushima Kaikyo	1999	1020	Suspension	Active control during construction / dry air injection for cable
Kita Bisan Seto	1988	990	Suspension	Dry air injection for cable
Shimotsui Seto	1988	940	Suspension	Dry air injection for cable
Tatara	1999	890	Cable stayed	
Onaruto	1985	876	Suspension	Dry air injection for cable
Innoshima	1983	770	Suspension	Dry air injection for cable
Akinada	2000	750	Suspension	Dry air injection for cable
Hakucho	1998	720	Suspension	Active control during Construction, dry air injection for cable, permanent monitoring system
Kanmon	1973	712	Suspension	During rehabilitation
First Kurushima Kaikyo	1999	600	Suspension	Active control during Construction, dry air injection for cable, seismic damage at Geiyo earthquake
Meiko Central	1998	590	Cable stayed	Active control during construction
Rainbow (Tokyo Port)	1993	570	Suspension	Active control during Construction, dry air injection for cable, seismic monitoring system
Oshima	1988	560	Suspension	Dry air injection for cable
Toyoshima	2008	540	Suspension	Dry air injection for cable

Practically, active control is considered superior to passive devices when, 1) multiple vibration modes are present; 2) natural frequencies change as typically observed during construction; and 3) installation space is limited thus compact devices are preferred. These three conditions apply to flexible long span bridges, especially, at the construction stage. Hakucho Bridge is one of the bridges controlled actively during construction. Pendulum type control device shown in Fig. 14(a) was installed near the top of the tower as illustrated in Fig. 14(b) (Matsuda 2002). The system is so called "hybrid" system, which incorporates both passive control effects by the pendulum motion and active control force provided by rack and pinion with electric motors.

Monitoring is also important to ensure that control systems deployed on long-span bridges function as intended and to give feedbacks on efficacy of the control system performance. Long span bridges, at which active control is applied during construction, are remarked in Table 1.

4. Long-term monitoring for structural performance evaluation

As continuous monitoring data accumulates, it has proven to be especially useful to investigate structural performance during different kinds of environment and various levels of loading conditions. In the following sections, we shall describe several case studies of long-term monitoring of long-span bridges and the lessons learned from the experiences.

4.1 Monitoring for wind-induced structural response evaluation

During daily operations, bridge responses subjected to various levels of wind and earthquakes provide an insight into real structural performance during common and extreme events. We describe briefly in this section two cases studies of continuous monitoring of wind excitation of a long-span suspension bridge and continuous seismic monitoring of a cable-stayed bridge.

The first case involves the Hakucho Suspension Bridge –a three-span suspension bridge with the total length of 1380 m (330 m-720 m-330 m). After the construction completion, series of dynamic tests were performed, including ambient vibration test. Densely distributed accelerometers were placed at the spacing of 30m on the main span and of 55 m on the side span near the Jinya approach (Fig. 15). The measured wind-induced structure responses clearly show the quadratic relationship between wind velocity and response. By applying structural identification method consists of two steps: identification of vibration modes and inverse analysis of structural properties from the identified modes, bridge performance during ambient vibration and strong wind was evaluated (Nagayama *et al*. 2005, Siringoringo and Fujino 2008a).

Results show that in general the natural frequencies decrease as the wind velocities increase and damping ratios increase as the wind velocities increase. The decrease and increase of natural frequencies and damping ratios are more apparent in the low-order modes as evident by the slopes of the linear trend. The mode shape components reveal two distinctive trends. The real parts of mode shape vectors do not exhibit a distinct trend, indicating no obvious changes of modes. The modal phase angle computed from the imaginary part of the mode shape, however, revealed a clear trend. It was observed that the phase difference is large when the root-mean-square of acceleration is very small and decreases when the acceleration is large. The phase differences indicate that the system is non-proportionally damped. The locality effect of phase difference that was concentrated mainly at the edge of girder suggests the contribution of additional damping and stiffness caused by friction force at the bearings (Nagayama *et al.* 2005). In addition, the decrease and increase of natural frequencies and damping ratios indicated the effect of aerodynamic force along the girder.

By employing inverse analysis, contribution of aerodynamic and friction force with respect to wind velocity were quantified. The results suggest the contribution of aerodynamic force was much smaller than the effect of friction force at the bearing. The aerodynamic force contribution is in the order of one-percent when compared to the contribution of the friction force, and its behavior is in agreement with the aerodynamic force obtained from wind tunnel results (Fig. 16). Furthermore, the additional damping and stiffness due to friction force display clear trends, that is small damping and large stiffness during small vibration. When the wind speed increases the damping is also increases, which is when the bearings are unstuck, whereas the stiffness is decreasing as the result of increasing flexibility of the structure.

4.2 Monitoring for seismic-induced structural response evaluation

Monitoring for seismic response has been widely employed for decades in Japan especially for the bridge with special features such as curved bridge (Siringoringo and Fujino 2007) and bridges with new technology such as seismic isolation systems (PWRI 2014). Long-span bridges in Metropolitan Expressway network that connects Tokyo and Yokohama are important bridges consisting of three long-span bridges: Yokohama-Bay Bridge (Fig. 17), Rainbow Bridge (Fig. 18) and Tsurumi-Tsubasa Bridge (Fig. 19) and they all have permanent seismic monitoring systems. Study on applications of earthquake-induced response for system identification methodology to investigate structural behavior subjected to the 2004 Chuetsu-Niigata earthquake (Mw 6.8) was conducted on Yokohama-Bay Bridge, Rainbow Bridge and Tsurumi Fairway Bridge (Siringoringo and Fujino 2008b).

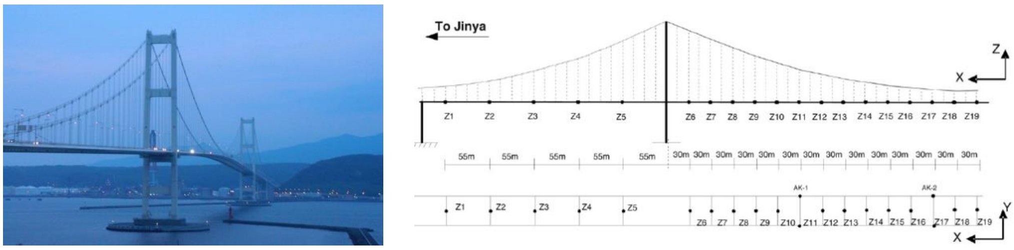


Fig. 15 Hakucho Suspension Bridge and sensor layout for ambient vibration measurement (Siringoringo and Fujino 2008a)

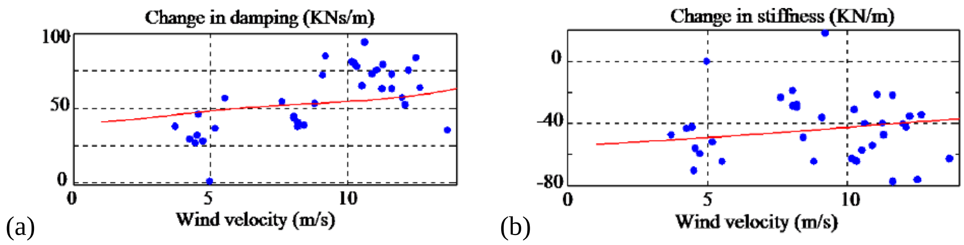


Fig. 16 Hakucho Suspension Bridge identified change in (a) aerodynamic damping and (b) stiffness with respect to wind velocity (Nagayama *et al*. 2005)

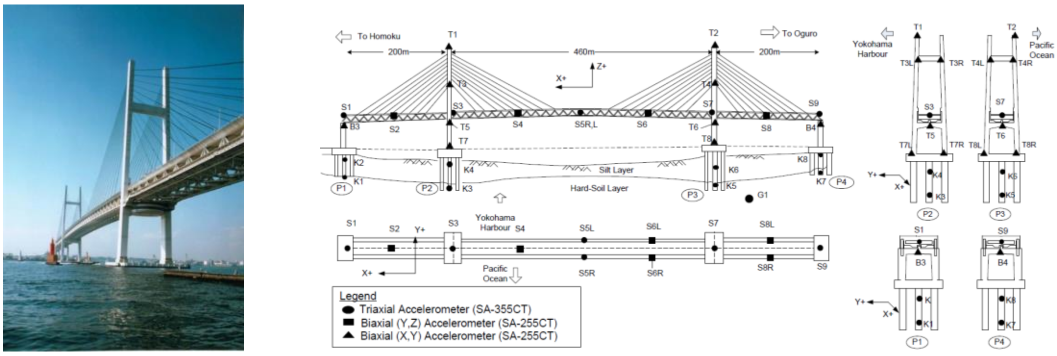


Fig. 17 Yokohama Bay Cable-Stayed Bridge sensor layout for permanent monitoring system (Siringoringo and Fujino 2008b)

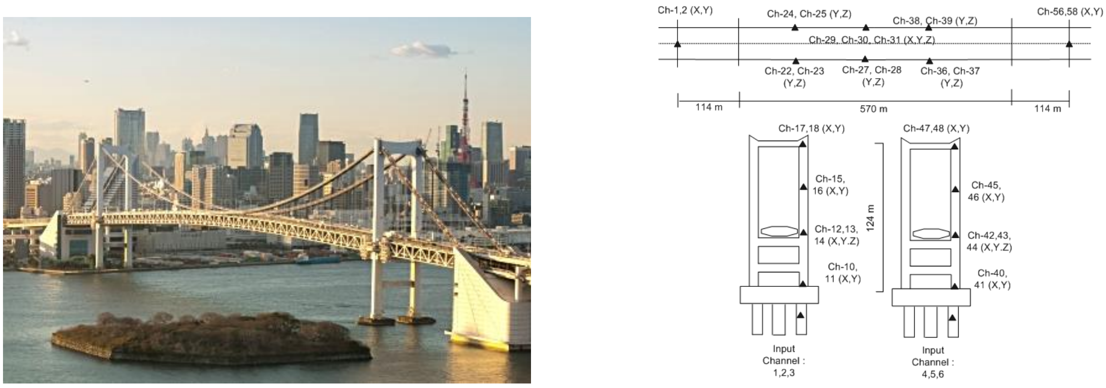


Fig. 18 Rainbow Bridge Tokyo and sensor layout for permanent monitoring system (Siringoringo and Fujino 2008b)

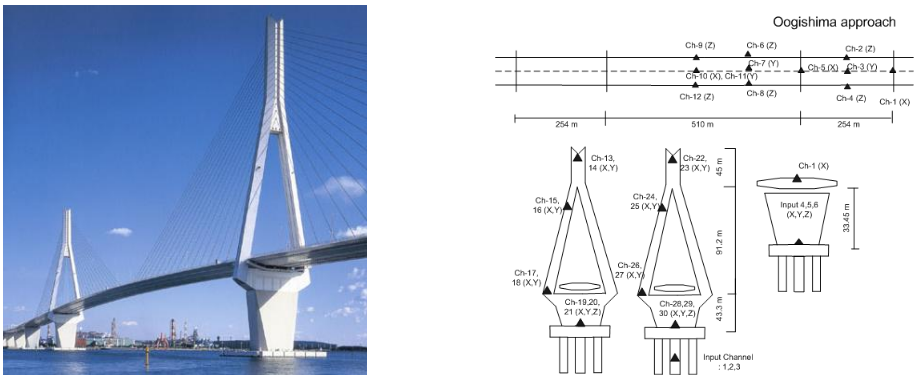


Fig. 19 Tsurumi-Tsubasa Bridge and sensor layout for permanent monitoring system (Siringoringo and Fujino 2008b)

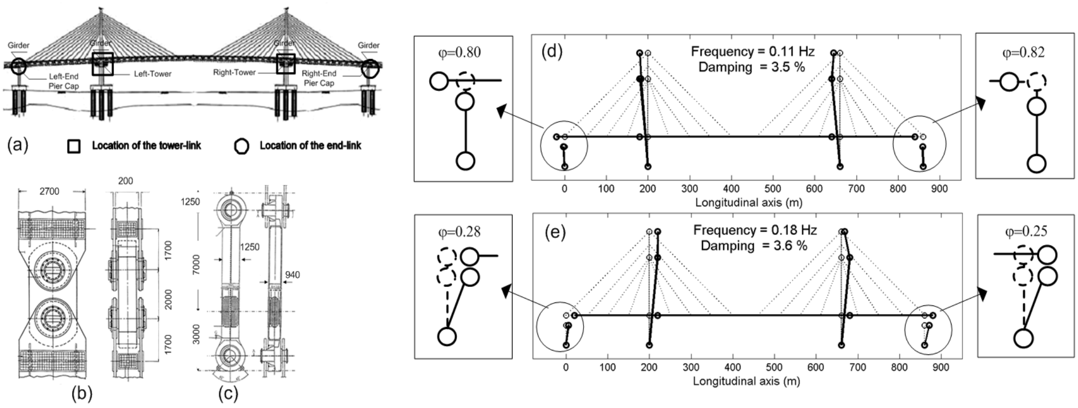


Fig. 20 Location of link bearing connection of Yokohama Bay Bridge. (b) Typical LBC at the tower, (c) typical LBC at end piers. Two of the three typical first modes of Yokohama Bay Bridge identified from the main shock at 17:57 (d) Hinged-hinged mode (e) Fixed-fixed mode (Siringoringo and Fujino 2008b)

The Yokohama-Bay Bridge was constructed on the soft soil is an example of where a special seismic isolation system was needed. It is located near an active fault and close to the epicenter of the 1923 Great Kanto Earthquake. These conditions have made seismic performance a major concern. Therefore to confirm the seismic design and to monitor the bridge performance during earthquake, a comprehensive and dense array monitoring system was installed. The objectives of the monitoring system are to evaluate seismic performance, verification and comparison with seismic design, and observation of possible damage. Particular attentions are given to the local component that is seismic isolation device in the form of Link Bearing Connection (LBC).

As part of a dynamic monitoring system, the bridge is equipped with 85 channels of accelerometers at 36 locations (Fig. 17). Seismic records with varying amplitude obtained from six major earthquakes from 1990 to 1997 were analyzed to evaluate global and local performance of the bridge (Siringoringo and Fujino 2006). System identification of the long-span bridge under seismic excitation requires that non-unique ground excitation records measured along the bridge and excitation in multiple directions be taken into account. Investigation on performance of LBCs of the bridge was carried out using the records from fourteen earthquake frames. The analysis involves system identification especially by observing the first longitudinal mode, analysis of the response between pier-caps and girder, and analysis using finite element model. Based on the analysis, the following findings are obtained (Fig. 20):

1. Three typical first longitudinal modes were found from system identification with the main focus on the relative modal displacement between end-piers and girder. They are: the hinged-hinged mode, mixed hinged-fixed mode and the fixed-fixed mode. The latter two modes are variations of what was highly expected mechanism (hinged–hinged mode). The response analysis of relative displacement between the end-piers and girder confirms these findings.
2. During small earthquake the LBC has yet to function as a full-hinged connection. Therefore higher natural frequencies due to the stiffer connection were observed. The mixed hinged-fixed mode was observed during moderate earthquake. The full-hinged connections at both of the end-piers were observed mostly during large earthquakes.

The above example shows that long-term seismic monitoring provides opportunity to observe the different structural performance during various levels of earthquakes. It should be mentioned however, that in many cases number of sensors is often insufficient to capture the seismic response of bridges. This is because in the past, sensors were installed mainly to determine aerodynamic stability; hence they are insufficient for recording earthquake response of structures. Also, in regards to seismic performance evaluation, we need to measure other parameters that are critical and important in evaluating the seismic performance and safety of a bridge and not only accelerations and velocities.

5. Monitoring for maintenance and management of bridge asset

Monitoring application to maintenance and management of existing structures is now being explored in the consequence of increasing aging stock. One of the earlier examples is monitoring during Wakato Bridge rehabilitation in 1990 as a part of roadway expansion (Ishii *et al.* 1991). Wakato Bridge is the first long span suspension bridge in Japan with central span of 367 m built in 1962 (Fig. 21(a)). Although not permanent, displacement of critical positions and representative vibration modes are monitored during rehabilitation and expansion of the bridge width. Because original measurement data at completion stage were also available, structural conditions can readily be compared with the original condition. Fig. 22(b) shows the displacement measurement points and corresponding measurements at various rehabilitation stages by optical surveying. Measured displacement values were below design values, and symmetric movement between Tobata and Wakamatsu sides, which implies that internal forces would have remained in balance during rehabilitation. Natural frequencies remained either unchanged or slight stiffening. Hence no major damage or softening that could influence aerodynamic stability was expected.

At Akashi Kaikyo Bridge, deformation during strong wind was observed during 2003 Typhoon Etau as shown in Fig. 23(a). Transverse displacements measured by GPS, accelerations, and wind speeds at the center of the girder are measured simultaneously. Because acceleration measurement can be applied to various structures with relatively arbitrary configuration and placement compared to conventional displacement measurement which requires certain fixed point for reference, estimation of displacement by acceleration measurement is studied using this data. The observed displacement values are successfully estimated by random vibration based on arithmetic method from accelerometer record proposed by the authors as shown in Fig. 21(b) (Abe and Fujino 2010). The proposed method enhances application of structure evaluation and monitoring by displacement, which is the major performance index.

Also at the Akashi Kaikyo Bridge, dry-air injection system, whose conceptual drawing is given in Fig. 24(a), is introduced to protect main cables from corrosion (Kitagawa *et al.* 2001). Monitoring of moisture in the cable is required to control quality of the air. Fig. 24(b) shows comparison of moisture between outside air and inside cable, indicating advantage of the system.

Environmental measurement is becoming more and more important for evaluation of structural durability. One of the challenging problems for monitoring of long span bridges are evaluation of scour. At the Akashi Kaikyo Bridge, for example, ultrasonic sounding by survey ship is employed every 2 years, and so far, no severe scour is observed (Kurino and Yumiyama 2002). However, it should be realized that natural disasters which could cause scour may occur during interval of ultrasonic survey so that development of continuous/permanent monitoring is desired. Conventional monitoring system designed for dynamic behavior during earthquake and/or wind are proven to be useful, and further development to match needs for maintenance and management would be required for rational management of long span bridges.

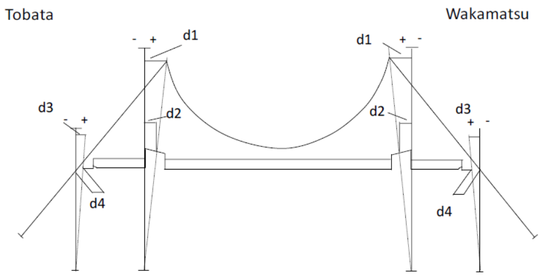


Fig. 21 Deformation measurement at Wakato Bridge (Note: d1: at the top of tower, d2: middle of tower, d3:top of pier; d4: end of the girder)

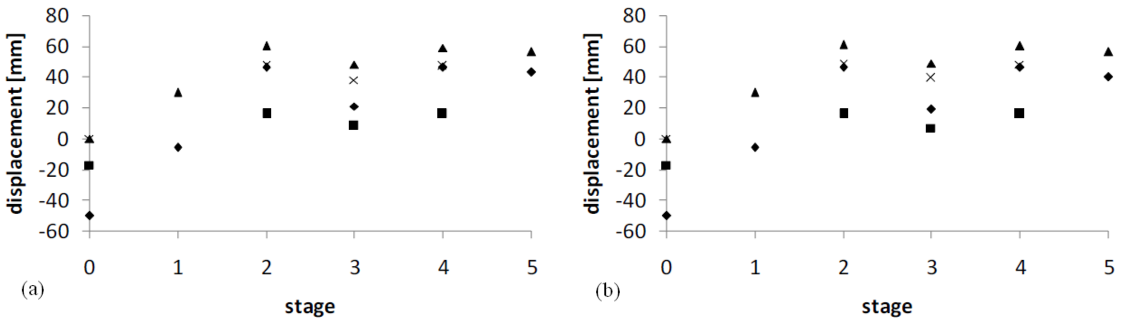


Fig. 22 Deformation measurement at the Wakato Bridge (a) Tobata side (b) Wakamatsu side Stage 0: design value; stage 1: at completion; stage 2: before rehabilitation; stage 3: at removal of half of central span; stage 4: design value after rehabilitation, stage 5: after rehabilitation. ◆:d1; ■:d2; ▲:d3; ×:d4

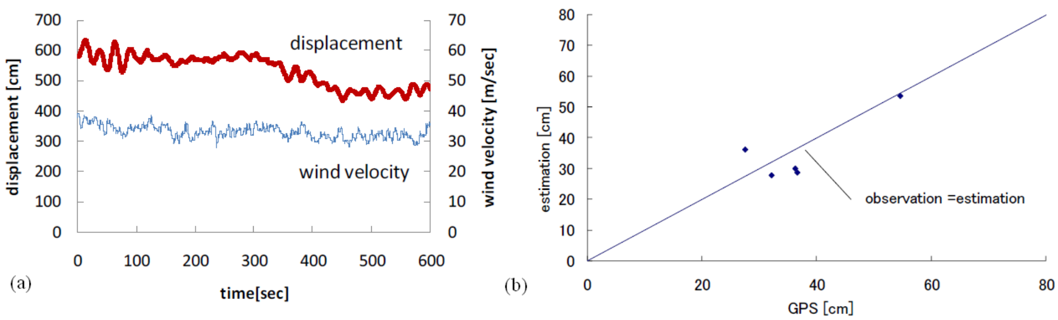


Fig. 23 Performance of Akashi-Kaikyo Bridge during typhoon Etau (a) displacement and wind velocity and (b) displacement estimation by accelerometer

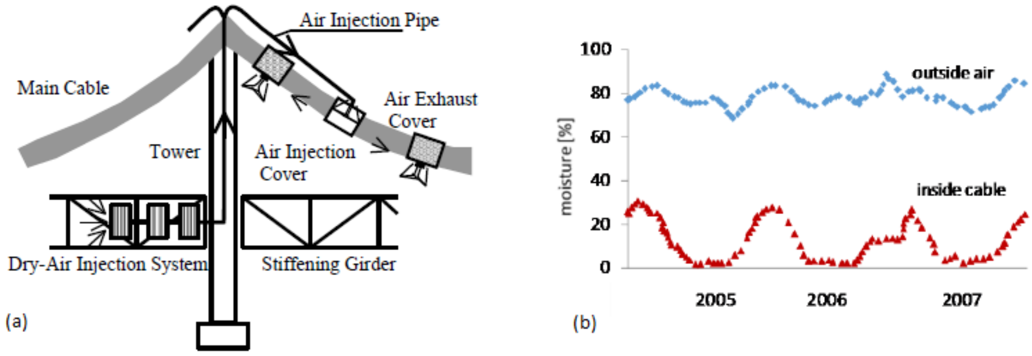


Fig. 24 (a) Dry air injection system for Akashi-Kaikyo Bridge and (b) Moisture monitoring of main cable

Fatigue damage on steel members is also an important issue for long-span bridges especially the ones that carry railway. This factor was carefully examined ever since the design and fabrication stage of the bridges at the Kojima–Sakaide Route of the Honshu-Shikoku bridge link. Large number of axle loading was expected for bridges during the service life of bridges in this route since they support the railway lines. The bridges employ high-strength steel, which is sensitive to fatigue, to primary members. Honshu-Shikoku Bridge Expressway authority has developed a welding methodology as well as a welding inspection system called "AUT: Automated Ultrasonic Testing system" (Okuda *et al.* 2010). The system was effective in detecting welding defect in the factories. Several improvements on the system have been made before it was deployed on-site for inspection so that the system can be available to inspect existing welding lines (Fig. 25). Two inspections were conducted by year 2000 and there was no sign of crack progress found on the bridges. This was attributed to the fact that actual load of train is smaller than the design assumption (Okawa and Kurihara 2010).

6. Conclusions

This paper describes development of structural monitoring of long span bridges in Japan. Monitoring strategy and purpose can be broadly classified into four categories namely: (1) monitoring for extreme events and/or retrofit, (2) monitoring for design verification and control, (3) long-term monitoring for structural performance evaluation, and (4) monitoring for maintenance and management of bridge asset.

Historically, bridge monitoring in Japan put more emphasis on structural evaluation against extreme events, which is a rational choice considering the severe environment conditions such as high seismic activity and strong wind. In the beginning of development of bridge monitoring system, monitoring data were utilized to verify design assumptions, update specifications, and facilitate the efficacy of vibration control system. These were among the first objectives of instrumentation of long-span bridges in a framework of monitoring system. Later, monitoring systems were employed to evaluate structural performance under various environment and loading conditions, and to detect the possible structural deterioration over the age of structures. Monitoring system can also be employed to form basis of investigation of causes and judgment for required repair and/or retrofitting. More recent interest in monitoring has been to further extend application of monitoring to operation and maintenance, through rationalization of risk and asset management by monitored data.

Representative examples of monitoring by densely distributed sensor arrays are described in this paper. Monitoring has evolved to continuous and permanent installation throughout life time of structures, where continuous monitoring provides data of structure behavior during under various loading and environment condition including unexpected extreme events. They have been shown to be particularly useful for understanding real structural behaviors in depth, revealing unknown factors that were not considered in design and providing structural information for necessary retrofit after extreme events. Detection, observation and clarification of rain-wind induced vibration of stay cables and wake galloping are presented as examples of the results of wind-induced vibration monitoring. Spike-like waveforms in seismic response and multiple ground excitations, which are not explicitly treated in the current design, are observed during seismic monitoring of long span bridges.

For seismic monitoring of long-span bridges, it is noted that in many cases number of sensors is insufficient to capture the response of bridges in an extreme event. This is because in the past, sensors were installed mainly to determine aerodynamic stability; hence they are insufficient for recording earthquake response of structures. In addition, we also need to measure other parameters that are critical and important in evaluating the seismic performance and safety of a bridge and not only accelerations and velocities.

Recently, as the number and age of long span bridge increase, more effort is placed to application of monitoring to maintenance and asset management. To ensure long service life and reduce a life cycle cost, a systematic maintenance that emphasizes on preventive maintenance is conducted for long span bridges. Detection of changes during rehabilitation is an earliest example, and continuous evaluation of performance indexes such as displacement is currently developed.

With the advance in the sensing technology, monitoring of environmental condition is also becoming more and more important for evaluation of bridge durability. Along this line, monitoring of long-span bridges should shift from the *fashionable (nice to have)* paradigm to *essential (useful to have)* paradigm, where the usefulness of monitoring system and its results for structural understanding becomes the key.

Fig. 25 Automated Ultrasonic Testing system for fatigue crack inspection on HSBE bridge girder. (Okuda *et al*. 2010)

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