Monitoring of bridges and transportation infrastructures

Yozo Fujino
Department of Civil Engineering, University of Tokyo, Tokyo, 113-8656, Japan
fujino@civil.t.u-tokyo.ac.jp

ABSTRACT: This paper firstly presents an overview of civil infrastructures and their problems in Japan, and discusses characteristic features of structural health monitoring for civil infrastructures. The focus of the discussion is placed on bridges in the transportation system. In the second place, research and development of health monitoring systems at the Bridge and Structure Laboratory of the University of Tokyo is presented. The topics described are 1)Monitoring of train-induced vibration in a steel railway bridge to clarify the cause of a fatigue cracks, 2)Measurement of 3-D motion by Laser Doppler Vibrometers, 3) Monitoring of rail truck condition from measurement of dynamic motion of trains, 4) Monitoring of pavement and expansion joints of urban elevated highway and 5) performance evaluation of a monitored large cable-stayed bridge using seismic records.

1 INTRODUCTION

Civil infrastructures include various units and systems such as buildings, transportation systems, communication systems, power generation and distribution systems, and water supply/drainage systems. Significant investment expenditures and long time periods are necessary for a society to build up its infrastructure systems; indeed, any advanced society has already put enormous effort toward this goal. The continued performance of these infrastructure units and systems is vital to our human and economic activities. Even a short-term malfunction affects the society tremendously.

The built environment supported by infrastructures is an asset if they are well maintained and properly operated. However, if they are degraded by improper maintenance, they can be, sooner or later, a big debt to the society. To prevent this, it is absolutely necessary to develop maintenance technology that involves inspection, monitoring, repair, rebuilding and even financial planning. Structural health monitoring is one of the important components in the maintenance technology for civil infrastructures.

This paper primarily presents an overview of civil infrastructures and their problems in Japan, and discusses characteristic features of structural health monitoring for civil infrastructures. The focus is placed on bridges in the transportation system. Recent research related to structural health monitoring conducted at Bridge & Structure Laboratory, Department of Civil Engineering, University of Tokyo (to which the authors belong) is described with examples.

2 INFRASTRUCTURE DEVELOPMENTS IN JAPAN

In Japan, construction of modern infrastructures started in Meiji Era (~1868). Investment in railway development was considered a high priority in those days. The present railway network was practically established in 1940's. Construction of roads was rather slow before the 1940's. Rapid growth of the Japanese economy started in the 1960's and intensive investment in road construction followed to meet expansion of industrial activities.

The first bullet train railway line and the first highway were started in the middle of 1960's. Construction of the bullet railway network has not yet been finished, but the total length of the railway system is saturated and even decreasing somewhat; consequently, new construction of railway bridges is rare. Many railway infrastructures are rather old and need elaborate maintenance.

Development of the highway network system is still actively on-going at present. However, the amount of new road bridge construction has also been decreasing in recent years. Road bridges constructed in the 1970's will be more than 50 years old in 20 years. Further, the number of the aged bridges is will constitute half of all road bridges by

the year 2020. A similar situation can be also observed in buildings in Japan.

Although Japan is considered to be active in new construction of infrastructures, maintenance is becoming increasingly important.

3 PROBLEMS OF EXISTING STRUCTURES

Infrastructures constructed in the rapid growth period (1960 to 1980) are enormous, but they are not considered to be of the highest quality because of the construction boom during that time. In concrete structures, for example, sand taken from the sea was often used without adequate washing, so chemical degradation is rapidly progressing in such concrete structures. It is reported that tendons in many prestressed concrete bridges are rusted because water leaked into the tendon tubes due to incomplete grouting.

In 2000, a number of un-reinforced concrete blocks fell from tunnel linings for a bullet train railway that were constructed around 1970. Fortunately, the concrete blocks did not hit any train cars. After these incidents, the concrete linings in old railway tunnels, as well as road tunnels, were extensively investigated by the "hitting-by-hammer" method. Thousands of potentially dangerous cracks were indeed found in the lining of many tunnels.

In an elevated viaduct of the Tokyo Metropolitan Expressway, a relatively new traffic sign pole fell down and hit a motorcar beneath the bridge. The cause of this accident was improper welding.

These accidents clearly highlight the importance of inspection and maintenance of civil infrastructures. Inspection rules for railway and roads were revised after these accidents; more frequent and detailed inspection was specified. The public had tended to think that civil structures are permanent maintenance-free fixtures, but has now realized that this is not the case.

Regarding the road bridges, the traffic volume in major highways is as large as 15,000 vehicles per lane and the ratio of heavy trucks in the traffic is also high (as high as 30%). On some of the bullet train routes, trains run quite frequently. The high intensity and high frequency loading generated by this high traffic volume creates many problems in bridges: the most typical problem is fatigue damage in concrete slabs and steel welded girders.

The cost of maintenance of the highway in Japan as a function of the service period shows that the maintenance and repair cost increases sharply when the service period exceeds 15 to 20 years. The average age of bridges in Japan will soon become 40 or even 50 years soon, so the necessity of frequent and detailed inspection and maintenance/repair is also expected to increase significantly in the near future. However, because of the tight budget in Japan, expenditures for maintenance-related activities are not expected to increase in proportion to the need. Consequently, more efficient and economical inspection and maintenance strategies are required.

4 MONITORING AND INSPECTION OF CIVIL INFRASTRUCTURES

So far, inspection of civil infrastructures is basically made by human beings. It is extremely labor-intensive. Inspectors are often exposed to dangerous conditions and/or uncomfortable working environments. Considering that the volume of inspection work will be significantly increased in the near future, inspection by human beings must be replaced with automated instruments and inspection efficiency and accuracy must be improved.

Inspection of civil infrastructures is considered by some to be an old fashioned, low-tech and unpleasant subject. Recent development of sensors and information technology, however, may completely modernize the content of the work and may increase the efficiency of the work dramatically. For example, the human-based "hit-by-hammer" method to find cracks in the dust-covered lining of tunnels has been considered the only reliable method. Recently, the ultra-red-ray method and digital video method were being developed; these methods can reduce the labor and the time considerably.

Many civil infrastructures are spatially distributed and the area and points to be inspected or monitored is huge. Certainly, sensors that measure spatial information rather than a point are to be developed. Accuracy of the sensors is important, but often simplicity, durability and robustness are more important than accuracy since they are exposed to severe and hostile environments.

To detect damage from the sensor signal is also another important subject. The signals from the sensors in civil infrastructures are in many cases contaminated by ambient noise due to traffic and other environmental effects. To efficiently remove noise from ambient signals and to obtain useful information requires not only data processing techniques, but also knowledge of traditional disciplines of civil engineering such as mechanics, dynamics, soil mechanics, material technology and so forth(For example, Nagayama *et al.*2005)

Structural Health Monitoring (SHM) for civil infrastructures is an extremely important research area to keep our advanced society functional and requires a balanced combination of new sensor/computer technology and civil engineering technology(Wu *et al.* 2003). In this sense, SHM attracts young promising engineers and researchers.

Due to increase in interest and importance of continuous structural health monitoring of infra-

structures, extensive monitoring has been implemented in some of the bridges in Japan(For example, Xia *et al.* 2005, Kobayashi *et al.* 2004 and Yamada *et al.* 2004)

The Bridge and Structure Laboratory at the University of Tokyo has made recent attempts to contribute to the development and establishment of vibration-based SHM(Fujino 2002). Some research activities at the Laboratory related to SHM are given hereafter.

5 MONITORING OF HIGH-SPEED TRAIN INDUCED VIBRATIONS OF A RAILWAY STEEL BOX GIRDER BRIDGE

At a railway steel box girder bridge, damage was observed on the web of a main girder at the bottom end of a welded vertical stiffener. The cause of the damage is thought to be an increase in the local stress due to vibrations induced by high-speed trains. Thus, parts of the bridge of similar detail were also strengthened and this proved to be effective in preventing further damage. However, the relationship between train-induced vibration and the increase in local stresses remains to be unclear. Similarly, the relationship between train speed and the induced vibrations need to be investigated, including the effects of acceleration on a railway bridge since there are plans of increasing the present speed of trains in service. In this research, the effect of train-induced vibrations and the train speed on local stresses at each part of the bridge were studied by analysis as well as field measurement.

The bridge studied for field measurement is a pair of steel mono-box girders with 4 spans(Miyashita *et al.* 2005). There are cross frames every 3.2m, and vertical stiffeners were installed midway between cross frames as shown in Figure 1. The stiffeners were not welded to the lower flange in the sections where positive moment affects the main girder in order to protect against fatigue damage of the lower flange. In bridges which have similar detail, damage was observed on the web of a main girder at the bottom end of a welded vertical stiffener. The parts of the bridge of similar detail were strengthened such as in Figure 1 using a T-shape member installed between the web and the lower flange using high tension bolts. The trains which pass on the bridge have 16 cars which are 25m long, and having maximum speed of 270km/h at present. Accelerometers and strain gauges were installed in the bridge. Since it is suspected that the cause of high local stresses was local vibration of the web or lower flange of the main girder, accelerometers were installed on both the web and the lower flange of the main girder. The strain gauges were installed on the main girder web 20mm away from the toe of welding between the web and the vertical stiffener.

Figure 2 shows examples of acceleration, stress and FFT results from the field data. Figure 2(a) shows that there is one large waveform caused by trains passing on the bridge and there are 17 cycles corresponding to each bogie passing the cross section. It is shown that the dominant frequency is 3.0 Hz, which depends on the 17 cycle waveform. This phenomenon is known as “speed effect by regular loading” which depends on the interval that bogies or train cars pass at constant speed. Figures 2(b), (c) and (d) show the presence of high frequency local vibration and stresses, and the frequencies of these vibrations are 20Hz～30Hz. Each result shows a dominant frequency and they have the same intervals.

Figure 3 shows the relation between the dominant frequency of acceleration of the main girder and train speed of 39 trains. This figure shows that the dominant frequency of vibration of the main girder depends on the train speed and it agrees with the relational expression “speed effect by regular loading” (fb=v/90). Figure 4 shows the relation between the dominant frequency of acceleration of the lower flange and train speed. This figure shows that the dominant frequency depends on train speed, similar to the vibration of the main girder and that the frequency is an integer multiple of the frequency of the main girder. Figure 5 shows the relation between local vibration and local stresses. This figure shows that there is a strong correlation between local stresses and the lower flange vibration, and that local stresses and web vibration correlate poorly. Thus, it was confirmed that the cause of high local stresses is local vibration of the lower flange.

Figure 6 shows the relation between the FFT amplitude and the frequency of the lower flange vibration. This figure shows that there are 2 peaks of different frequencies. A peak occurs at 28.7Hz and another one is at 32.8Hz. These 2 peaks are identified the natural frequencies of local vibration. Figure 7 shows the relation between the FFT amplitude of the lower flange vibration and the train speed in each mode. It is shown that each mode has each peak at a certain speed and that the frequency at each peak is an integer multiple of the frequency of the main girder. Thus, it was confirmed that the vibration is larger when the natural frequency of local vibration is an integer multiple of the frequency of the main girder induced by regular loading.

6 DEVELOPMENT OF THREE-DIMENSIONAL VIBRATION MEASUREMENT SYSTEM

Damage to civil infrastructure has occurred because of aging or deterioration, resulting in a need to

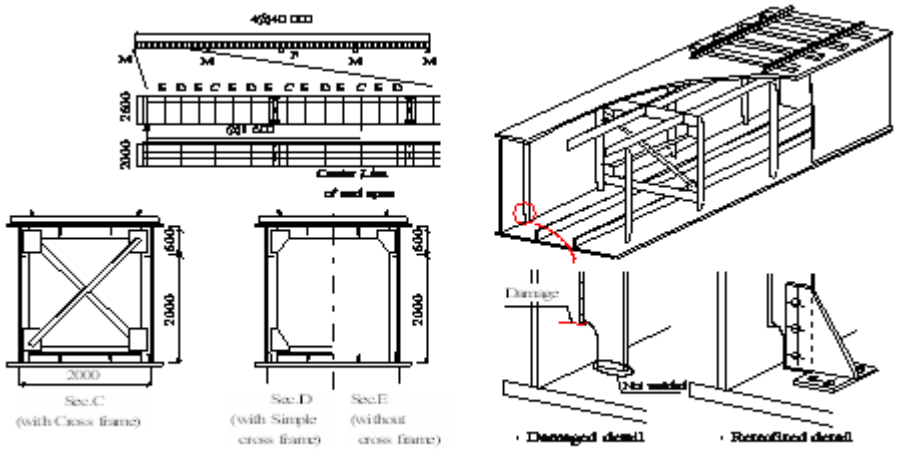
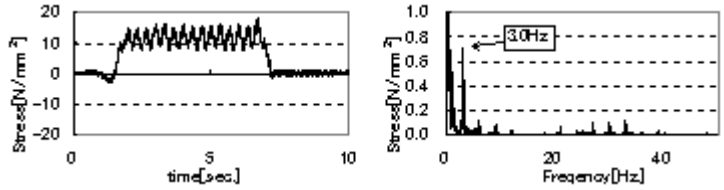
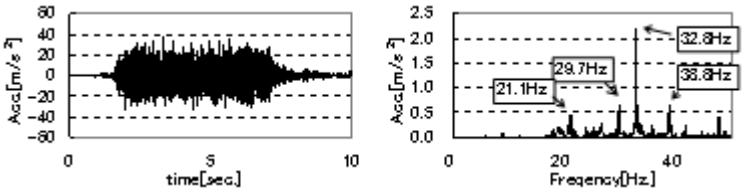


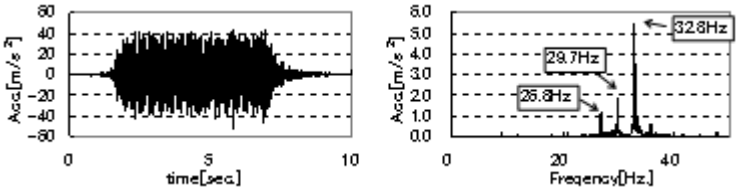
Figure 1. The outline of the field measurement



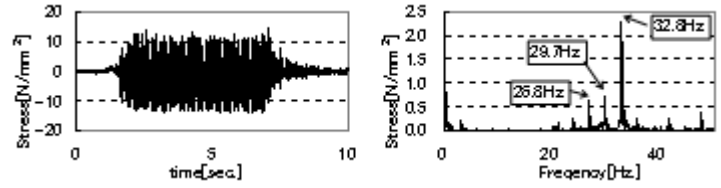
(a) Nominal stress of lower flange (S01R).



(b) Acceleration of web (A42L).



(c) Acceleration of lower flange (A42C).



(d) Local stress of web (S42LI).

Figure 2. The result of field measurement.

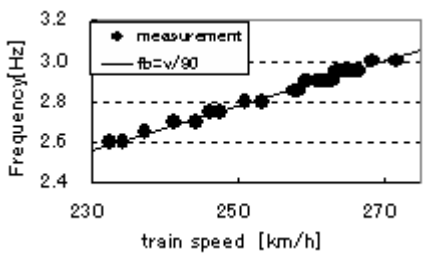


Figure 3. Relation between train velocity and frequency of main girder.

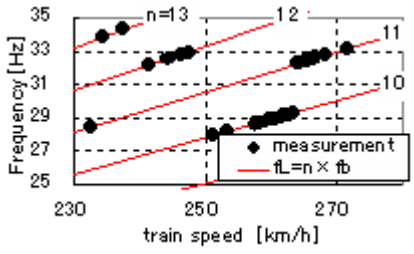


Figure 4. Relation between train velocity and frequencies of lower flange.

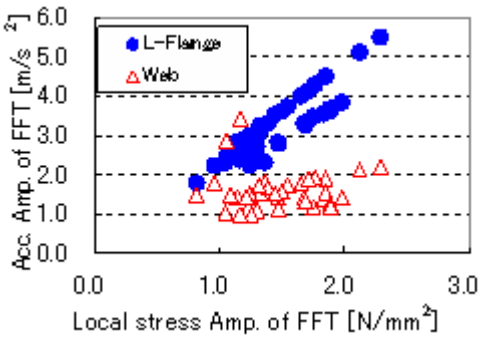


Figure 5. Relation between local vibration and local stress.

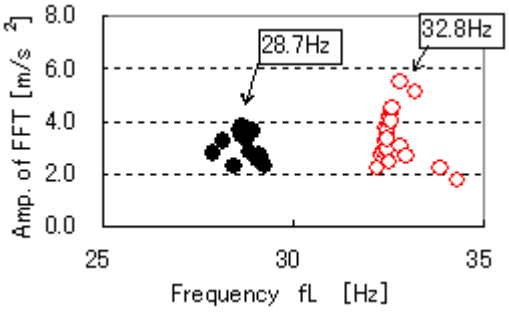


Figure 6. Relation between FFT amplitude and frequencies of lower flange vibration.

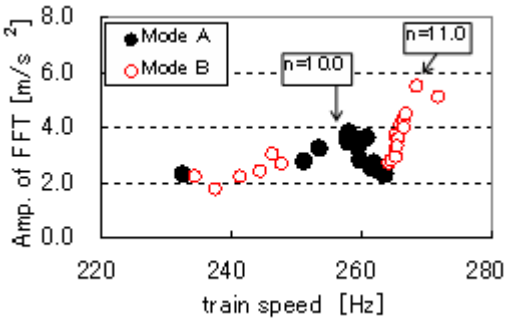


Figure 7. Relation between FFT amplitude of lower flange and train speed.

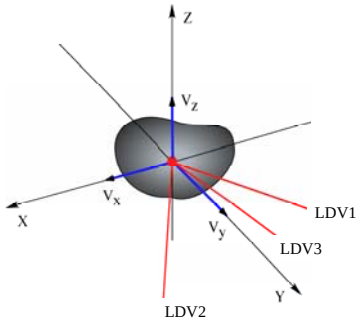


Figure 8. Three-Dimensional Measurement using LDVs

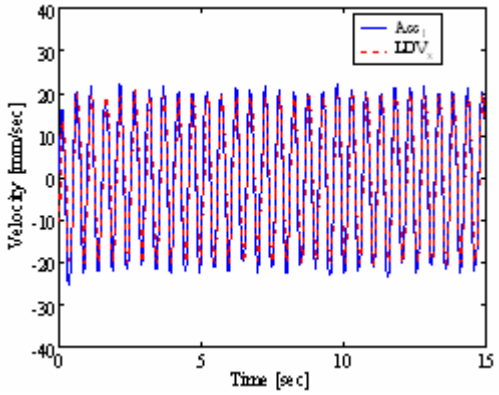


Figure 9. Comparison(LDVs and accelerometer)

develop effective and qualitative Structural Health Monitoring (SHM) techniques. The Laser Doppler Vibrometer (LDV) is an effective device for SHM applications. Kaito *et. al*(2000, 2001, 2005) and Fujino(2005) developed a non-contact scanning vibration measurement system using LDVs. The LDV system was applied to measure the vibration of steel and reinforced concrete girders of existing viaducts. The modal shapes were identified and the quality of measurement was quantitatively evaluated. In previous studies[1)] using the LDV system, only one-dimensional measurements were conducted focused on the maximum vibration component in three-dimensional vibration. When accurate measurements are needed, problems in a one-dimensional measurement system should be overcome. First, compensation of laser beam angles is required. Second, it is impossible to distinguish in-plane vibration components and out-of-plane vibration components.

In three-dimensional vibration measurement using LDVs, the measurements are conducted in three directions by operating three LDVs shown in Fig. 8(Miyashita *et al.* 2005). Three-dimensional vibration components of a body are measured by transforming the measured vibration components to the coordinates of the body. Since vibration components in each direction are determined based on laser beam angles, the system is capable of overcoming the problems in one-dimensional vibration measurement. For three-dimensional vibration measurement using LDVs, we have studied the system combining three single point type LDVs. In this research, for the three-dimensional vibration measurement of multiple points, a system using three LDVs with scanning unit is studied. This system makes possible to measure three-dimensional vibration components of multiple points in a vibrating body. First, to verify the measurement principle, a fundamental study is conducted. Then,

is conducted. Then, an application example on field measurement is shown.

6.1 *Measurement system*

LDV is an optical instrument employing laser technology to measure velocity of points on a vibrating object. Velocity is measured based on the Doppler effect, which induces a frequency change between the radiation laser beam and the reflected laser beam. The characteristics of the LDV are: first, compared with conventional transducers such as accelerometers, non-contact and long distance measurement (up to 100 m) is possible without adding mass or stiffness to an object. Second, resolution of velocity is very high (0.1μm/s) and bandwidth is very wide (0 - 1.5M Hz). Therefore, in situations when the installation of the measurement devices is difficult, it is possible to measure high frequency components of small vibration such as ambient vibration. Third, by attaching a scanning mirror unit in front of the laser sensor head, measurement on multiple points is made possible. In this research, measurement system was developed by operating three similar LDVs with scanning unit.

6.2 *Measurement principle*

The principle of three-dimensional measurement involves a geometrical transformation of coordinates shown in Fig.1. Three-dimensional vibration components of the body are determined from measurements of the three LDVs by premultiplying a transformation matrix consisting of the direction cosines of the laser beam angles.

$$\begin{Bmatrix} V_X \\ V_Y \\ V_Z \end{Bmatrix} = \begin{bmatrix} \cos\alpha_1 & \cos\beta_1 & \cos\gamma_1 \\ \cos\alpha_2 & \cos\beta_2 & \cos\gamma_2 \\ \cos\alpha_3 & \cos\beta_3 & \cos\gamma_3 \end{bmatrix}^{-1} \begin{Bmatrix} V_1 \\ V_2 \\ V_3 \end{Bmatrix} \tag{1}$$

where V_X, V_Y, V_Z are vibration components of the body in each axes, V_1, V_2, V_3 are vibration components measured by each LDV and $\alpha_i, \beta_i, \gamma_i$ $(i = 1,2,3)$ are the laser beam angles of each LDV.

Although, in the system using single point type LDVs, calibration steps are needed to know laser beam angles[2)], in the system using LDVs with scanning unit, the angles are determined directly based on input voltages to two axial Galvanomirrors.

6.3 *Laboratory testing*

First, to verify the measurement principle represented by Eq. (1), a fundamental study was conducted. An accelerometer was put on a uni-axial shaking table and vibrated by applying a sine wave voltage in each direction V_X, V_Y, V_Z shown in Fig.8. Three-dimensional vibration components were measured on the surface of the accelerometer. Fig. 9 shows a comparison of measurements by velocity. Acceleration measured by the accelerometer was integrated numerically. Vibration component is V_X direction component shown in Fig. 8. Solid line is measurement by the accelerometer and dotted line is measurement by LDVs calculated using Eq. (1). Since, with regard to the other components V_Y, V_Z, results were in good agreement, the measurement principle was verified.

6.4 *Field measurement*

The developed system was applied to field measurement. At a steel box girder railway bridge, a triaxial accelerometer was put on the surface of the web and measured the surface vibration three-dimensionally using LDVs. Fig. 10 shows the configuration of LDVs and the measurement axes of the accelerometer. Figs. 11 show comparisons of measurements in each axis by acceleration. Velocities measured by LDVs were differentiated numerically. Vibration components shown in Fig. 11were free vibration components of the steel girder bridge after a train passed. Solid lines are measurement by accelerometer and dotted lines are measurement by LDVs calculated by Eq.(1). One can find good agreement.

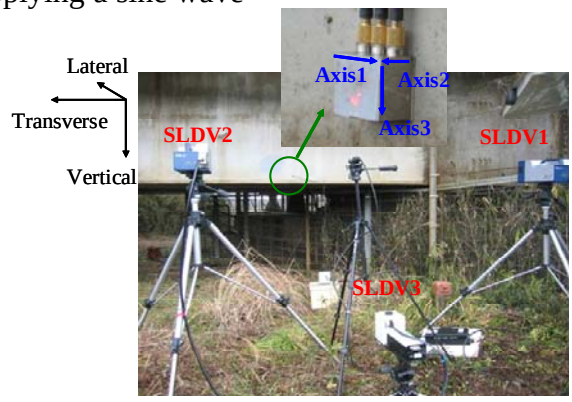


Fig.10 Configuration of LDVs and Measurement axes of accelerometer measurement axes of accelerometer

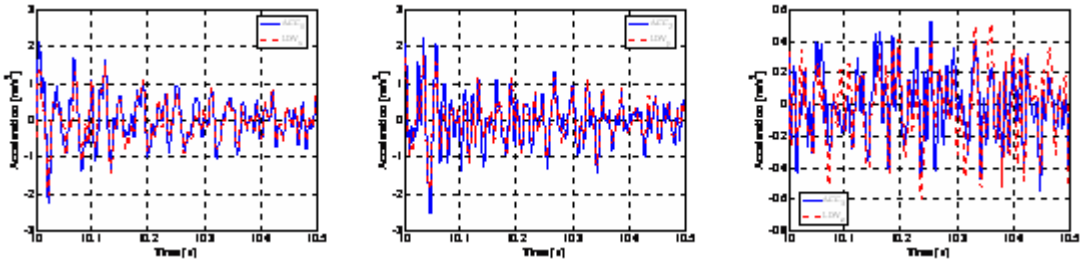


Fig.11 Field measurement results

7 DEVELOPMENT OF TRAIN INTELLIGENT MONITORING SYSTEM（TIMS）

Local railways, even though they have a very limited budget, need to keep high safety standards and to maintain their railway system. Especially, maintenance of track is very important because there is possibility that track irregularity induce serious accidents including derailment. Therefore, we try to develop low-priced railway monitoring system (TIMS) that can be installed in usual train for mainly Local railways.

Figure 12 shows the outline of train intelligent monitoring system (TIMS). The 3 axis accelerometer is installed on the floor of train. The track irregularity is calculated from the acceleration using the frequency response function of train. It's expected that maintenance of track is rationalized and the safety of train is enhanced because track condition can be monitored anytime.

Figure 13 shows the measurement system of TIMS. The measurement system consists of the 3 axis accelerometer to measure train acceleration, GPS sensor to determine train position and PC to record data. In this system, it is important that the train acceleration data and train position are synchronized. The train position is identified from GPS data and the train running distance got by integrating twice longitudinal acceleration. The output from accelerometer and GPS sensor are recorded in PC. In this system, AC power supply is needless because all equipments are worked by each battery. Figure 14 shows the acceleration measured twice by this system. Graf (1) shows a vertical acceleration and Graf (2) shows a lateral acceleration. It is confirmed that train acceleration depends on track irregularity because acceleration data have good repeatability in both direction.

The frequency response function is calculated using equation of motion of train model (Figure 15). The train model has 3 mass and 9 degree of freedom. The values needed in calculation are design values considering train aging.

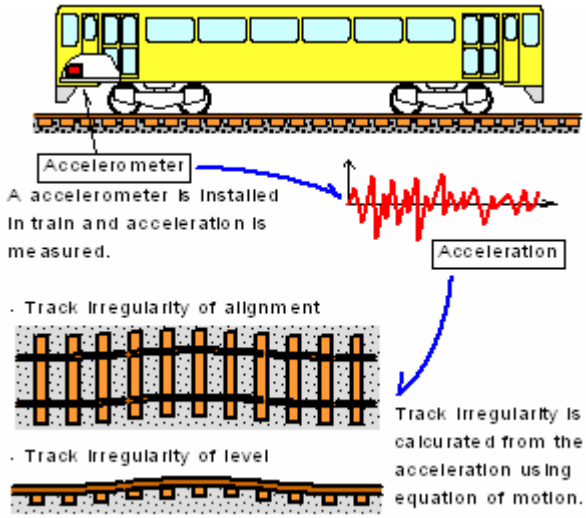


Figure 12. Outline of TIMS

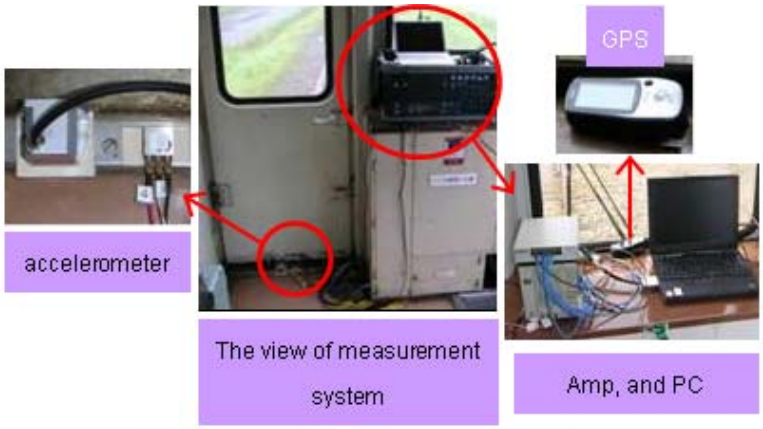


Figure 13. The measurement system

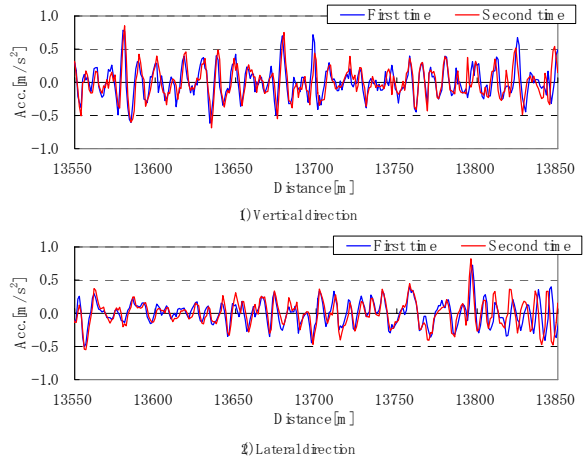


Figure 14. The example of train acceleration

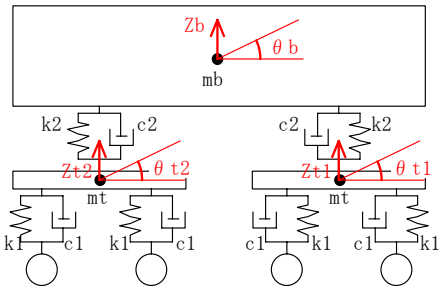


Figure 15. The train model

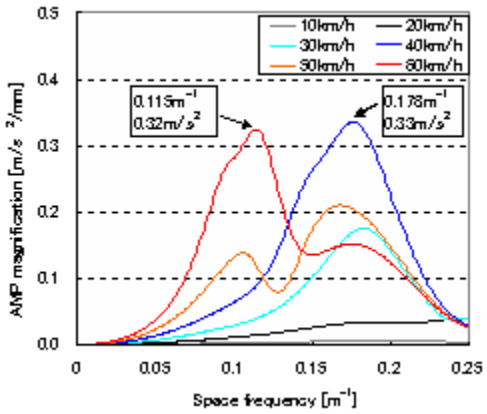


Figure 16. Frequency responses of train model

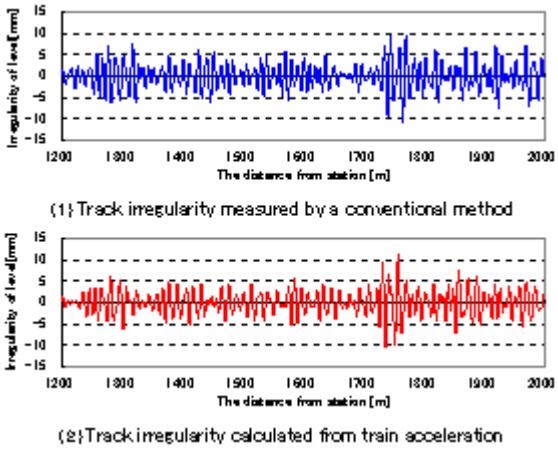


Figure 17. An example of results of identification

The equation of motion of train model is expressed as follow equation.

$$M\ddot{x} + C(\dot{x} - \dot{y}) + K(x - y) = 0 \quad \cdots (2)$$

where M is a mass matrix, C is a dumping matrix, K is a stiffness matrix, x is a deformation vector of the train, y is a track irregularity vector. Stationary track irregularities which have all kind of space frequency are inputted in this equation then the frequency response function is calculated. Figure 16 shows the frequency response function of train. The figure shows that train response depends on train speed and the space frequency of track irregularity. Track irregularity is calculated by multiplying acceleration got from measurement system by inverse function of the frequency response function.

Track irregularity identification is carried out in a section where train ran in constant speed about 50km/h. The track irregularity of level is aimed. Figure 17 shows the results of identification of track irregularity. Graf (1) is the track irregularity of level measured by a conventional method. Graf (2) is the track irregularity of level calculated by TIMS. The validity of this method is confirmed from comparing both results.

8 DEVELOPMENT OF VEHICLE INTELLIGENT MONITORING SYSTEM (VIMS)

In an urban highway network system such as Tokyo Metropolitan Expressway, to detect conditions of road pavement and expansion joints is a very important issue. Although accurate surface condition can be captured by using a road profiler system, the operating cost is expensive and development of a simpler and more inexpensive system is really needed to reduce monitoring cost. Inspection of the expansion joints from the inside is generally very difficult because of the squeezed space and visual inspection from the road surface is practically the only way. Road profile cannot be applicable to the expansion joints. The condition of the expansion joints is not well captured and failure of expansion joints indeed occurs, resulting in traffic accidents in some cases. “Vehicle Intelligent Monitoring System (VIMS)” developed for this purpose is described in this paper(Fujino 2005). A prototype of VIMS is installed to a motor car and measurement is made at the actual roads. Accuracy of measuring result and effectiveness of this system are demonstrated; the outline of the system and some of the measurement results are reported herein.

Concepts of VIMS are (Figure 18)are 1) VIMS should be a simple and inexpensive system so that it can be installed to an ordinary road patrol car. 2) To detect conditions of road pavements and expansion joints, dynamic response measured by VIMS installed to an ordinary road patrol car is used as a scale of the road pavements surface condition as well as the expansion joints.

Hardware of VIMS is mainly composed of three equipments(Figure 19) and they are;

1) Accelerometer (Measure the dynamic response to understand the condition of road pavements and expansion joints quantitatively) 2)GPS (Identify the position where the dynamic response is) 3)Notebook PC (Store the measurement data)

A prototype of VIMS was installed to a regular motor car (Figure 19) and a number of field tests were made at Tokyo Metropolitan Expressway Route No.7 (Komatsugawa-Line).

In this testing, installation to a regular car and data acquisition were confirmed.

To confirm repeatability of measured data, time history data are converted to distance-axis data using the velocity data form the vehicle. Figure 20 shows distance-axis of acceleration measured twice at the same road and Figure 21 shows enlarged figure at one part.

The acceleration of a portion exceeding a threshold of 100 gal is considered to be due to the expansion joints, and the others to be road pavements. RMS of acceleration at each portion is shown in Table 1. (The data shown in Figure 20 are used in the analysis) RMS accelerations for two measurements are more less the same and the largest difference is only 12 percents. From Figure 20 and Table 1, one can understand this measurement system is very stable.

In this analysis, 100 gal is used as a threshold level that distinguishes road pavements and expansion joints. Selection of an appropriate threshold level is a subject of future investigation.

The distance from the origin shown in Figure6 is obtained by integrating the velocity data of the vehicle, and it was found that each of distance-axis data is a little out of synchronization. To synchronize perfectly, the position data from GPS must be used. However, paths by position data from GPS don't perfectly agree at each measurement because of lack of GPS accuracy. (Figure 22) To synchronize distance-axis data of each time, how to use position data from GPS is a subject of future investigation.

To confirm that dynamic response by VIMS reflects conditions of road pavement, correlation between RMS of acceleration and the deepest rut of road pavement at every span is focused. (Figure 23) Coefficient of correlation between them is 0.8, and one can understand that dynamic response reflects conditions of road pavement.

Concerning expansion joints, dynamic response of expansion joints is bigger than that of road pavements, and so one can understand that dynamic response of expansion joint reflects conditions of expansion joint.

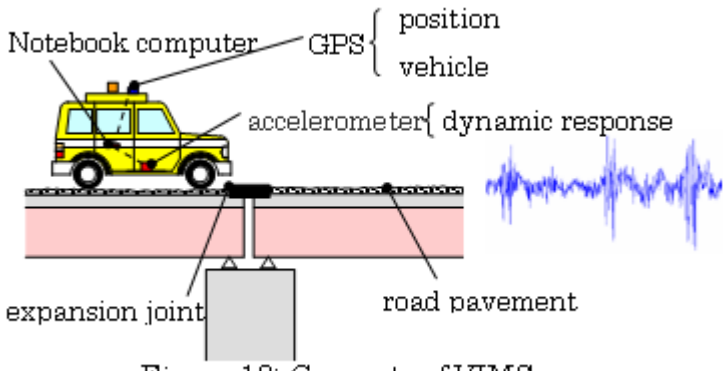


Figure 18: Concepts of VIMS

Table 1: RMS of acceleration at road pavements and

measurement		first time (gal)	second time (gal)	first/ second
Road pavement	Rd-1	31.3	28.0	1.12
	Rd-2	35.4	36.0	0.98
	Rd-3	28.5	29.0	0.98
	Rd-4	31.4	30.4	1.03
	Rd-5	32.7	33.7	0.97
Expansion joint	Ej-1	109.9	101.4	1.08
	Ej-2	101.8	97.8	1.04
	Ej-3	78.6	77.3	1.02
	Ej-4	101.6	95.1	1.07
	Ej-5	86.0	83.8	1.03

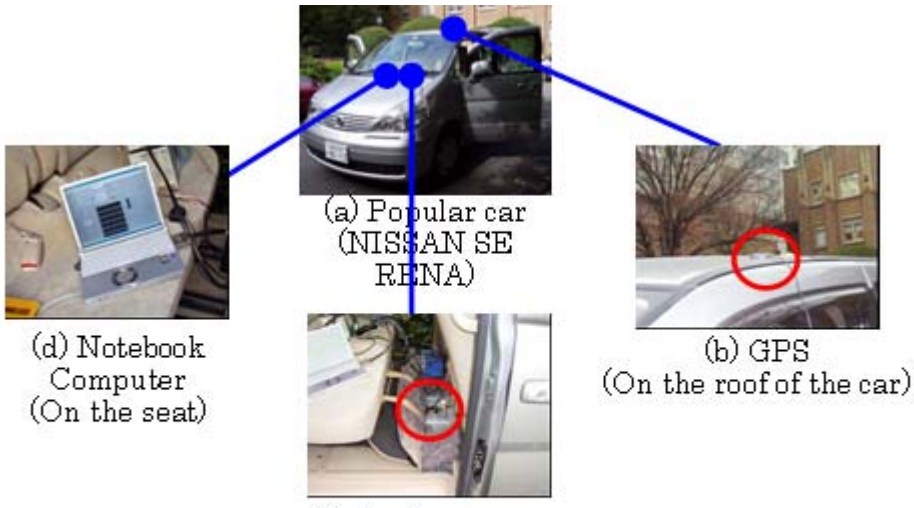


Figure 19 Prototype of VIMS installed to a popular car

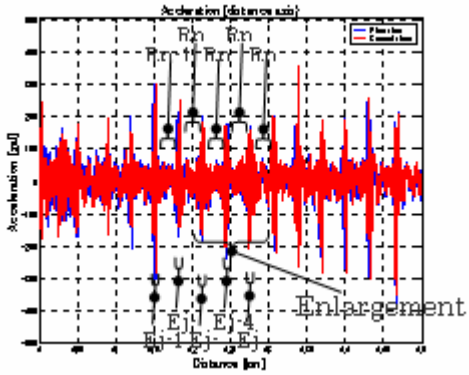


Figure 20. Distance-axis of acceleration

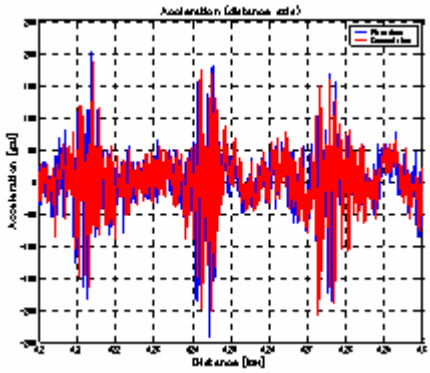


Figure 21. Distance-axis of acceleration

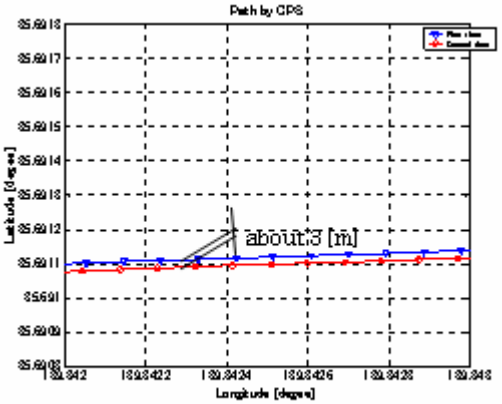


Figure 22: Paths of two times

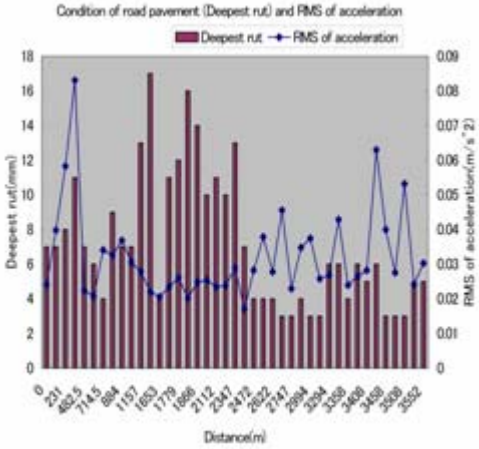


Figure 23. Conditions of road pavement & RMS

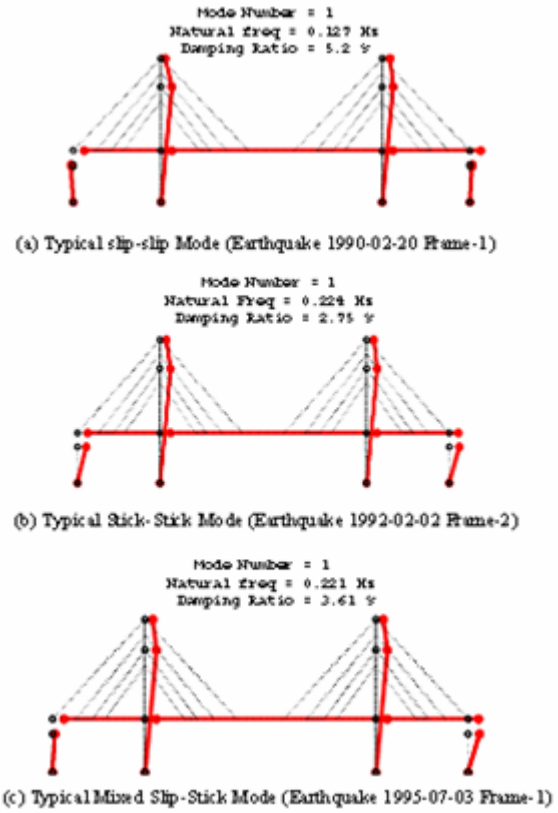


Fig.24 Three typical mode shapes in longitudinal direction

Table 2. Identified Natural Frequencies, Damping and type of mode

No	Earthquake (Frame number)	Maximum Input Acceleration (cm/s2)	Identified Freq (Hz)	Damp Ratio (%)	Type of mode		Pier-Girder relative modal displ (φ)	
					left-end pier	right-end pier	left-end pier	right-end pier
1	February 20th 1990 (3)	2.13	0.209	4.775	stick	slip	0.41	0.69
2	September 11th 1996 (3)	2.21	0.249	3.480	stick	stick	-0.29	-0.24
3	December 21st 1996 (2)	2.45	0.213	5.255	stick	stick	0.55	0.67
4	June 5th 1990 (1)	2.71	0.202	6.334	stick	stick	-0.25	-0.31
5	February 2nd 1992 (2)	4.16	0.223	2.751	stick	stick	0.20	-0.46
6	September 11th 1996 (2)	5.08	0.186	5.067	stick	stick	-0.30	-0.48
7	December 21st 1996 (1)	5.38	0.216	9.214	stick	slip	0.24	-0.82
8	July 3rd 1995 (1)	5.62	0.221	3.614	slip	stick	0.79	-0.12
9	February 20th 1990 (2)	7.09	0.129	11.630	stick	slip	0.20	-1.02
10	August 9th 1997 (1)	6.1	0.234	6.215	stick	stick	-0.11	0.36
11	September 11th 1996 (1)	8.5	0.248	2.979	stick	stick	-0.29	-0.24
12	July 9th 1997 (1)	9.49	0.247	9.980	stick	stick	0.54	0.28
13	February 20th 1990 (1)	12.04	0.128	5.203	slip	slip	0.72	0.60
14	February 2nd 1992 (1)	13.08	0.133	4.307	slip	slip	0.80	1.18

9 PERFORMANCE EVALUATION OF YOKOHAMA-BAY BRIDGE FROM SEISMIC RECORDSD

The earthquake resistance of Yokohama bay bridge was carefully reviewed in design. To limit the effect of large inertia force from superstructure into sub-structures, the towers and end-piers are connected to the main girder by link-bearing connection. These links are designed to function as perfectly hinged connection during bridge movement in the longitudinal axis. Hence, this prevents excessive inertia force from superstructure to be transmitted to the end-piers and other sub-structures. This reports describes the investigation of performance evaluation of the link-bearing connection using system identification technique. Earthquake responses from various ground motion are used for modal analysis using time domain state-space system identification to investigate the performance of the link-bearing connection during earthquake.

Using a finite dimensional, discrete-time, linear, time-invariant, state variable dynamical system, the earthquake-induced equation of motion of structure can be expressed as:

$$x(k+1) = \mathbf{[A]}\, x(k) + \mathbf{[B]}\, z(k) \quad (3)$$

$$y(k) = \mathbf{[R]}\, x(k) + \mathbf{[D]}\, z(k)$$

Vector ***z***(k) denotes the *q* x 1 input acceleration of ground motion and ***y***(k) denotes the *m* x 1, structural acceleration responses as outputs. The integer *k=0,1,2…l* denotes the time-step number i.e. ***x***(k+1)=***x***(k(Δt)+ Δt), with Δt being the time interval. Further expanding the above state-space equations into a matrix equation of p sampled data, using the observability matrix matrix $\mathbf{[O_p]}$ and $\mathbf{[T_p]}$ yields,

$$Y_p(k) = [O_p]X(k) + [T_p]Z_p(k) \quad (4)$$

The objective of state-space system identification is to determine the unknown matrices **[A],[B],[R]** and **[D],** which are embedded in the observability matrix $\mathbf{[O_p]}$ and the Toeplitz matrix of system Markov parameters $\mathbf{[T_p]}$ from sets of input-output data. One can start by computing the matrix $\mathbf{[O_p]}$ and $\mathbf{[T_p]}$, and later compute the state matrix [**A]** and output influence matrix **[R]** using algebraic manipulation of the observability matrix $\mathbf{[O_p]}$ such as:

$$\mathbf{[A]} = [O_p^*](1:(p-1)m,:)[O_p](m+1:pm,:) \quad (5)$$

Adopting this approach, reference [1] suggested that the observability matrix $\mathbf{[O_p]}$ could be obtained by factoring the information matrix $R_{hh} = R_{yy} - R_{yz}R_{zz}^{-1}R_{yz}^T$,

which consists of correlation (R) of input (z) and output (y) data such follows:

$$R_{hh} = [O_p]\hat{R}_{xx}[O_p]^T = H_{2N}\Sigma_{2N}^2 H_{2N}^T \quad (6)$$

This equality produces one solution each for $\mathbf{[O_p]}$ $= H_{2N}$. Hence, the system matrix **[A]** is obtained. M odal parameters of structural system can be estimate d by solving the eigenvalues problem of matrix**[A]**as $[A][\Phi] = [\Phi][\tilde{\Lambda}]$, and after transformation yields the natural frequency and modal damping ratio:

$$\omega_i = \sqrt{\mathrm{Re}(\lambda_i)^2 + \mathrm{Im}(\lambda_i)^2} \quad \xi_i = \frac{-\mathrm{Re}(\lambda_i)}{\omega_i} \quad (7)$$

The mode shapes matrix in coordinate system is obtained by transforming the eigenvectors using the output-transformation matrix **[R]**. The complete

procedure of system identification can be found in (Siringoringo *et al.* 2005.

9.1 *Application to the Yokohama Bay Bridge*

For the purpose of evaluation of link-bearing connection only the first longitudinal mode from system identification is presented. Responses were obtained from 20 output sensors: 4 sensors along side the girder, 12 sensors at the both towers and 4 sensors at the both piers, all are measuring accelerations in the longitudinal (x) direction. As inputs, four triaxial sensors (x,y and z directions) at the bottom of the left-end and right-end piers and both of towers were utilized. The outputs end-piers sensors are located on the pier-cap and the girder just above the adjacent pier cap. All responses were sampled at sampling rate of 100 Hz. Responses for analysis were obtained from acceleration records of eight earthquakes recorded from 1990 to 1997. These records were divided into fourteen frames to (see Table I for detail). Modal parameters for these records were calculated separately.

Three typical first longitudinal modes(Figure 24) were obtained from system identification. The first mode was identified at around 0.128 Hz to 1.334 Hz, which is a typical first mode when compared to the finite element model [3]. This mode shows a large relative modal displacement between pier-cap of end-piers and the girder. This large gap indicates that a slip mechanism between pier-cap and girder has taken place during the earthquake as in the condition of a fully hinged connection. The second and third modes were identified at higher frequencies between 0.18 to 0.24 Hz. The second mode displays a mixed mechanism in which one of the end-pier slips while the other one remains closely connected with the girder. In the third mode, smaller relative modal displacement between pier cap and girder was observed at the both of end-piers. In this mode both of end-piers remain stick or closely connected to the girder, which indicates that the hinged mechanism of link bearing has yet to take place. The third mode suggests a stiffer connection and consequently, has a higher natural frequency.

To quantify the relative modal displacement between pier-cap and the adjacent girder a normalized relative modal displacement index (φ) is calculated for each end-pier. This index is defined as:

$$\varphi = \phi_{girder} - \phi_{pier-cap} \tag{8}$$

where the ϕ denotes the modal displacement normalized to unity for maximum value. When the pier-cap and girder are largely separated the index value is closer to unity, otherwise its value is close to zero. The results of system identification from 14 frames of earthquake are listed in Table 2. The last two columns list the index of relative modal displacement between pier-caps and girder. These values determine the condition of end-pier and girder connection and thus the types of identified mode either slip or stick mode. Modes with the value of φ larger than 0.6 is considered as slip mode.

Based on system identification, it is found that
1)During small earthquake the link-bearing connection has yet to function as a full-hinged connection. Therefore a stiffer connection with higher mode was observed. The mixed slip-stick mode was observed during smaller and moderate earthquake. The full-hinged type of connection at both of end-piers was only observed during large earthquake. This slip-slip mode suggests that link-bearings have performed as its intended function.
2)This investigation shows that during earthquake connection might not perform as it is predicted (fully hinged) or as it is modeled in finite-element. And this deviation or unwanted mechanism would have not been discovered without the vibration based system identification.

10 CONCLUDING REMARKS

Present condition of bridges and transportation systems in Japan and importance of structural health monitoring are reviewed. Vibration-based structural health monitoring research and developments being conducted at the Bridge and Structure Laboratory, University of Tokyo are presented. Development of structural health monitoring technology is extremely important to efficient operation and reduction of the cost of the infrastructure maintenance. Theoretical as well as practical approaches have to be made continuously toward the goal.

REFERENCES

Fujino, Y.,2002. Vibration control and monitoring of long-span bridges –recent developments and practice in Japan, Journal of Construction Steel Resrach, No.57, 71-97, 2002.

Fujino, Y., et al. 2005. Development of Vehicle Intelligent Monitoring System(VIMS), SPIE International Symposia on Smart Structures & Materials/NDE, Conference#5765.

Kaito K, Abe M., Fujono Y., and Yoda H. 2000. Detection of Structural Damage by Ambient Vibration Measurement using Laser Doppler Vibrometer, Non-Destructive Testing in Civil Engineering 2000, Uomoto (ed.), Tokyo.

Kaito, K., Abe, M., Fujino, Y. 2001. An experimental modal analysis for RC bridge decks based on non-contact vibration measurement," Proceedings of the

International Modal Analysis Conference - IMAC, v 2, 1561-1567

Kaito, K., Masato A and Fujino Y. 2005.6. Development of non-contact scanning vibration measurement system for real-scale structures, J. of Structure and Infrastructure Engineering, Vol..1, no.3, 189-205.

Kobayashi, Y., Miki, C and Tanabe, A. 2004. Long-term monitoring of traffic loads by automatic real-time weigh-in-motion, Journal of Civil Engineering, JSCE, No. 773, (in Japanese).

Miyashita, T. and Fujino, Y. 2005. Development of Three Dimensional Vibration Measurement System Using Laser Doppler Vibrometers", Proc. of Int. Conference on Advances in Experimental Structural Engineering, Nagoya.

Miyashita, T., Ishi, H., Fujino, Y. Shoji, A. And Seki, M. 2005. Clarification of the Effect of High-Speed Train Induced Vibrations on a Railway Steel Box Girder Bridge Using Laser Doppler Vibrometer, Proc. of Experimental Vibration Analysis for Civil Engineering Structures, Bordeaux, France.

Nagayama, T., Abe, M., Fujino, Y. and Ikeda, K. 2005. 7. Structural identification of non-proportionally damped systems and its application to a full scale suspension bridge, Journal of Structural Eng.(to appear)

Siringoringo, D M and Fujino Y. Observation of Dynamic Performance of Yokohama-Bay Bridge using system identification from seismic records", Journal of Structural Control and Monitoring, (accepted for publication)

Yamada, K., Ojio, T., Fukada, S. and Kajikawa. Y. 2004. Use of Bridge Weigh-in-Motion System with Environmental Monitoring of Viaducts", Proc. of 2nd Int. Workshop on Structural Health Monitoring of innovative Civil Engineering Structures, 335-345.

Xia, X. Fujino, Y., Abe, M. and Murakoshi, J. 2005.3. Short-term and long-term health monitoring experience of a short highway bridge: case study, Journal of Bridge Structures, Vol.1, No1, 43-53.

Wu, Z., Xu, B. and Harada, T. 2003. Review on Structural Health Monitoring for Infrastructure", Journal of Applied Mechanics, JSCE, Vol. 6, 1-12. (in Japanese).