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Control, simulation and monitoring of bridge vibration – Japan's recent development and practice

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ABSTRACT: This paper describes the recent research and experience on control, simulation and monitoring of bridge vibration in Japan. In the first part, vibration due to motion-dependent forces such as wind-induced aerodynamic forces and its control is discussed. New development and implementation of passive and control of stay cable of the cable-stayed bridges in Japan are described. In simulation part, development in bridge vibration under the high-speed train is explained. In the last part of the paper, importance and usefulness of vibration monitoring of long-span bridges under wind and seismic excitation are discussed with actual examples.

1 INTRODUCTION

In the past two decades, construction of long-span cable-supported bridges has been very active in the world. The world's longest suspension bridge and the world's longest cable-stayed bridge, namely Akashi Kaikyo suspension bridge in Japan (central span: 1991 m, Fig. 1) and Sutong cable-stayed bridge in China (central span: 1088 m) were completed at the end of 20th century and in the beginning of 21th century, respectively. In Asia, especially in China, Korea and Hong Kong, a number of long-span bridges were constructed in recent years and further construction of large numbers of cable-supported bridges are planned Super long–span projects are also seriously discussed in Japan, Italy and other parts of the world. In this century, long-span bridges that attract engineers and offer many exciting technical problems will continue to be constructed. Long-span bridges are flexible and low-damped and hence they are prone to vibrate under dynamic loading. Steel girders and towers, especially, are much lighter than those made of concrete and indeed many vibrations occur in real long-span bridges. Extensive research and development has been carried out to understand the vibration of such long-span bridges against natural sources of vibration such as wind and seismic, and to reduce the undesirable vibration through active and passive control countermeasures.

The other bridge vibration problem that has a special interest in Japan is the vehicular-induced vibration, especially the high-speed-train-induced vibration of viaducts. Among the primarily concerns are the effect of high-speed train vibration to the supporting structures such as viaducts and the reliability of the existing structures to support further increase in train speed. The latter aspect is very important considering the plan of Japan Railway Corporation to introduce new high-speed train. To understand the mechanism, extensive research and development has been carried out that includes simulation study on the train-bridge interactive dynamics that will be discussed in this paper.

Looking at a bigger picture of bridge vibration problems, we will arrive at the issues of safety, durability and long-term performance. These major issues have been long recognized by governments, bridge authorities, and engineers. In this regard, adding to the issue of bridge design, vibration control and simulation, is the bridge vibration monitoring. As the result, more efforts are now being devoted to enhance the implementation of technologies and methodologies for structural health monitoring (SHM) of bridges. Such research activities and applications will also be discussed in this paper.

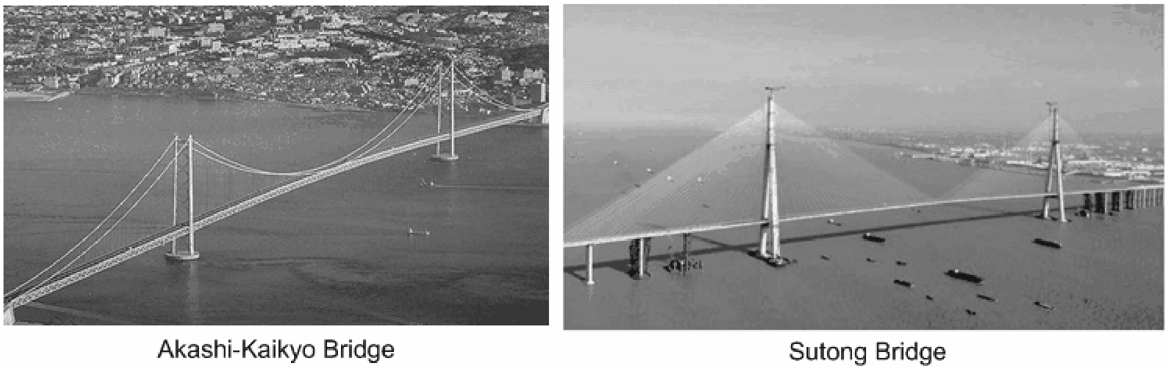
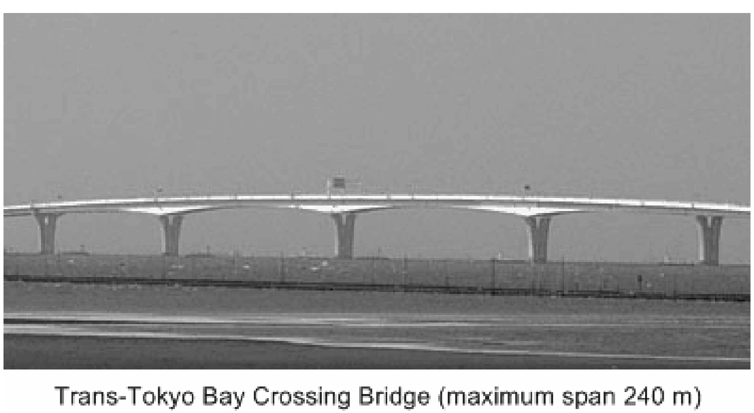


Figure 1. Akashi-Kaikyo Suspension Bridge and Sutong Cable-Stayed Bridge

2 WIND-INDUCED VIBRATION

Among the various types of dynamic excitation, wind-induced vibration may be the most critical for long-span bridges, such as suspension bridges and cable-stayed bridges. It should be noted that wind flow around the bridge (deck) is easily modified when the bridge moves and the bridge receives additional forces, namely motion-dependent forces, from the wind; these motion-dependent forces may lead to a *self-excited* vibration with very large amplitudes. A typical example can be found in the collapse of the Tacoma Narrows Bridge in 1940. Vortex-induced vibration is primarily a resonance of the structure due to the periodic forces generated by the Karman vortices behind the structure. However, it possesses a non-linear self-excited property characterized by *lock-in* phenomena and the amplitude is limited. Girders with box sections often exhibit large amplitude vortex-induced vibrations, whereas truss girders rarely show large vortex-induced vibrations. Flutter/galloping is a divergent-type of vibration, due to motion-dependent aerodynamic forces, leading to the collapse of the bridge. Fluctuating wind forces produce a gust response that is basically random and increases in proportion to the wind velocity.

Wind-induced vibration may occur in the completed bridges, but may be more serious in their erection stage. Free standing pylons and girders during erection before closing are extremely flexible and control of wind-induced vibration, especially the vortex-induced vibration that occurs under a moderate wind, becomes necessary on many occasions. Wind-induced vibration of bridges and their components, such as cables, is in general strongly non-linear and turbulence contained in the natural wind sometimes significantly changes the characteristics of wind-induced vibration. Due to the strong nonlinearities associated with the fluid, complete analytical models that can accurately describe various types of flow-induced vibration have not been obtained. Wind tunnel testing using full and sectional models is the only reliable method to predict the performance of structures in the wind. Analytical methods using the aerodynamic forces measured experimentally in wind tunnel testing are extensively used to predict the dynamic behavior of three-dimensional bridge structures. A computer fluid dynamics (CFD) technique has been developed significantly and used as a supplement in recent years. Rapid development of CFD is expected to become a strong tool; however it may take some time to become a complete replacement of wind tunnel testing because of the high complexity of the geometry of the bridge section and the three-dimensional nature of the bridge.



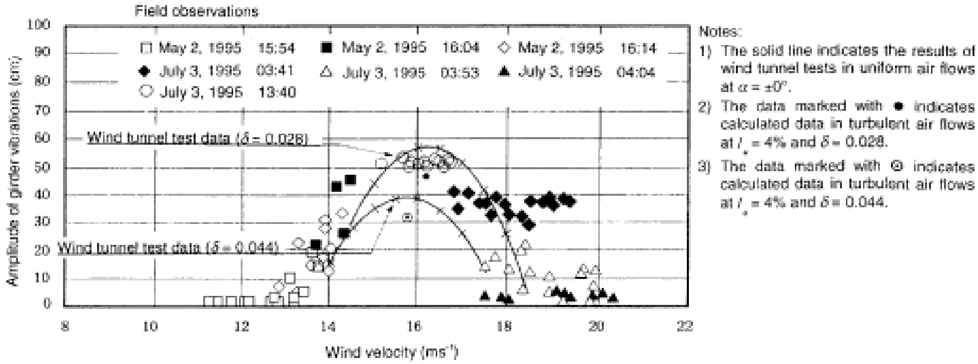


Figure 2. Trans-Tokyo Bay Crossing and wind-induced vertical vibration measured

Predictability of wind-induced vibration by wind tunnel testing has been one of vibration control remedies are examined in wind tunnel testing and implemented and unfortunately there are not many bridges that show noticeable vibration under wind. The Trans-Tokyo Bay Highway Crossing (Fig. 2), completed in 1997, is 11 km in total length and is a combined tunnel and multiple bridge route that include a ten-span continuous steel box-girder bridge with a total length of 1630 m. The two longest spans of this bridge measure 240 m, and the highway consists of four lanes with an overall width with 22.9 m. During the construction of the crossing, significant vibration due to vortex shedding was observed in the bridge under prevailing winds almost transverse to the bridge axis. The vortex-induced first-mode vibration peaked at a wind velocity around 16–17 m/s, with maximum amplitude exceeding 50 cm (Fig. 3) [Fujino & Yoshida (2002)]. Extensive wind tunnel testing using sectional models as well as three dimensional models had been made in the design stage of that bridge. The comparison between the wind-tunnel prediction and the observed response supports validity of wind tunnel testing.

In the bridge, TMDs inside the box girder and aerodynamic flaps were installed and the vibration was significantly reduced. It should be noted that the vortex-induced vibration was indeed observed in the wind tunnel experiments during the design stage. This bridge also provided access for construction of the man-made reclaimed island and the shield tunnel of the crossing; the highway was not put into service until long after completion of the bridge. Higher structural damping and larger turbulence intensity of the wind at the site were also expected. All these led to the decision not to install control devices a priori and, instead, to monitor the performance of the bridge under wind during the erection. The damping of the bridge and the turbulence intensity of the wind were found to be much lower than the values thought in the design stage and indeed the vortex-induced vibration took place.

3 CONTROL OF BRIDGE VIBRATION

In cases where intolerant wind-induced vibration is found after completion, some remedies are to be sought. The controls used can be classified into three methods: structural; aerodynamic; and mechanical remedies. The first one includes increasing the structural stiffness and mass. Control remedies for girders, pylons and cables are discussed, respectively

3.1 *Control of girders vibration*

Traditionally truss girders are widely used in suspension bridges in Japan. This is influenced by the collapse of the Tacoma Narrows Bridge in 1940. The truss girder can posses a high torsional stiffness and empirically it is true that truss girders have less aerodynamical problems compared with box girders. The Akashi-Kaikyo Bridge finally employed to truss girders, even though box girders were seriously considered (Fujino et.al. 1992). Because of its span length (almost 2000 m) and high design wind speed for flutter, 78m/s, the geometry of the truss girder needs special care to improve the aerodynamic stability.

Box girders with high depth have galloping as well as vortex-induced vibrations. Flat box girders used in cable-supported bridges have torsional (or classical) flutter and vortex-induced vibrations. In these girders, passive aerodynamic control has been successfully employed. The aerodynamic forces are very sensitive to the geometry and hence a small change of the section by adding flaps and fairings can stabilize the girder.

Often it is difficult to find aerodynamic control devices that are effective for vortex-induced vibrations as well as flutter/galloping. When satisfactory aerodynamic solutions cannot be found, control of wind-induced vibration relies on mechanical means such as TMD. An example is found in the Kansai Airport bridge which has parallely-placed box girders for roadway and railway; TMDs were installed from the beginning to suppress amplitude-limited vibration due to the wakes (Honda et.al. 1993). As mentioned, in the Tokyo Trans Bay Crossing 16 TMDs were installed for the vortex-induced vibration of the first and second modes (Fujino & Yoshida 2002). When the wind-induced vibration is found after bridge completion, aerodynamic control is practically difficult to implement and mechanical control by TMD is a natural choice.

3.2 *Control of pylons vibration*

In cable-supported bridges, concrete pylons are common internationally, while steel pylons have been used extensively in Japan primarily because of high seismic design load. Steel pylons are much lighter than concrete pylons by about 1/3 and consequently prone to vibrate under wind when the height of the pylons exceeds 100 m. Corner-cut is one of the well-known methods to reduce the vortex-induced vibration of pylons with rectangular sections (Yamada et.al 1991). This was used in many pylons including the Higashi Kobe Bridge and Akashi Kaikyo Bridge. It should be mentioned that the effectiveness of such aerodynamic control using small changes of the section depends upon the Re-number. In the wind tunnel experiments a high Re-number, say greater than 10,000, is recommended.

Wind-induced vibration during erection affects the safety and serviceability of the structure; typical examples are pylons in the free standing stage. Tall steel pylons require temporal control for wind-induced vibrations. Aerodynamic control, even if it exists, may not be suitable because it is needed for a short period, such as several months. In these cases, mechanical control is more suitable because it can be removed afterward. Sliding-block-type dampers were installed to control the vortex-induced vibration of pylons of the Forth Suspension Bridge during the erection. The dampers were connected to the top of the pylons by wires. This is probably the first application of mechanical control to large bridges. Similar applications can be found in long-span cable-supported bridges including Yokohama Bay Bridge, in which viscous dampers were employed. TMDs compact and can be installed on the pylons during the erection.

The first application of TMDs can be found in the Meiko Bridge. TLD, a passive damper that utilizes the liquid motion as an energy dissipater (Fujino et.al. 1992), is used in Higashi Kobe Bridge and Toda Bridge. TMD was also used to control the cantilevered girder of a cable-stayed bridge during erection. In tall pylons, not only the first fundamental mode but also higher modes need to be controlled. TMD and TLD are effective only for a small band of frequency. Active control that basically generates the damping force by actively moving a small mass can reduce the vibration of various modes of structures. The pylons of the Rainbow Suspension Bridge in Tokyo are the first application of the active control system. In the pylons of the Akashi Kaikyo Bridge, active control was used during erection, whereas to control the vibration after completion, TMDs were installed in the middle of the pylons (Koshimura et.al. 1994). Issues regarding cost, maintenance and reliability prefer passive TMD rather than active control systems for the permanent use in bridges.

3.3 *Control of cable vibration*

Stay cables are essential members in cable-stayed bridges and they can be almost as long as 500m in span, resulting in low natural frequencies such as 0.2 or 0.3 Hz in the lowest mode. Furthermore, their inherent damping is as low as 0.1% critical damping ratio (Fig. 3). It can be said that stay cables are the most vibrating structural members in bridges. Cables exhibit various types of wind-induced vibration. Rain-and wind-induced vibration of cables was first found during the erection of the Meiko Nishi cable-stayed bridge (Hikami 1990). Measurement at the site and wind tunnel experiments indicate the vibration can occur only when certain conditions are satisfied: a limited range of the wind direction and velocity, moderate rainfall and smooth approaching wind. Although the amplitude is an order of the diameter of the cable, it takes place not only in the first mode, but also in higher modes. This rain-and wind-induced vibration was found to occur in many cable-stayed bridges in the world and control of this vibration became one of the concerns in long-span cable-stayed bridges.

Studies suggest that inclined cables may have other types of self-excited vibration (Matsumoto 1998). In the case of twin parallel cables with a distance of 3–5 diameters, wake galloping may be induced; this is more violent vibration. Even if the distance between the cables is an order of ten diameters, violent vibration, so-called wake flutter, may occur. This vibration was observed in the suspender cables covered with PE in the Akashi Kaikyo Bridge (Furuya et.al. 1998).

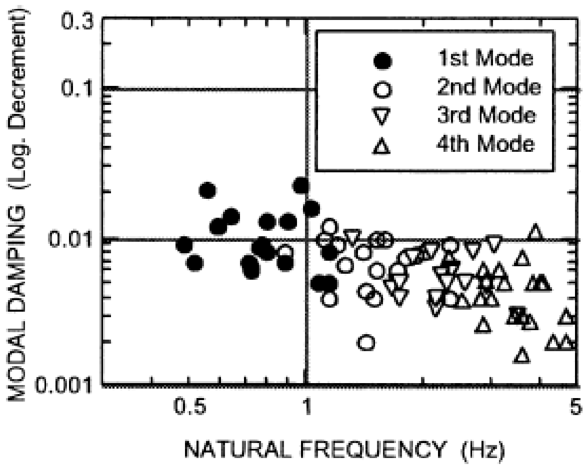


Figure 3. Relation between modal damping and natural frequency of stay cable

Control of stay cables is recognized to be a very common practice. Connecting cables by wires was used, for example in Faro Bridge (Denmark) and Meiko Nishi Bridge (Japan). However, since increasing stiffness by wires does not dampen the exciting force, some damage to the connection of PE stay cables and the wires was observed. At this moment, the most widely used control remedy is a passive damper installed near the cable anchorage. The damping force is generated by relative motion between the cable and the girder (Pacheco et.al. 1993). Aerodynamic control of cables, i.e. a roughness increase of the cable PE surface, was proposed for rain-and wind-induced vibration and implemented in some of the cable-stayed bridges, for example in Higashi Kobe, Tatara, Tenozan bridges. In these bridges, vortex-induced vibration was not suppressed (Yamaguchi & Fujino 1998).

Passive control by dampers has a limited performance and adequate damping cannot be provided for extremely long stay cables. The damper has customarily been designed by neglecting several influencing factors; one of which is the cable flexural rigidity (Pacheco et al. 1993). It can be anticipated that the small flexural rigidity that real cable systems possess affects the performance of the damper since it is usually installed near the anchorage. As a result, the additional damping provided to the cable may not be as high as designed.

Recent studies by author (Hoang & Fujino 2007, Fujino & Hoang 2008) on the theoretical aspects have provided deeper understanding on the effects of sag, bending stiffness of cable and flexibility of damper supports to the optimal performance passive control by dampers. Two types of damper were analyzed in the study namely the viscous damper and high-damping-rubber (HDR). The effects of sag, stiffness and flexibility on damping ratio ξ_n are quantified using compact formulae as follow:

$$\frac{\xi_n}{x_c/L} = R_k R_f R_{sn} \frac{\eta_k \eta_{sn} \eta_n}{1+(\eta_k \eta_{sn} \eta_n)^2} \qquad \text{for viscous damper,} \tag{1}$$

and

$$\frac{\xi_n}{x_c/L} = R_k R_f R_{sn} \frac{\varphi \eta_k \bar{K}}{(1+\eta_k \bar{K})^2 + (\varphi \eta_k \bar{K})^2} \qquad \text{for high-damping-rubber (HDR)} \tag{2}$$

where the R_k, R_f, and R_{sn} denotes the reduction factor in maximum damping ratio due to cable flexural rigidity, damper support stiffness and cable sag, respectively.

Furthermore, a recent study (Hoang & Fujino 2008) has investigated damping performance of a nonlinear damper attached to a stay cable. By utilizing an energy equivalent linear viscosity, it was found that for friction dampers, the obtained damper parameter is independent of the frequency, and is dependent on the damper motion. An optimal value of the friction force can be applied to multiple vibration modes; however, the maximum damping level in the cable varies according to the damper displacement. The performance of a nonlinear damper is both frequency and amplitude dependent. It was noted that a nonlinear damper has potential design advantages than does a linear damper over a wide range of modes.

4 HUMAN-INDUCED VIBRATION AND CONTROL

Similar to wind-induced vibration, human walking possesses adaptive and feedback nature, inducing motion-dependent human walking forces on bridges. In pedestrian bridges, it is well known that vertical periodic forces due to human walking excite the vertical motion of the girders. The frequency of human walking is about 2 Hz (two steps per second) and the design code in Japan specifies that the natural frequency of the vertical modes should be outside from 1.7 to 2.5 Hz to avoid the resonance vibration of the girder. There is a small fraction of horizontal periodic force generated by human walking, because human beings use two legs in walking (Fujino et.al. 1993) and its frequency is around 1 Hz. This force can potentially excite lateral motion of the girder if the natural frequency of the lateral modes is close to 1 Hz.

Indeed, such vibration was observed on the T-Park Pedestrian Bridge in Japan during the congested passage of pedestrians. Analysis of the motion of walking pedestrians reveals that human walking tends to synchronize the lateral motion of the girder; the frequency of the human walk is lock-in to the frequency of the lateral motion. (Figure 5) This excites the bridge in a resonant manner, resulting in a large lateral girder vibration with amplitude of several centimeters.

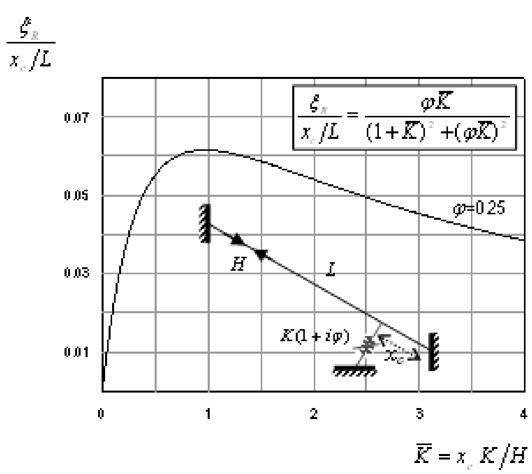


Figure 4 A general model of a cable with an HDR damper and a typical damping plot for an ideal taut non-flexural cable with damper having rigid support

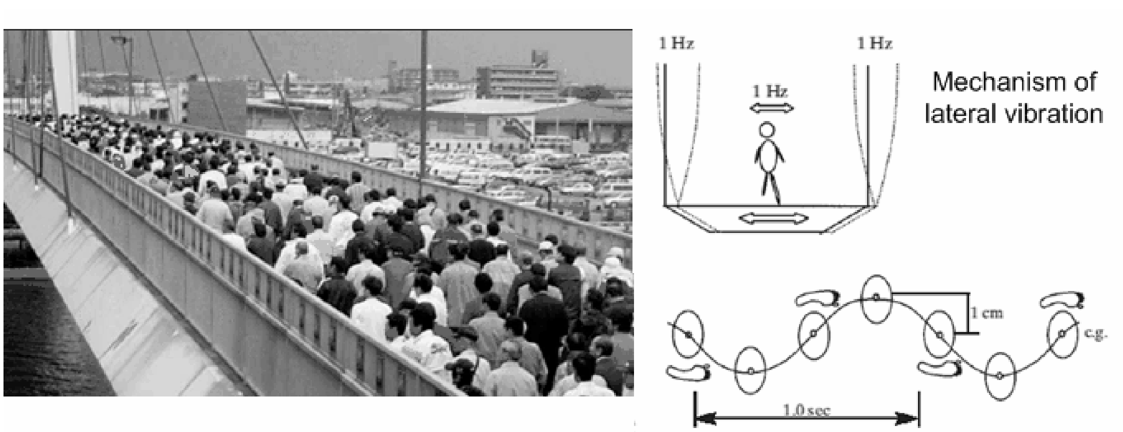


Figure 5. Pedestrian-induced lateral vibration at T-Park Bridge and the mechanism

To suppress excessive lateral vibration of the pedestrian bridge, two counter measures were used [Fujino et.al 1993, Nakamura & Fujino 2002]. Firstly, the secondary wires with a diameter of 10 mm were installed in the cable plane, and connected to the adjacent stays to restrict the stay movement and the interaction with the girder. The secondary wires were effective in reducing the vibration amplitude in the in-plane but not effective in the out-of-plane, and could not stop the girder movement. Secondly, the Tuned liquid dampers (TLDs) were then installed inside the box girder. The TLD is a sloshing type of damper and a plastic box is filled with water (Fujino et. al. 1992) developed a numerical method to predict the suppression effectiveness of TLDs using a dynamic structure and TLD interaction model by applying the non-linear shallow water wave theory to describe the water oscillation.

5 HIGH-SPEED-TRAIN-INDUCED VIBRATION

In 1964, the commercial operation of the first high-speed train in Japan marked the beginning of a new era for high-speed railways. Since then, the Japanese Railway has continuously increased the speed of its trains, from 210 km/h at the beginning to currently 270 km/h. At the same time, the number of train increases. The current number of train is five times greater than that of the initial stage. Such increase in speed and service frequency is expected to continue in the future. Because of the huge amount of kinetic energy carried at high speeds, a train might interact significantly with a bridge and even resonate with it under certain conditions, which should be considered in railway dynamics research. With further increases in train speed, the accumulated numbers of train passages were in many cases led to fatigue problems.

The high-speed-train-induced vibration poses multidimensional problems. Vertical vibration cause by train speed and railway irregularities raise the comfort problem for the passenger and the excessive vibration for the viaducts supporting the railway. The lateral vibration is of primarily concern with regard to safety against train derailment, and longitudinal or along the train movement vibration is often become the problem to the surrounding residential area since it creates the environmental noises. Among the primarily concerns, are the effect of high-speed train vibration to the supporting structures such as viaducts and the reliability of the existing structures to support further increase in train speed. The latter aspect is very important considering the plan of Japan Railway Corporation to introduce new high-speed train.

Although some dynamic phenomena have been explained qualitatively by in-situ dynamic measurement, the accurate numerical simulation is an indispensable investigation approach. It is expected to develop a quantitative evaluation method of the bridge dynamic characteristics under the train passage, which can not only inspect the present dynamic performance of existed bridges, but also predict the performance of them in extremely high speed case in the future.

The dynamic response of a railway bridge under the passage of a load results from several main factors, such as the moving load at a constant speed, the inertial forces due to the acceleration of the mass of the carriage and the bridge, the influence of the track irregularities and so on. Therefore, the dynamic model for a train-bridge interaction system is composed of a train model and a bridge model that are linked by an assumed wheel-track relation. The Train-Bridge Interaction (TBI) problem is complex because the contact points move in time.

In order to understand the mechanism and the extent effect of vibration, extensive research and development has been carried out using simulation study on the train-bridge interactive dynamics (Figure 6) (Su et.al 2007). A versatile method is utilized to analyze the train-bridge dynamics by treating the moving train and bridge as two separate systems, which interact with each other through the contact forces. Solving the contact forces from the train equations allows these to be treated as external forces on the bridge, which can then be solved using conventional finite element procedures. Furthermore, besides the moving gravity loading of the train, other external excitation factors such as rail irregularity and hunting motion of the train are also taken into account. All these procedures are realized using the general finite element software.

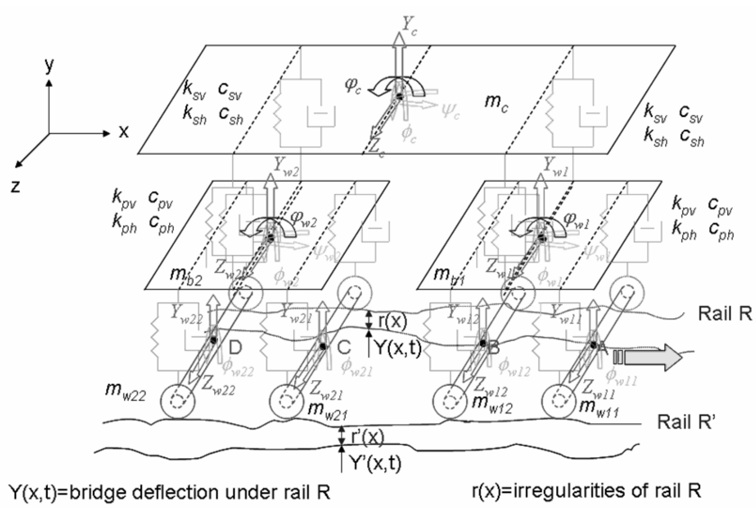


Figure 6. Model of high-speed train using 27 DOFs and interaction with railway

The simulation approach was used to analyze the damage observed on the webs of the main girder of a railway steel bridge. The damages were observed on the webs of a main girder at the bottom end of welded vertical stiffeners. Since train-induced vibration might cause and aggravate the damage, all parts having same structural details were retrofitted regardless the presence of damages. From the simulation results under high-speed train passage, it was found that the mode shapes, on which the bottom-end of a welded vertical stiffener and the center of a lower flange intersect and create a supporting point causes local stress. The local stress is in the pure bending. The retrofit, this connection point structurally altered and does not become a supporting point anymore. Hence the local bending-shear stress is greatly reduced. The cause of damages induced by local vibration was then confirmed by simulation (Su et.al. 2009).

The advantage of simulation approach is that it provides a benchmark for specific analysis to simulate various cases of three-dimensional train-bridge systems. It is not only an accurate simulation tool for train-induced vibration, but also instructive information for the retrofit of railway structures, especially at higher speeds.

6 VIBRATION-BASED STRUCTURAL HEALTH MONITORING OF BRIDGES

A large number of long-span bridges were constructed in Japan in the past decades. These bridges, especially the cable-supported ones, are expensive to inspect and maintain. They have long service periods of over 100 years, during which they inevitably suffer from environmental long-term loads effects such as fatigue, material deterioration and other extreme load effects. Governments, bridge authorities, as well as scientists and engineers, are now very much concerned with the issues of health, durability, and safety of these major bridges in a long-term service period. Structural health monitoring (SHM), therefore, becomes an increasingly important, and more concerted efforts are now being devoted to enhance our understanding in implementation of both monitoring technologies and methodologies.

Structural health monitoring using vibration data is recommended for global assessment of bridges. The system consists of bridge instrumentation, signal processing, data transmission and storage, and the most important part is data analysis and interpretation. Many large bridges such as long-span cable-supported bridges are now instrumented in Japan. Some of them are continually monitored using vibration response recorded from normal traffic, wind, typhoon and seismic responses. The objectives of vibration-based SHM can be broadly categorized in two: environment and disaster prevention, and stock management.

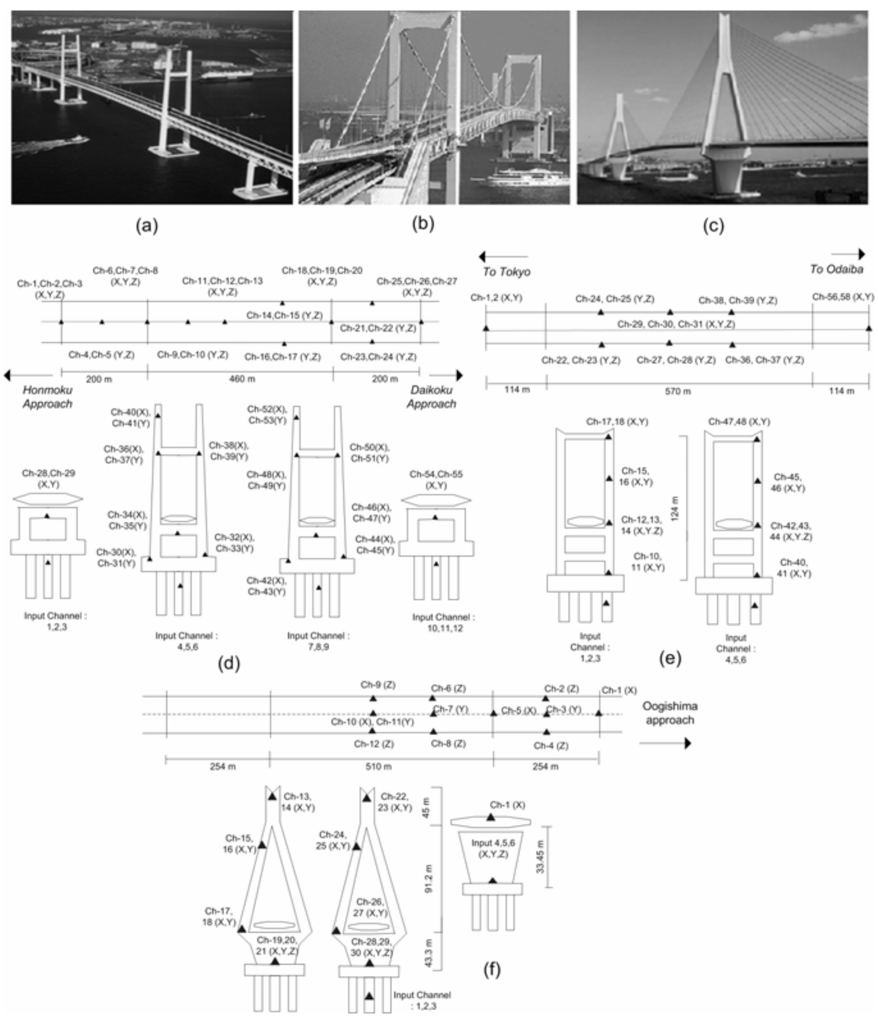


Figure 7 . Three permanently instrumented long-span cable-supported bridges in Japan (a&d) Yokohama-Bay Bridge, (b&e) Rainbow Bridge, (c&f) Tsurumi-Tsubasa Bridge

Earthquake

Monitoring for seismic response has been widely employed especially for structures with special features such as long span bridges (Siringoringo & Fujino 2008a, 2007, 2008b), and bridges with new technology such as base-isolated bridges (Chaudhary et.al 2000). One of such examples is the Yokohama-Bay Bridge (Fig. 7.a) (Siringoringo & Fujino 2006, 2008a). Dense seismic measurement system has been installed as shown Fig. 7.d. The study shows that by employing a system identification technique, performance of dynamic characteristics of the bridge can be evaluated. In addition to global behavior such as amplitude dependency of damping ratios, variations of local components were also observed. The identification results of the Yokohama Bay Bridge revealed two types of the first longitudinal mode, where the main difference was the relative modal displacement between the end-piers and girder (Fig. 8). In design, the end-piers and girder are connected by link-bearing connections (LBC) whose essential function is to prevent the large inertial force of superstructure from being imparted to substructures during large excitation. For this purpose, the LBC is expected to function as a longitudinal hinge connection to indicate that the girder and pier caps work as separate units in design.

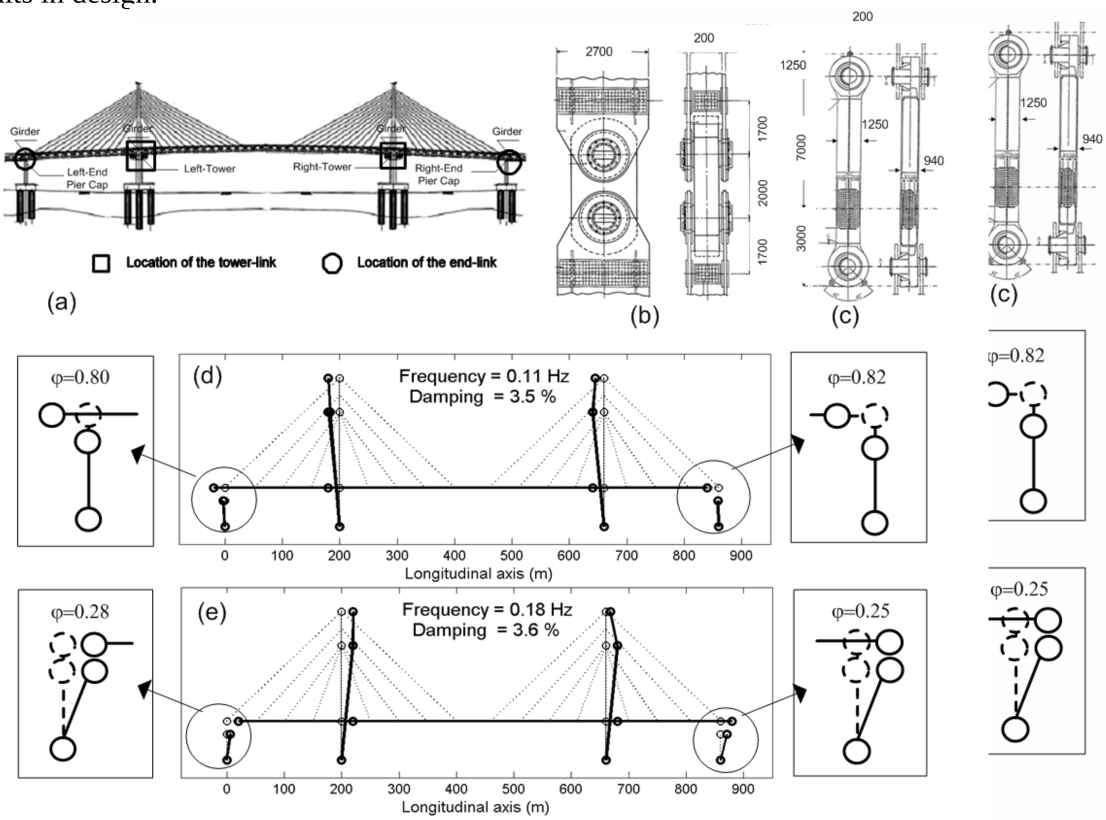


Figure 8. (a) Location of Link Bearing Connection of Yokohama Bay Bridge. (b) Typical LBC at the tower, (c) Typical LBC at end piers. Two typical first modes of Yokohama Bay Bridge identified from the main shock at 17:57 (d) Hinged-hinged mode: φ_{left}=0.80 φ_{right}=0.82, (e) Fixed-fixed mode: φ_{left}=0.28, φ_{right}=0.25.

The first type of longitudinal mode exhibited a large relative modal displacement as evidenced by $\varphi = 0.80$ and $\varphi = 0.82$ for the left and right end piers, respectively. The φ factor is defined to express the relative motion between pier cap and the girder. The shape characteristics and frequency of this mode were very close to that of the analytically obtained first longitudinal mode. The large relative modal displacement suggested that the hinge mechanism might have had occurred. The second type of longitudinal mode exhibited smaller relative modal displacements between end-piers and girder (i.e., $\varphi = 0.28$ and $\varphi = 0.25$ for the left and right end piers, respectively) indicating that the LBCs had yet to function as full-hinged connections. As a result stiffer connection and higher natural frequency were identified. These results indicate that performance of LBCs depend on the amplitude of earthquake excitation and does not always follow the analytical prediction.

The types of LBC mode significantly affect the amount of bending moment at the end-piers caused by the earthquake loading. When the pier and girder are rigidly connected, such as indicated by the fixed mode, larger portion of inertia force from superstructure would be imparted into the end-pier. And as a consequence, this will result in larger bending moment.

Although the study was focused on comparison with design and analysis, the result implies that damage or malfunction of the bearing could be detected by the monitoring system. In addition, this result is reflected to the on-going seismic retrofit of the bridge, connecting the girder end to the footing by PC cables.

Wind

Ambient vibration measurement has gained popularity due to its convenience in measuring vibration response while the structure is under service loading. This is viewed as an effective method to frequently assess structure's performance during its service life. Ambient vibration measurement uses loading from wind, waves, vehicular or pedestrian traffic or any combination of service loading. This offers economical advantage since it eliminates the need for loading testing devices and also does not require the suspension of structure services during measurement.

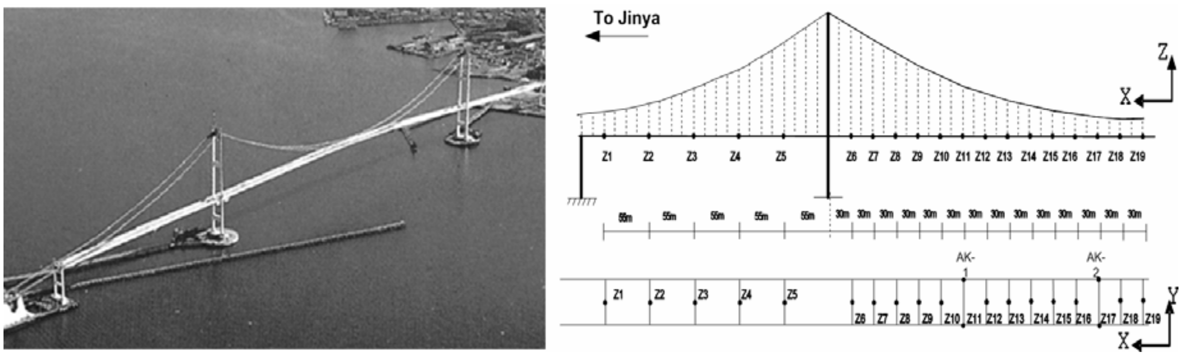


Figure 9 Hakucho Bridge and its sensor system

One of such analysis in conducted in Hakucho Suspension Bridge. During ambient tests the bridge response was monitored by densely distributed vibration measurement system. Forty accelerometers were installed on the girder, with the space of 30 m at the main span and at every 55 m on the side span. In order to measure wind velocity during ambient measurement an anemometer is installed at the center of the span of the deck.

To estimate dynamic characteristics of the bridge system identification using random response is implemented. In this approach, dynamic signatures that have the characteristics of free vibration responses were extracted from random response. Using these dynamic signatures, effect of randomness and transient can be eliminated to obtain the dynamic characteristics such as natural frequencies, damping ratios and mode shape vectors.

Owing to detailed measurement and dense sensor deployment, modal characteristics up to 28 modes can be identified from ambient vibration data. These characteristics exhibited dependencies on response amplitude level and wind velocity. In general the natural frequency decreases as the wind velocity increases and damping ratio increases as the wind velocity increases. The decrease and increase of natural frequency and damping ratio were found more apparent in the low order modes.

The similarity of the aerodynamic damping effect on vertical modes is observed in both wind tunnel experiment and the identification results. However, the results of aerodynamic stiffness from wind tunnel test are different from identification results. This difference is due to the effect of the rotational stiffness of the bearing, which was not included during wind tunnel test. The mode shape components have two different trends. The real parts of mode shape vectors do not exhibit distinct trend, indicating no obvious changes of modes. The modal phase angle computed from imaginary part of the mode shape vectors, however, reveal a clear trend.

The phase difference exists between the measuring point closest to the main tower and at the center of the main span is large when the acceleration *rms* is very small. When the acceleration *rms* becomes large, the difference decreases (Figure 10). This phase difference indicates the non-proportionally damped of the system. The primarily sources of this non-proportionality are the friction force at the bearings, expansion devices located at the main tower, and aerodynamic force. The phase difference, the decrease of modal frequency, and the increase in damping ratio imply friction force while the wind tunnel experiment indicates certain effect of the aerodynamic force on the modal frequency and damping. Effects of aerodynamic force and friction force have been verified by employing an inverse identification technique (Nagayama et.al 2005). The phase difference, the decrease of modal frequency, and the increase in damping ratio imply friction force while the wind tunnel experiment indicates certain effect of the aerodynamic force on the modal frequency and damping. Therefore, they are modeled as additional damping and stiffness at the edge of the girder and those distributed alongside it. One set of modal characteristics identified for 2-hour measurement is chosen as reference state, while other states are regarded as the systems that consist of the girder, the additional damping and stiffness. The additional damping and stiffness were identified by the inversion method.

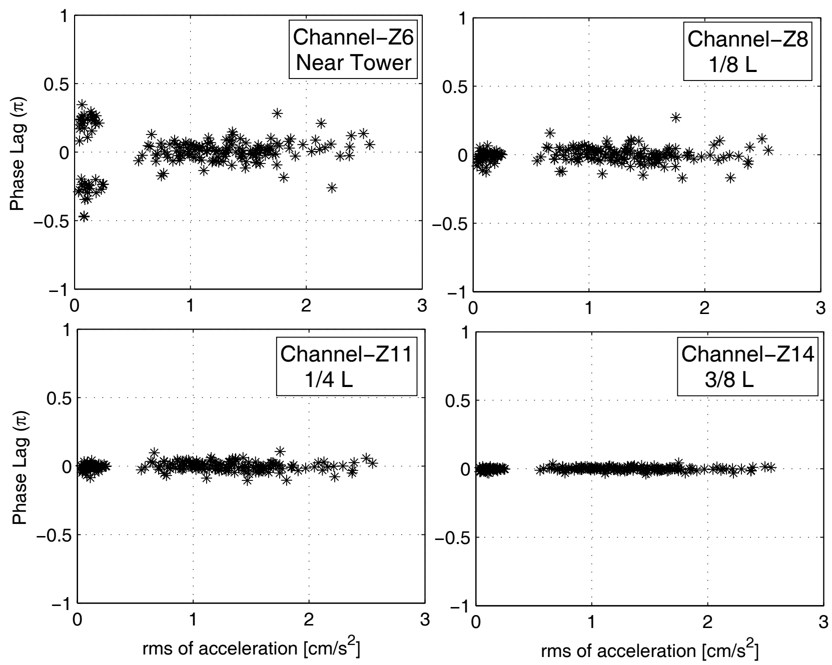


Figure 10 Phase Lag of the 1st mode with respect to rms of deck vertical acceleration

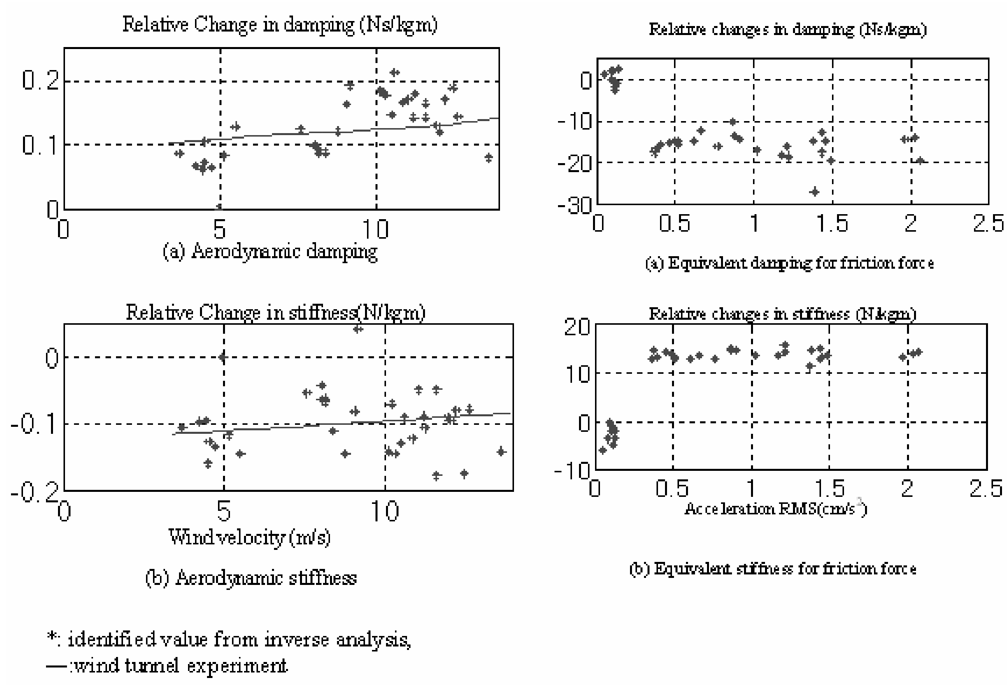


Figure11. Relative changes of stiffness and damping with respect to wind velocity resulted from inverse analysis and wind tunnel experiment.

It was found that the contribution of aerodynamic force was smaller than the effect of friction force from the bearing. The aerodynamic force contribution is in order of one-percent when compared to the contribution of the friction force (Fig.11). These results are consistent with the expected increase in equivalent stiffness and decrease in damping when the amplitude level is small enough to suppress any relative movement at the bearing or extension devices. This study shows that ambient monitoring data can be utilized to assess the force-structure mechanism with a proper application of the modal and structural inverse analysis.

6.2 *Structural monitoring for stock management*

Monitoring is expected to improve the conventional inspection procedures and to rationalize stock management. Here, several examples of this mode of monitoring that use vibration are briefly discussed (Fujino et.al 2009).

6.3 *Typical short-span bridges*

Monitoring of short to medium span bridges, which are the major portion of the stock is essential for efficient stock management. For this purpose, selecting the appropriate sensor, processing and interpreting the large number of data are of great challenges for engineers. The study in (Xia et.al. 2004) demonstrates how to apply some available algorithms to the data measured by a practical structural health monitoring system and how to evaluate the results for a simple ordinary medium span bridge. Identification of dynamic characteristics, temperature effects, and fatigue damage evaluation were studied using monitored data. Results indicate that the structural condition is deficient and that further special inspection is required, even though the bridge itself was relatively new (12 years old). The deficiency would be related to the skew effect, which may have not been appropriately treated in design. The demonstrated implementation and methodology can be considered as the general basis for monitoring of common workhorse structures.

6.4 *Non-contact measurement of Shinkansen viaducts*

The effects of speed on the structure and train were studied using a non-contact measurement system by means of laser Doppler vibrometer (LDV) (Siringoringo & Fujino 2006). This system was developed so that dynamic response of the viaduct can be measured three-dimensional wise with high sensitivity (Miyashita et.al 2007). The measurement system was applied to clarify the effect of train speed on the local stress at the vertical stiffener of the lower flange of steel viaduct's girder. It was confirmed that the vertical crack at the welded portion was due to the stress changes caused by the various train speeds, and therefore retrofit measures must take this factor in to account.

Another important aspect is the evaluation of conditions of viaducts columns. As a part of scheduled structural evaluations and after the occurrence of earthquake, supporting structures namely, columns and girders are inspected. The current technique employed by JR is based on frequency analysis of the column modes. In this system, the structures are excited by impact hammer and the responses at several locations on the columns are measured. Obviously this technique is time-consuming and requires massive manpower to meet the workload. In the study by (Hernandez et.al. 2006), an automatic non-contact inspection system using LDV is proposed. Despite the difficulty in separating the column's local modes from viaduct's global mode, the study shows that by carefully analyzing the column local mode, one can evaluate the column as well as the integrity of its substructures.

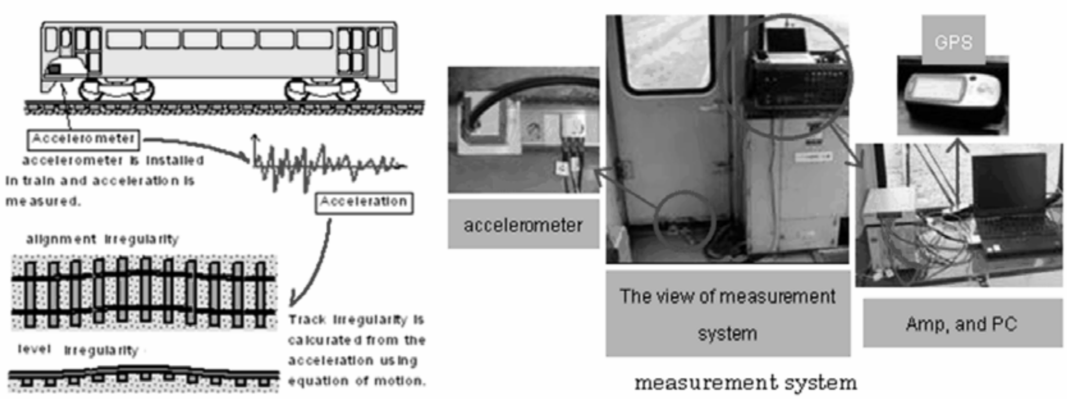


Figure 12. Train Intelligent Monitoring System

6.5 *Advanced routine inspection: TIMS & VIMS*

Having limited operational budget, local railways operators, cannot implement the sophisticated and costly maintenance strategies such as the one employed for the bullet train railways. Therefore, they require an accurate, simple, but low-cost maintenance system. To fill the need, the Train Intelligent Monitoring System (TIMS) was developed (Ishii et.al., 2006). The system consists of: 1) triaxal accelerometer(s) mounted on floor of train's car to measure train acceleration, 2) Global Positioning System (GPS) sensor to locate train position and, 3) a portable computer to record position and response data (Fig. 13). The measurement system is simple and able to detect irregularity on the surface of railway track. The facts that the system is portable and compact make it suitable for application on ordinary trains with relatively low cost. The developed system was applied to an actual railway and measurements with different time instances were compared. By looking at the root mean square (RMS) of the acceleration responses, it was found that the responses at the same location were relatively stable with repetitive measurements. It was also observed that repair and deterioration of track beds caused significant change in measurement. Further development of this is system is still under ongoing research.

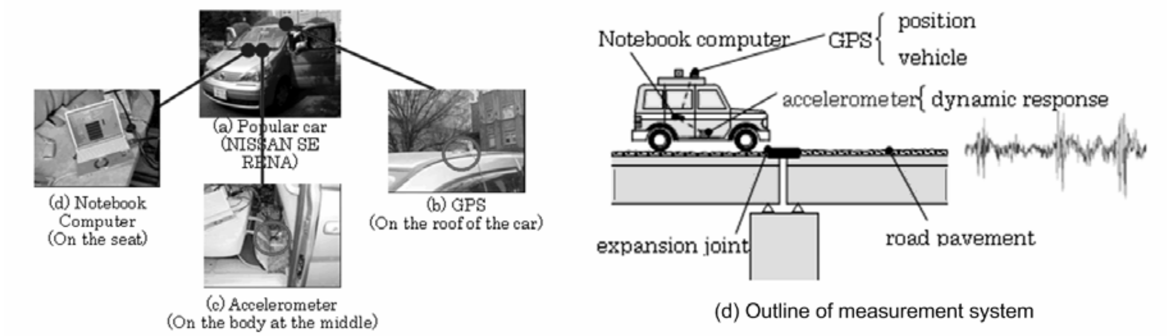


Figure 13. Vehicle Intelligent Monitoring System

A similar monitoring system was developed to monitor the highway pavement and expansion joints. The Vehicle Intelligent Monitoring System (VIMS) (Fujino 2005) utilizes dynamic response of a car to capture the condition of road pavements surface as well as the expansion joints. The system is proposed as an alternative to the conventional road profiler system, whose operational cost is relatively high. The VIMS system consists of: 1) accelerometer(s) to measure the dynamic response of the car, 2) GPS sensor to identify the position where the dynamic response is recorded, and 3) a portable computer to store the measurement data. Using the acceleration response and the pre-established correlation functions between the response and road surface roughness, one can identify the condition of expansion joint. Furthermore, by comparing the amplitude of dynamic response and the correlation function, one can continuously monitor the condition as well as the possible defects on expansion joints. This system was applied to the Tokyo Metropolitan Expressway, and its effectiveness to the road maintenance, i.e., frequent monitoring of the conditions of road pavement and expansion joints, was confirmed.

7 CONCLUDING REMARKS

Review on the recent experience in vibration problems, control, simulation and monitoring of bridges in Japan has been presented in this paper. Vibration problems of long-span bridges are presented along with countermeasures. Future plan to construct more and longer long-span bridge is expected to create new challenges in research as well as development of vibration control and countermeasures. Experience on vibration-based monitoring system has also been presented. Systematic study on the assessment of large bridges has shown that with a proper identification and detail analysis, monitoring data can be utilized as tools to understand the real mechanisms of structures. Using vibration-based monitoring it has shown that any deviation or unwanted mechanism that might hinder the structure from functioning can also be discovered. These are important features for future structural health monitoring. The lesson learned from each case study is expected to give a better understanding that might lead to a promising future application for bridge assessment and monitoring.

REFERENCE

Chaudhary MTA, Abe M, Fujino Y, Yoshida J, (2000), System Identification of Two Base-Isolated Bridges Using Seismic Records, *J. of Struct. Eng. ASCE* , 126 (10) : 1181-1194.
Fujino Y. and Yoshida Y.(2002). Wind-induced vibration and control of trans-tokyo bay crossing bridge", *J. of Struct. Eng. ASCE, ASCE*, 128: 1012-1025
Fujino Y, Iwamoto M, Ito M, Hikami Y. (1992) Wind tunnel experiments using 3D models and response prediction for a long-span suspension bridge. *J. of Wind Engineering and Industrial Aerodynamics* 42:1333–44..
Fujino Y. and Hoang N (2008).Design Formulas for Damping of a Stay Cable with a Damper, J. *Journal of Structural Engineering, ASCE*, 134, Issue 2: 269-278.
Fujino Y, Sun L, Pacheco BM, Chaiseri P.(1992), Tuned liquid damper (TLD) for suppressing horizontal motion of structures. *Journal of Engineering Mechanics, ASCE* 118(10):2017–30.
Fujino Y, Pacheco B, Nakamura S, Pennung W. (1993) Synchronization of human walking observed during lateral vibration of a congested pedestrian bridge. *Earthquake Engineering and Structural Dynamics* 1993;22:741–58.
Fujino, Y., (2005), "Development of Vehicle Intelligent Monitoring System (VIMS)", Proc of SPIE International Symposia on Smart Structures & Materials/NDE, Conference#5765
Fujino Y, Siringoringo DM, Abe M., (2009), The needs for advanced sensor technologies in risk assessment of civil infrastructures, Smart Structures and System
Furuya M, Miyazaki M. (1998), Wind induced vibration of parallel hangers in Akashi Kaikyo Bridge and its aerodynamic remedy. *In: Proc. of 2nd Cable Dynamics Seminar, Norway*.
Hernandez, J. Jr., Miyashita, T., Ishii, H., Vorabouth, P., Fujino, Y. (2006). "Identification of modal characteristics of shinkansen RC viaducts using laser doppler vibrometers". *Proc. of the First Asia-Pacific Workshop on SHM*, Yokohama, Japan.Hoang N. and Fujino Y. (2007) Analytical Study on Bending Effects in a Stay Cable with a Damper *J. Engrg. Mech.* Volume 133, Issue 11, pp. 1241-1246
Honda A, Shiraishi N, Matsumoto M, Fuse A, Sumi K, Sasaki S. (1993) Aerodynamic stability of Kansai international airport access bridge. *Journal of Wind Engineering and Industrial Aerodynamic*,49:533–42.
Hikami Y. (1990) Rain-and wind-induced vibration. Journal of Wind Engineering;40.
Ishii, Y., Fujino, Y., Mizuno, Y., Kaito, K., (2006), The Study of Train Intelligent Monitoring System using acceleration of ordinary trains, *Proc of Asia-Pasific Workshop on Structural Health Monitoring*, Yokohama Japan (CD)
Koshimura K, Tatsumi M, Hata K. (1994) Vibration control of the main towers of the Akashi Kaikyo Bridge. In: Proc. of 1st Int. Conf. on Structural Control, TP3, 2.:98–106.
Matsumoto M.(1998) Observed behavior of prototype cable vibration and its generation mechanism. In:Larsen A, Esdahl S, editors. Bridge aerodynamics, Rotterdam: Balkema:. 189–211.
Miyashita, T., Ishii, H., Fujino, Y., Shoji, T., and Seki, M. (2007). "Understanding of high-speed-train induced local vibration of a railway steel bridge using laser measurement and its effect by train speed." *JSCE Journal of Structural and Earthquake Engineering*, 63(2), 277– 296 (in Japanese).
Nagayama T, Abe M, Fujino Y, Ikeda K, (2005),Structural Identification of a Nonproportionally Damped System and Its Application to a Full-Scale Suspension Bridge, *Journal of Structural Engineering*, Vol. 131, No. 10: 1536-1545.
Nakamura S.I, and Fujino Y, (2002), Lateral vibration on a pedestrian cable-stayed bridge *Structural Engineering International: Journal of the International Association for Bridge and Structural Engineering (IABSE)* 12(4): 295-300.
Pacheco B, Fujino Y, Suelk A.(1993) Estimation curve for modal damping in stay cables with viscous damper. *Journal of Structural Engineering, ASCE* :119(6):1961–79.
Siringoringo D.M and Fujino Y, (2008), System identification applied to instrumented long-span cable-supported bridges using seismic records, *Earthquake Engineering & Structural Dynamics*, Vol.37: 361-386,
Siringoringo D.M and Fujino Y, (2008), System identification of suspension bridge form ambient response measurement, *Engineering Structures*, 30 No..2 :462-477
Siringoringo D.M and Fujino Y (2007), Dynamic characteristics of a curved cable-stayed bridge identified from seismic records, *Engineering Structures*, Vol. 29, No.8: 2001-2017
Siringoringo D.M and Fujino Y (2006), Experimental study of Laser Doppler Vibrometer and ambient vibration for vibration-based damage detection, *Engineering Structures*, Vol. 28, No.13, :1803-1915
Siringoringo D.M and Fujino Y., (2006), Observed dynamic performance of the Yokohama-Bay Bridge from system identification using seismic records, *Journal of Structural Control and Health Monitoring*, Vol. 13(1), pp 226-244
Su, D., Hernandez, J., Fujino, Y., Ollinger, E., and Miyashita, T. (2007). "Local dynamic vibration prediction of interaction between high speed train and railway bridge." *Proceedings of Experimental Vibration Analysis for Civil Engineering Structures*, Porto, Portugal. 1261–1270.
Xia, Y., Fujino, Y., Abe, M., and Murakoshi, J., (2005) "Short-term and long-term health monitoring experience of a short highway bridge: case study, *J .of Bridge Structures*, 1(1); 43-53
Yamada Y et al. (1991) Earthquake-resistant and wind-resistant design of the HigashiKobe Bridge, In:Yamada Y, editor. Cable-stayed bridges—their recent developments and their future. Elsevier:397–416.
Yamaguchi H, Fujino Y.(1998) Stayed cable dynamics and its vibration control. In: Larsen A, Esdahl S, editors. Bridge aerodynamics, Rotterdam: Balkema:. 235–53.