OVERVIEW AND APPLICATION OF LOCAL EXCITATION METHOD (LEM) FOR MONITORING OF SMALL DAMAGE OF REAL BRIDGES

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ABSTRACT : By using piezoelectric actuator real steel bridge can be accelerated locally to detect the effect of small damage like fatigue crack and connection damage as a response change of vibration. Power spectrum density (PSD) function of the response can give us the indication of existence of damage after the damage progressed. Detail analysis of PSD before and after damage can give us the information of location and size (area) of damage. The experimental result of damage detection by using local excitation method (LEM) on real bridge is shown in this paper.

KEYWORDS: Structural health monitoring , Local excitation, Power spectrum density(PSD), Piezoelectric actuator, Vibration-based damage identification

1. BACKGROUND

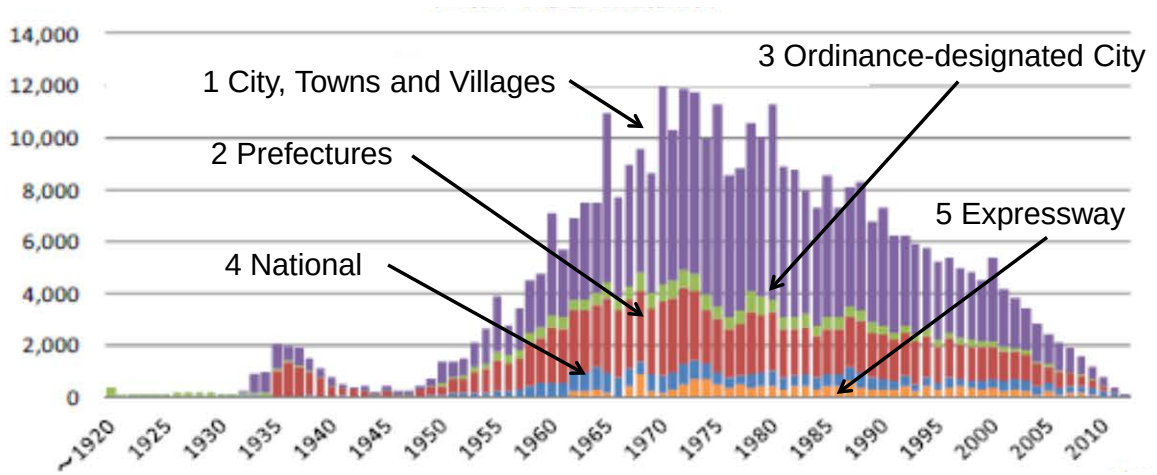

Fig. 1 Bridge construction numbers along with its years

During 1960-1990 much bridges have constructed according to rapid economic growth of Japan as shown in Fig. 1 and nowadays total about 700,000 bridges were stocked. Then 536,000 bridges (68%) are managed by city, town and villages and 133,000 (19%) are managed by prefectures as shown in Fig. 2. As a result of this history short and medium span bridges are dominated in those total 669,000 bridges and it is clear that maintenance of structural health and its monitoring of those bridges are very important task here.

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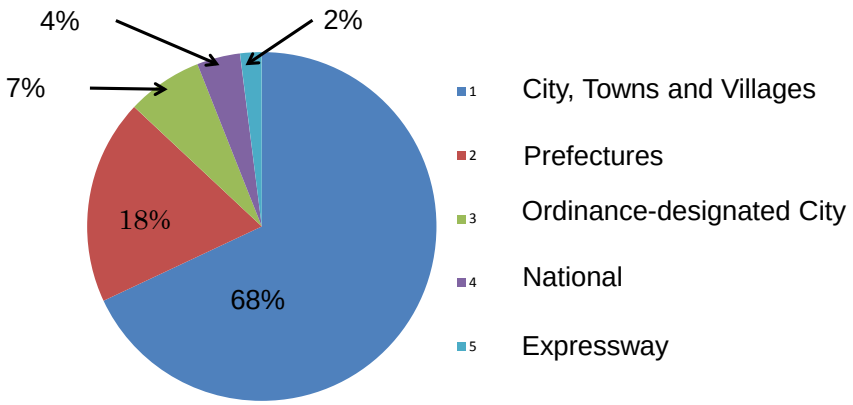


Fig. 2 Distribution of management agency over 700,000 bridges

2. INTRODUCTION

In general bridge inspection the damage investigation and structural health monitoring of existing bridges have become important task for the maintenance of infrastructures. Visual inspection is mostly used in the field and once the damage was found detailed inspection will start. Then vibration-based damage detection is well used as the one of in-situ inspection method of bridges [1]-[5].
When the changes of structural characteristics of bridges are detected through vibration-based damage detection, these measured results can be further used for a bridge monitoring system to evaluate the condition, or damage, or remaining capacity of the structure.

The general modal parameters, however, have been shown to be less sensitive in the detection of bridge damages. And the modal parameters of conventional modal testing such as frequencies and modal damping are global parameters, which cannot locate the damages properly [6]. Research efforts have been made to detect structural damage directly from dynamic response measurements in the time domain, e.g. the random decrement technique [7], or from frequency response functions (FRF) [8].

Also, some damage detection methods have been proposed to detect damage using system identification techniques [9]-[10]. In this paper, an algorithm based on changes in Power Spectrum Density (PSD) is shown. The algorithm is used to detect damage, locate its position and monitor the increase of damage using only the measured data without the need for any modal identification or numerical models. The method is applied to the experimental data extracted from a real steel bridge after inducing some damages to its members.
One difficulty with determining dynamic parameters of a structure through ambient vibrations is that the forcing function is not precisely characterized. The implementation of piezoelectric actuators [11]-[13] as a local excitation source for real structures such as steel bridges of medium or short span, is presented in this paper. The advantages of using piezoelectric actuators instead of shakers, hammers or ambient vibrations will be discussed in details in the following chapters [14]-[22].

3. DAMAGE IDENTIFICATION ALGORITHM

In the area of non-destructive damage detection using change of modal parameters many techniques have been proposed. In many structures, however, only few modes are available which may be used to evaluate the effect of detecting and localizing damage using these techniques.
In order to overcome the problem of the limited number of identified modal parameters, PSD result estimated through accelerometer measurements at all frequencies in the measured range and not just

the modal frequencies, will be able to be compared before and after damage progressed using the proposed method. PSD data can then be analyzed using statistical procedures to determine the damage location, as will be explained in details in this section.

PSD is determined from the acceleration time histories without the need to measure the excitation forces. Let $G_i(f)$ denotes the PSD magnitude measured at channel number i at frequency value f. The absolute difference in PSD magnitude before and after damage can then be defined as

$$D_i = \frac{\left| G_i(f) - G^*{}_i(f) \right|}{1 + \left| G_i(f) \right|} \tag{1}$$

where $G_i(f)$ and $G_i^*(f)$ represent PSD magnitude for the undamaged and damaged structures, respectively. In the denominator of Eq.(1) 1 is added for analytical reason not to set zero just in case. The excitation forces used for the undamaged and damaged structure must have the same amplitude, location and waveform in order to ensure that the changes in PSD data are mainly due to damage and not due to the change in excitation force characteristics. When the change in PSD is measured at different frequencies on the measurement range from f_1 to f_m , a matrix [**D**] can be formulated as follows

$$\mathbf{D} = \begin{bmatrix} D_1(f_1) & D_2(f_1) & ..... & D_n(f_1) \\ D_1(f_2) & D_2(f_2) & ..... & D_n(f_2) \\ . & . & ..... & . \\ . & . & ..... & . \\ . & . & ..... & . \\ D_1(f_m) & D_2(f_m) & ..... & D_n(f_m) \end{bmatrix} \tag{2}$$

where n represents the number of measuring points. In matrix [**D**], every row represents the changes in PSD at different measuring channels but at the same frequency value. The summation of PSD changes over different frequencies can be used as the indicator of damage occurrence and the increase in damage. In other words, the first damage indicator is calculated from the sum of columns of matrix [**D**] as total change (**TC**)

$$\mathbf{SM} = \left\{ \sum_f D_1(f) \quad \sum_f D_2(f) \quad . \quad . \quad . \quad \sum_f D_n(f) \right\} \tag{3}$$

Then total changes **SM** in PSD is calculated from the sum of columns of matrix [D] on each nodes.

However, it was found that total change **SM** is a weak indicator of damage localization. A statistical decision making procedure is employed to determine the location of damage. The first step in this procedure is the selection of the maximum change in PSD at each frequency value (the maximum value in each row of matrix [**D**]) and dividing all other changes in PSD measured at other nodes. For example in matrix [**D**], if $D_3(f_1)$ is the maximum value in the first row, then this value will be used as denominator $D_3(f_1)$ and all other values in this row are divided as shown in Eq.(4). The same process is applied to the different rows in matrix [**D**] to formulate the matrix of maximum changes of PSD at different frequencies, [C]

In order to monitor the frequency of damage detection at any node, a new matrix [**C**] is formulated. This procedure can normalize the big differences over frequency ranges at each measuring nodes. For example in the matrix [**C**], a value of 1 is used corresponding to the locations of $C_3(f_1)$, $C_2(f_2)$ and so on, as shown in Eq.(4)

$$\mathrm{C}=\begin{bmatrix} C_1(f_1)=\dfrac{D_1(f_1)}{D_3(f_1)} & C_2(f_1)=\dfrac{D_2(f_1)}{D_3(f_1)} & \cdots & C_n(f_1)=\dfrac{D_n(f_1)}{D_3(f_1)} \\ C_1(f_2)=\dfrac{D_1(f_2)}{D_2(f_2)} & C_2(f_2)=\dfrac{D_2(f_2)}{D_2(f_2)} & \cdots & C_n(f_2)=\dfrac{D_n(f_2)}{D_2(f_2)} \\ \vdots & \vdots & & \vdots \\ \vdots & \vdots & & \vdots \\ C_1(f_m)=\dfrac{D_1(f_m)}{D_5(f_m)} & C_2(f_m)=\dfrac{D_2(f_m)}{D_5(f_m)} & \cdots & C_n(f_m)=\dfrac{D_n(f_m)}{D_5(f_m)} \end{bmatrix} \tag{4}$$

$$\mathbf{SC}=\left\{\sum_f C_1(f) \quad \sum_f C_2(f) \quad . \quad . \quad . \quad \sum_f C_n(f)\right\} \tag{5}$$

Then **SC** is also the indicator of damage occurrence and it is called damage detection (**DD**) in this paper. The first damage localization indicator **D I 0** is defined as the scalar product of {***SM***} and {***SC***} as shown in the following expression;

$$\mathbf{D\ I\ 0} = \{SM(1)\times SC(1) \quad SM(2)\times SC(2) \ldots. \quad SM(n)\times SC(n)\} \tag{6}$$

In order to reduce the effect of noise or measurement errors, a value of one and two times standard deviation of the elements in vector {**SM**} will be subtracted from the vector {**SM**}. Any resulting negative values will be removed. The same procedure will be applied to the vector {**SC**} . Then Eq. (6) is modified for second and third damage location indicators **D I 1** and **D I 2** as follows;

$$\boldsymbol{SMD1}=\left\{\sum_f D_1(f)-\sigma \quad \sum_f D_2(f)-\sigma \quad . \quad . \quad . \quad \sum_f D_n(f)-\sigma\right\} \tag{7}$$

where $\sigma=\sqrt{\sum_{i=1}^{n}(SM(i)-\overline{SM})^2}\Big/(n-1)$, $\overline{SM}=\sum_{i=1}^{n}SM(i)\Big/n$,

$$\boldsymbol{SCD1}=\left\{\sum_f C_1(f)-\lambda \quad \sum_f C_2(f)-\lambda \quad . \quad . \quad . \quad \sum_f C_n(f)-\lambda\right\} \tag{8}$$

where $\lambda=\sqrt{\sum_{i=1}^{n}(SC(i)-\overline{SC})^2}\Big/(n-1)$, $\overline{SC}=\sum_{i=1}^{n}SC(i)\Big/n$.

The second damage localization indicator **D I 1** is defined as the scalar product of {***SMD1***} and {***SCD1***} as follows;

$$\mathbf{D\ I\ 1}=\{SMD1(1)\times SCD1(1) \quad SMD1(2)\times SCD1(2) \ldots. \quad SMD1(n)\times SCD1(n)\} \tag{9}$$

And for third damage location indicator **D I 2**

$$\boldsymbol{SMD2}=\left\{\sum_f D_1(f)-2\sigma \quad \sum_f D_2(f)-2\sigma \quad . \quad . \quad . \quad \sum_f D_n(f)-2\sigma\right\} \tag{10}$$

$$\boldsymbol{SCD2} = \left\{ \sum_f C_1(f) - 2\lambda \quad \sum_f C_2(f) - 2\lambda \quad . \quad . \quad . \quad \sum_f C_n(f) - 2\lambda \right\} \tag{11}$$

Then damage localization indicator **D I 2** is defined as the scalar product of {***SMD2***} and {***SCD2***} as follows;

$$\textbf{D I 2} = \{ SMD2(1) \times SCD2(1) \quad SMD2(2) \times SCD2(2) \ldots SMD2(n) \times SCD2(n) \} \tag{12}$$

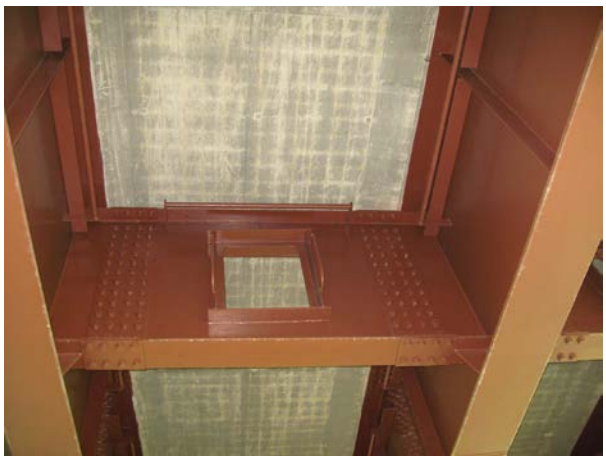
Damage localization indicators **D I 0** , **D I 1** and **D I 2** will be used to determine the damage location and the total change **TC** in PSD (Eq. (3)) and damage detection **DD**(Eq. (5) will be used to detect the occurrence of damage and monitor the growth in damage.

4. DAMAGE DETECTION OF MIDIUM SPAN BRIDGE BY USING LEM

4.1 EXPERIMENTAL SETUP OF MEASUREMENT

(a) Overview of bridge

(b) Main girder and Lateral



(c) Cross Beam

Photo 1 (a), (b) and (c)
Bridge used for measurement

Excitation System

Piezoelectric actuator as sown in Fig. 3 is used to excite the real steel structure of Photo 1 (a), (b) and (c) with wide range of excitation frequencies, but the excitation wave and the excitation amplitude are not changed. A wave function generator is used to adjust the required excitation frequency range, the time in which the actuator reaches the maximum frequency and then excitation wave form. The adjusted wave is transferred to the power supplier, which in turn provides the actuator with the adjusted voltage. Actuators were fixed to the steel girder by magnetic holders. A steel spring is fixed over the actuator to control the excitation force amplitude. The main characteristics of the equipment used are shown in Table 1.

As can be seen in Fig.4, the excitation frequency is gradually increasing over time until it reaches its maximum value after the designated time (15 seconds in this example). In this example, the excitation wave form is a sweep function. It should also be noted that because the actuator is pressed but not glued to the structure, the actuator provides pressure force only.

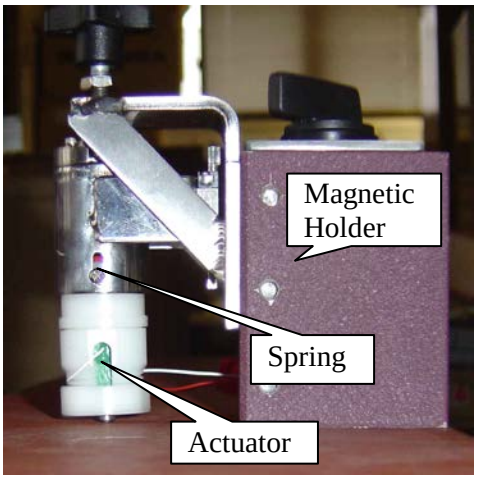


Fig. 3 Actuator and holder

Piezo-electric actuator	
Dimension(W*T*H)	6*8*10 (mm)
Min. and Max. Freq.	0 - 982 Hz
Displacement(100V)	12.3 μm
Max Force	350 N
Wave function generator	
Min. and Max. Freq.	0.01 μHz- 15MHz
Max. Output Volt	20Vp-p/ OPEN
	±10V/OPEN
Power source	
Max. Power	150 V

Table 1 Specification of Actuator

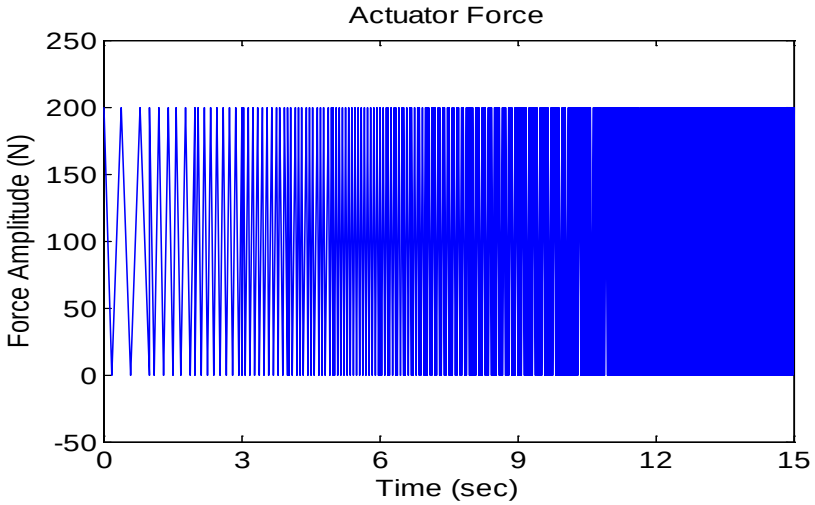


Fig. 4 Wave form of force input on structure

Sensor Distribution

The experiment was done on a real composite steel bridge with concrete piers and deck. The details of the span of the bridge measured are shown in Fig. 5 and Fig. 6. The bridge is 4 span continuous type and measurement was done for the end span of the bridge. The inspected portion consists of four main longitudinal girders spanning 30m and spaced at 8.5m in the transverse direction. Two piezoelectric actuators attached on the web are used to excite the chosen main girder of bridge in the out of plane direction. The details of the location of the actuators are shown in Fig. 5 and Fig. 6. The plan of the steel girder in the portion between the two piers inspected is shown in Fig. 5 along with the positions of the accelerometers. The sectional side view elevation of the cross beam of the bridge is shown in Fig.6. G1-G4 represents the longitudinal main girders while 16 channels in all Ch1-16 are used to evaluate the response at different portions of the bridge.

The locations of the accelerometer position are chosen as an array in such a way that they correspond to the locations in the vicinity of damage and excitation, so that any minor changes in the bridge could be easily detected.

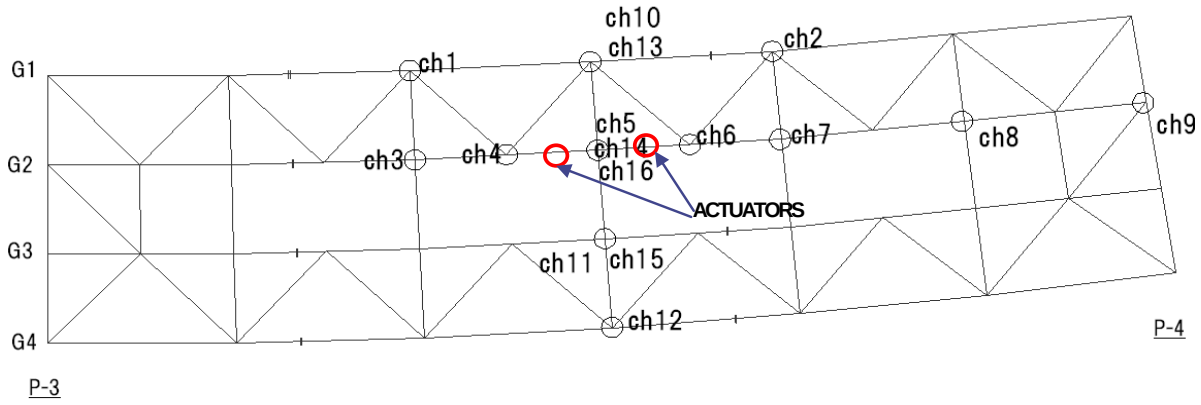


Fig. 5 Details of accelerometers and actuators

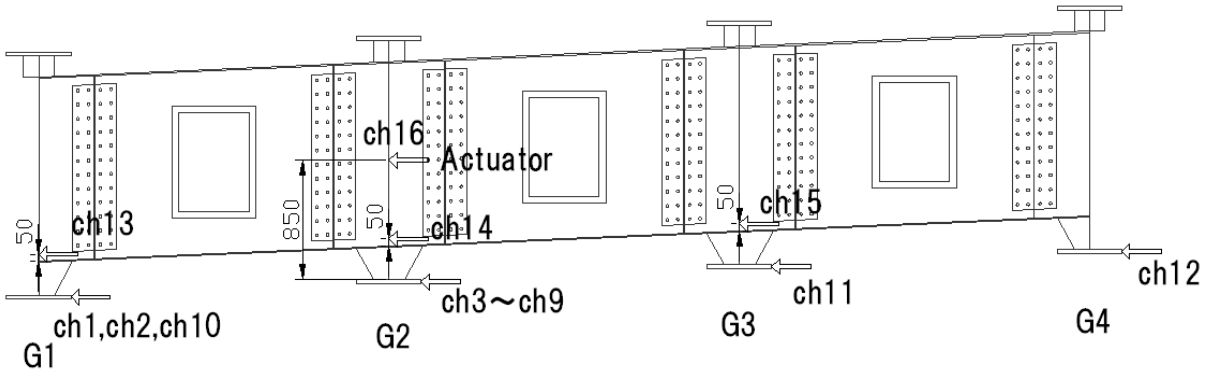


Fig. 6 Typical side view of cross beam and accelerometer locations

All the connections of the bridge girder were with bolts and hence the releasing of the bolts on the cross beam was adopted to introduce artificial damage to the cross beam connecting to the longitudinal main girder (G2). As explained earlier two actuators, located at approximately the middle part on the web of the main girder in the horizontal direction as shown in Fig. 5 and Fig. 6, were used for excitation. The excitation forces used for the undamaged and the damaged structures were sweep, equal in amplitude and had the same vibration waveform. The actuator force amplitude was estimated

to be around 200N by measuring the displacement in the spring that presses the actuator (Fig. 3). Although this force amplitude was very small compared with the shaker force or ambient vibration, it was sufficient to excite the web of the main girder even at the farthest accelerometer. The acceleration response is collected at various locations (Channels 1-16). One accelerometer each was mounted at the various nodal points as shown in Fig. 5 and Fig. 6. For this measurement, 20 second time histories were sampled at a rate of 6500Hz, producing 130000 time points. A matrix of response data sets was recorded before and after damage (Photo 2 and Photo 3).

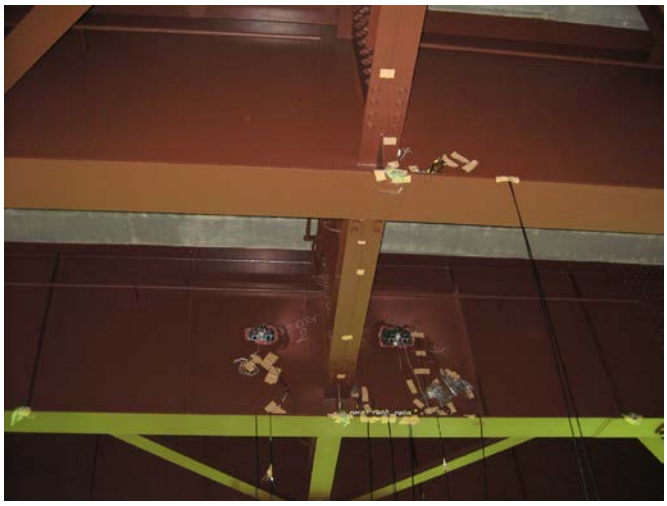


Photo 2 Two Actuators Set on the Web Plate of Main Girder

Photo 3 Measurement Station

Artificial Damage by Releasing Connection Bolts

In this example the three cases namely **D3**, **D4** and **D5** are taken to compare with the intact condition named **D0**. The damages to the bridge are introduced by releasing the bolts of the cross beam which are connecting to the main longitudinal girders at span center. In this example the cross beam chosen is the one which is supporting the main longitudinal girder G2. The three damage cases considered here are ;

Case D3: Releasing of sixteen bolts (4 x 4) from the lower side of the cross beam on either side of the main longitudinal girder (Fig. 7). This condition is the damage case **D3**.

Case D4: Releasing of thirty-two bolts (8 x 4) on the cross beam on either side of the main longitudinal girder (Fig. 8). This condition is the damage case **D4**.

Case D5: Releasing of forty-eight bolts (12 x 4) on the cross beam on either side of the main longitudinal girder (Fig. 9). This condition is the damage case **D5**.

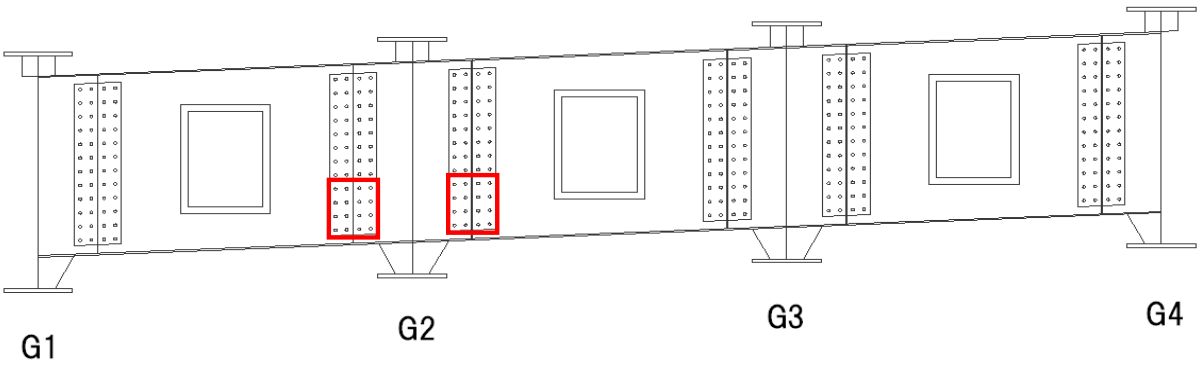


Fig. 7 Damage Case D3: Release of 4 X 4 bolts

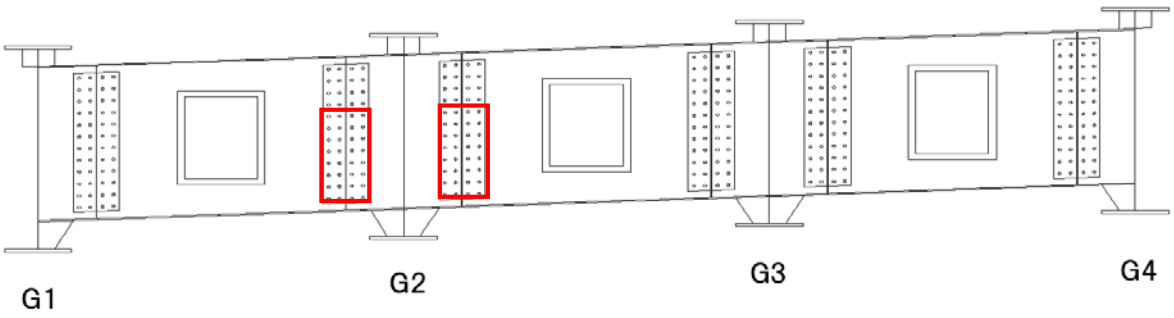


Fig. 8 Damage Case D4: Release of 8 X 4 bolts

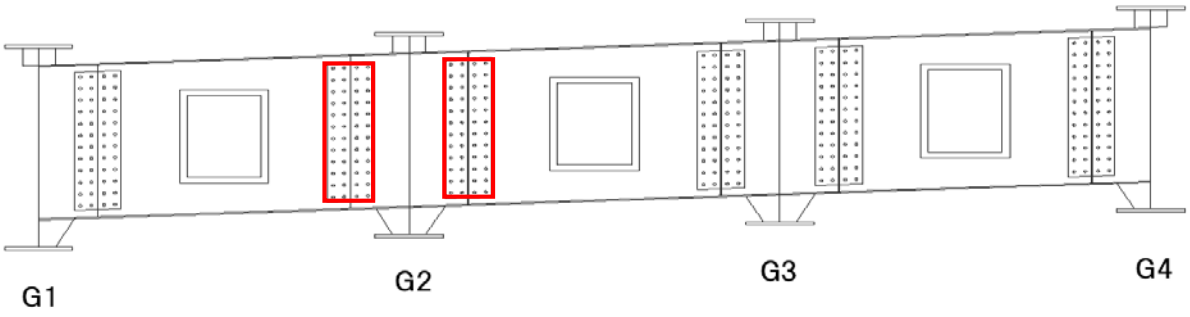


Fig. 9 Damage Case D5: Release of 12 X 4 bolts

4.2 RESULT OF DAMAGE IDENTIFICATION

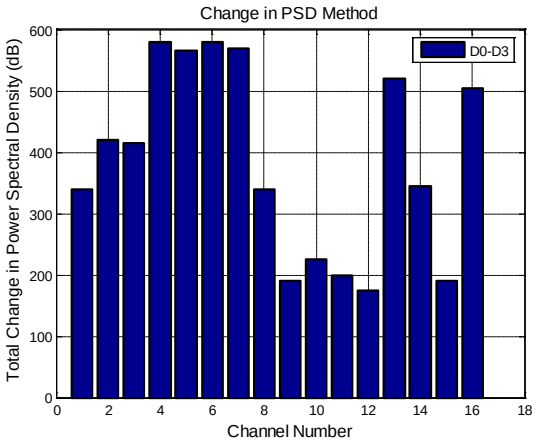


Fig. 10 Total Change on PSD for Case D3

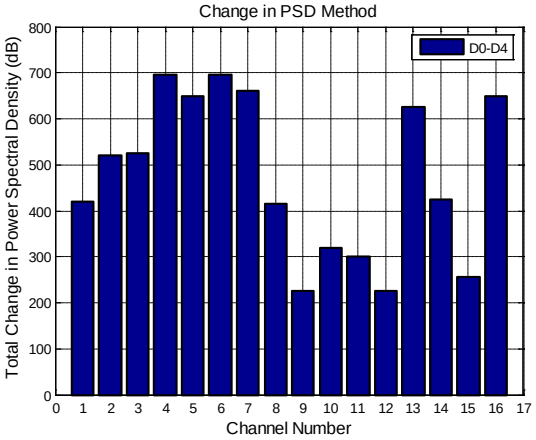


Fig. 11 Total Change on PSD for Case D4

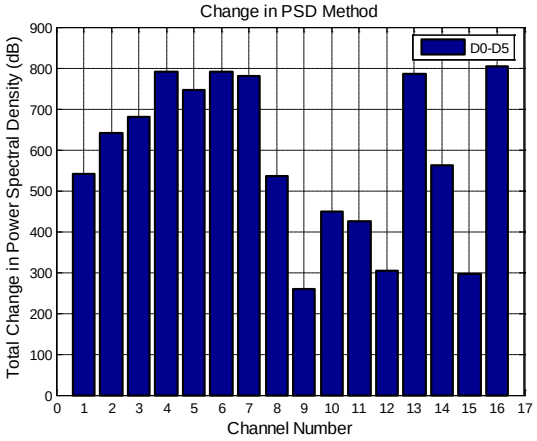


Fig. 12 Total change on PSD for Case D5

The results of Total change **(TC)** given by Eq. (3) are shown in Fig. 10 for damage case **D3**, Fig. 11 for **D4** and Fig. 12 for **D5**. In all cases sensors which detected the effect of damage are well coincide with the location where artificial damage were introduced. The results for damage cases are given by comparing with intact condition of **D0**. It is shown that the method using LEM can detect well the damage condition for medium span bridge.

5. DAMAGE DETECTION OF SHORT SPAN BRIDGE BY USING LEM

5.1 EXPERIMENTAL SETUP OF MEASUREMENT

Photo 4 Short Span Bridge Photo used for Measurement (Assembled steel I beam, length 6.96m Railway bridge)

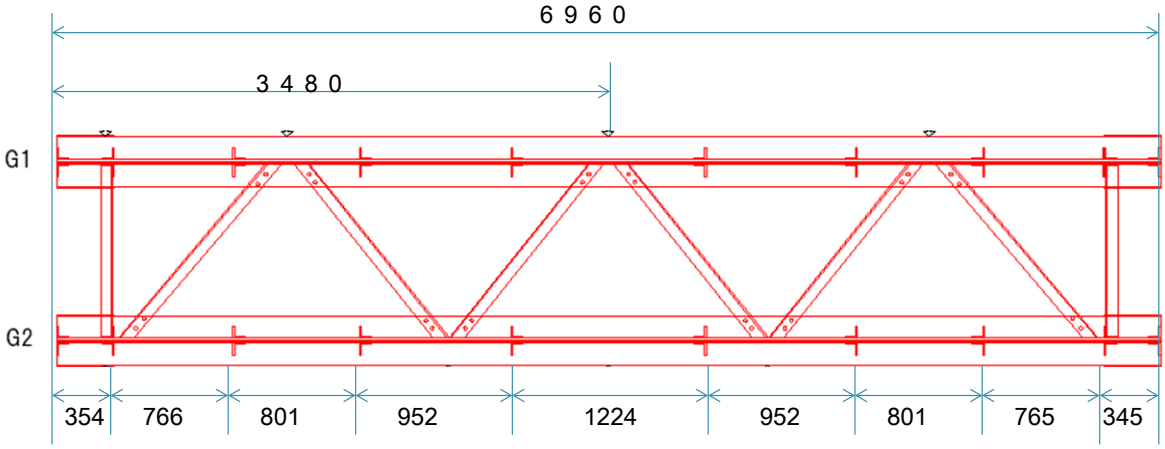


Fig. 13 Bridge Dimensions (Unit: mm)

Bridge which was constructed in the year of 1909, has elements of 2 main girders (assembled I beams by rivets) and 2 diaphragms at the end of girder. Horizontal laterals are connected at the upper flange of girder by rivets as shown in Photo 4. In experiment bridge is supported on two wooden blocks at the end with fixation. Detail dimension of bridge is shown in Fig.13 and Table 2.

Cross sectional dimension (L type angle) of vertical stiffeners is 90× 80× 10 (mm) .

Table 2 Dimensions of assembled I beam girder

	Total length	6960
	Girder spacing	1127
Web	Length	6960
	Height	686
	Thickness	10
U-Flg	Length	6960
	Width	310
	Thickness	12
L-Flg	Length	6960
	Width	310
	Thickness	12
		(mm)

5.2 SENSOR ARRANGEMENT

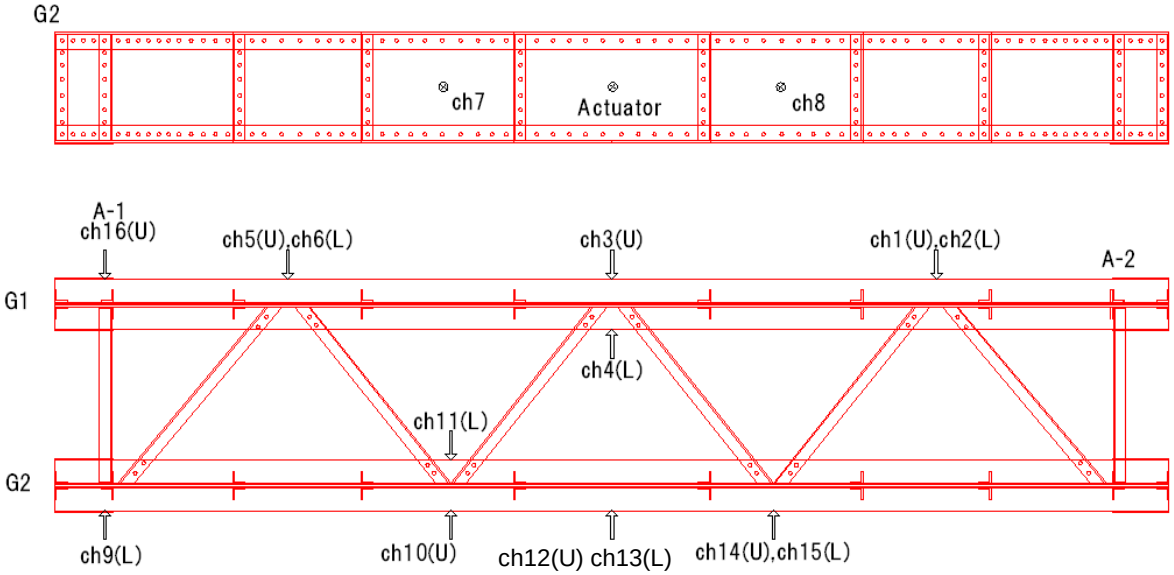


Fig. 14 Sensor locations on horizontal plane

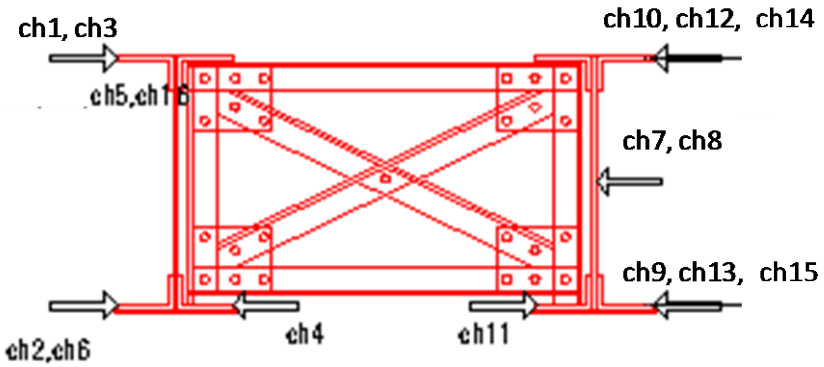


Fig. 15 Sensor locations on cross sectional plane

Sensors are 16 accelerometers and located in horizontal direction as shown in Fig. 14 and Fig. 15. One actuator is set on the center of web plate at the middle of G1 girder between ch7 and ch8 in horizontal direction as shown in Fig. 15 and Photo 5. Then out of plane vibration of web plate is generated by this setup.

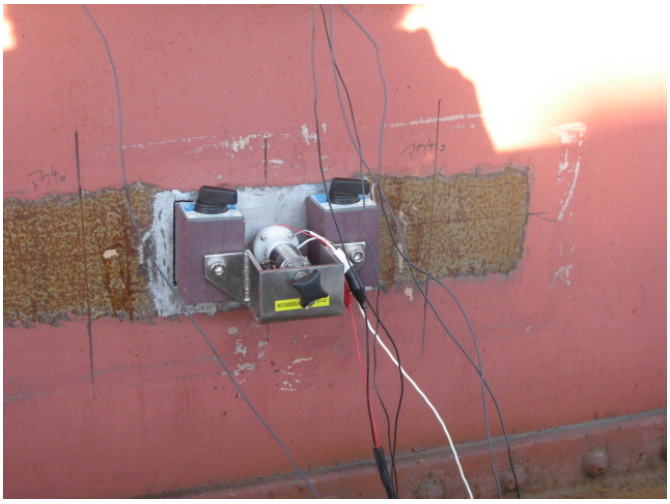


Photo 5 Actuator set on center of web plate in the middle of G1 girder

Excitation

As shown in previous article a piezoelectric actuator is permanently fixed at the center point of the web plate of G1 girder as shown in Photo 5. Excitation force amplitude is set as 30 (N). Excitation force direction is horizontal and excitation wave form is sine sweep wave. Excitation frequency is around 1~750Hz. Excitation time is 20 sec and recording time is 25 sec. Sampling rate is set as 10000 Hz.
Same as before the excitation frequency is gradually increasing over time until it reaches its maximum value after the designated time (20 seconds in this case). And the excitation wave form is a sweep function.

5.3 DAMMAGE DETAILS OF CRACK

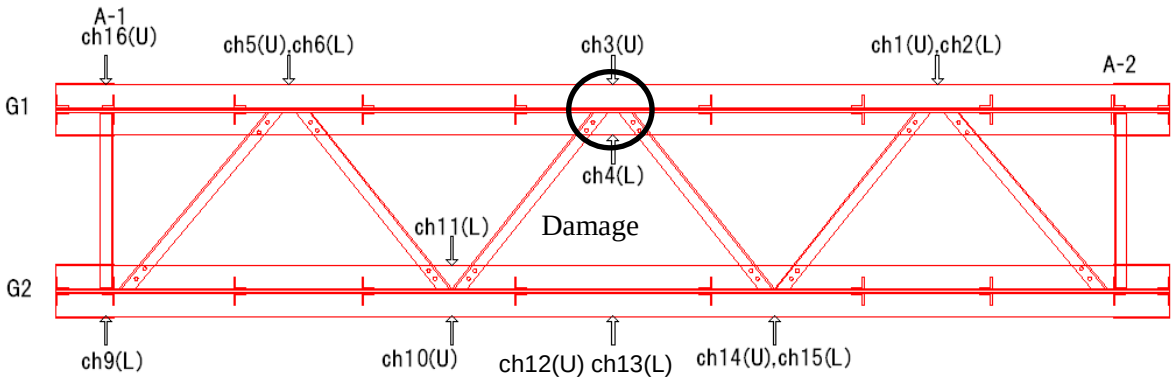


Fig. 16 Location of artificial damage crack on girder G1

Artificial crack is made at the center of girder G1 of the outer lower flange plate as shown in Fig. 16. Location is the opposite side of accelerometer **ch4** and beneath **ch 3** in the lower flange plate of girder G1. Length of artificial damage crack is chosen as one fourth of the flange width for Case **D1** as shown in Photo 6 and one half of flange width for Case **D2** as shown in Photo 7. Crack is clearly opened without contact between their side surfaces. Environmental temperature around the bridge and meassument equipment is measured as 23 ℃ during the measurement.

Photo 6 Artificial Crack of Case D1 by one fourth of flange width

Photo 7 Artificial Crack of Case D2 by one half of flange width

5.4 DAMAGE IDENTIFICATION IN CASE OF SHORT SPAN BRIDGE

In the case of Damage **D1** and **D2** Eq. (3) is slightly changed as shown in Eq. (13). Local excitation by actuator is set on the girder G2 which is opposite location from girder G1 where the crack damage is located. The reason why the artificial crack is made on the opposite side girder G1 is that how the detection by this damage identification algorithm works even in the case when distance between excitation and damage are not close. The effect of small crack in the vibration data is small and analysis is focused on how analytical method can detect the small damage of crack. In order to distinguish the effect of small crack in the analysis, the denominator of Eq. (3) is chosen as a smaller

value from the PSD value over each frequency and channel of both before and after damage as shown in Eq. (13).

$$D = \frac{|G_i(f) - G^*{}_i(f)|}{1 + (G_i(f), G^*{}_i(f))_{\min}} \quad (13)$$

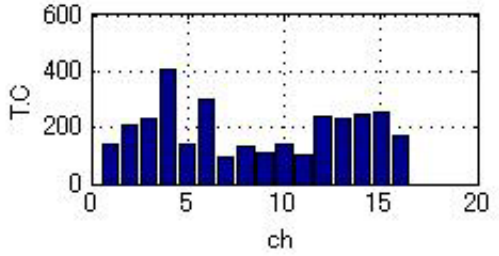


Fig. 17 Total change (TC) in Case of D1

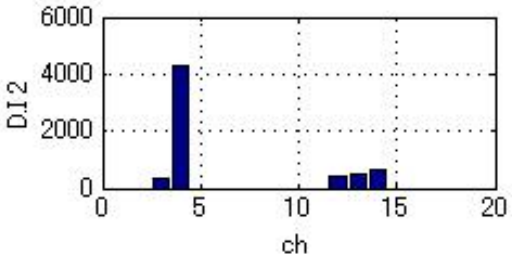


Fig. 18 Damage Indicator D I 2 in Case of D1

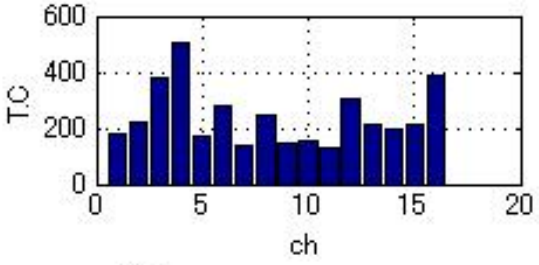


Fig. 19 Total change (TC) in Case of D2

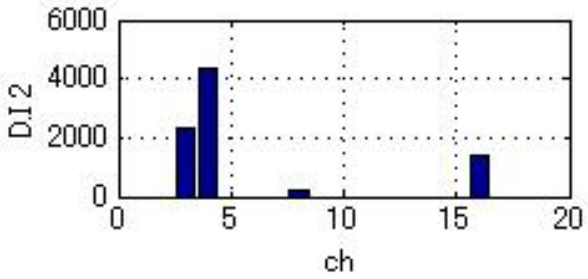


Fig. 20 Damage Indicator D I 2 in Case of D2

The total change (**TC**) calculated by Eq. (3) in case of **D1** is shown in Fig. 17 and in the case of **D2** the total change (**TC**) is shown in Fig. 19.
The damage indicators **D I 2** in cases of **D1** and **D2** are shown in Fig. 18 and Fig. 20. In Fig.18 by using **D I 2** the location of the crack is clearly identified as on channel 3 and channel 4 . By comparing the results on both cases of **D1** and **D2** the influence of bottom crack in vibration data is more distributed over to the neighbor portion of girders due to the size of crack.

6. OVERVIEW AND REVIEW OF METHOD

It is clearly necessary for medium and short span bridges (MSSB) to detect the small change of structural characteristics and to do their structural health monitoring. Ultrasonic testing and X-ray inspection are used mainly in fabrication factory for quality control of construction, so that it is normally difficult for in-situ inspection for MSSB. By this reason vibration-based monitoring method is used and well adapted for the diagnosis of structural condition and detection of damage of MSSB. Source of vibration for vibration-based monitoring are ambient vibration, environmental vibration and traffic-induced vibration, but vibration amplitude is small in case of MSSB and then this condition decrease the accuracy of damage evaluation.

By using Local Excitation Method (LEM) the repeating of excitation is stable and reliable. And by moving excitation point in the area where the possibility of damage existence is high, and then by using array sensing of accelerometer, higher accuracy of damage detection for MSSB is possible.

The accuracy of detection of location and size of small damage for MSSB by using LEM and its applicability, are shown in this paper. By the periodical in-situ inspection over years on real MSSB, the proposed method will be verified to be able to detect the damage by high accuracy for real application.

The change of boundary condition and temperature difference between before and after damage progress are big elements which decrease the detection accuracy of damage. In the case of LEM the propagation and reflection of local vibration through the detail structural component are complicated and the measured data include those effect. So that multiple moving excitation point in LEM can be used and taking the average value over measured results gives high reliability.
And numerical simulation for detailed structure under implosive point excitation is also a good additional information to evaluate the results of damage effect. The need to solve these problem is the future work to apply LEM for real MSSB.

7. FINAL REMARKS

By array sensing through local excitation using piezoelectric actuator we can get the local damage information of MSSB properly. It is important to detect the smaller damage as soon as possible when it is occured. In the structural health monitoring of bridge the initial structural condition is measured and after damage progressed the damaged condition is again measured to compare with initial condition. Just after new bridge construction the masurement on undamaged condition for the structure is desirable as the data of initial condition. But the initial condition, generally, is measured when the damage is found and then the damage progress is monitored by the succesive mesurement on damaged structure.

Distribution of array sensors in the measurement is important and location of excitation effects also on the accuracy of the results. Normally damage is hidden behind the details of the structure, so that the location of excitation should be moved in the area of suspected damage area and their average of the result of damage indicator over the moved locations of excitation gives more accurate information of the damage.

This research also aims at establishment of experimental environment to enable the verification of applicability and the effective evaluation of the structural health monitoring (SHM) technology that uses the high performance sensor system on real bridges. The primary goal of the this project is the development of test beds that meet wider needs for SHM experiments. Several steel bridges of an abolished railway in Hokkaido, Japan are available in this test beds as shown in Photo 8 and Photo 9. The test beds can be utilized by not only researchers of this project but also other researchers. The final goal of the project is the international collaboration by means of cooperative utilization of the test beds. This research was supported by JST(Japan Science and Technology) Agency as international collaboration during 2007 to 2010. Authors acknowledged to their support on this project.

Photo 8 Collected short span bridges

Photo 9 Collected medium span bridges

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