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Bridge safety monitoring based-*GPS* technique: case study Zhujiang Huangpu Bridge

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Abstract. *GPS* has become an established technique in structural health monitoring. This paper presents the application of an on-line *GPS RTK* system on the Zhujiang Huangpu Bridge (China) for monitoring bridge deck and towers movements. In this study, both the form and functions of movements of the deck and towers of the bridge under affecting loads were monitored in lateral, longitudinal and vertical directions. Such movements were described in time and frequency domains by determining the trend, torsion, periodical of the series using probability density function (*PDF*). The results of the time series *GPS* data are practical and useful to bridge health monitoring.

Keywords: *GPS*; monitoring; bridge; *SHM*; *PDF*

1. Introduction

The field of structural health monitoring (*SHM*) is an integrated paradigm of networked sensing and actuation, data interrogation, and statistical assessment that treats structural health assessments in a systematic way (Mascarenas *et al*. 2009). An appropriate sensor network is always required in observing the structural system behavior in such a way that suitable signal processing and damage-sensitive feature extraction on the measured data can be performed efficiently (Mascarenas *et al*. 2009).

Global Positioning System (*GPS*) has been extensively used in recent years for many *SHM* applications (Meng *et al*. 2007, Kaloop *et al*. 2009a, Ramin *et al*. 2009). While *GPS* can provide continuous, all-weather, automated high accuracy measurements in the presence of strong satellite geometry, its deficiencies compared to traditional techniques for such applications have been investigated by (Dodson *et al*. 2003).

In addition, the *GPS* technology can measure directly the position coordinates, and nowadays relative displacements can be measured at rates of 10 Hz and higher (Ogaja *et al*. 2001). This provides a great opportunity to monitor, in real-time, the displacement or deflection behavior of engineering structures under different loading conditions, through automated 'change detection' and alarm notification procedures. The measurement principle works worldwide, continuously and under all meteorological conditions, and therefore holds promise as a way to monitor the movement of structures (Kaloop *et al*. 2011).

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The stochastic parameters with the structures movements can be regarded as stochastic structure. For the applied loads (traffic, wind, seismic,…etc) response analysis of stochastic structures, existing developed methods include the random simulation method, the random perturbation method and the orthogonal polynomial expansion method, etc., have been extensively studied and a variety of literatures are available (Li *et al*. 2006). These analytical methods can only obtain the statistical characteristics such as the mean and the standard deviation of the response. In resent years, the probability density evaluation (*PDE*) method for stochastic structures has been developed, which can get the distribution of probability density and it is evolvement with the time, the random response and reliability evaluation of nonlinear structures could be easily implemented through the numerical solution (Li 1996, Li *et al*. 2003, Liu *et al*. 2008).

A class of probability density evolution equation for dynamic responses of stochastic structures is derived according to the principle of preservation of probability. The equation is reduced to a one-dimensional *PDF* that is convenience for numerical solving (Li and Chen 2004). Also, in the *PDF* method, the dynamic response of non-linear stochastic structures is firstly expressed in a formal solution, which is a function of the random parameters. In this sense, the dynamic responses are mutually uncoupled. A state equation is then constructed in the augmented state space. Based on the principle of preservation of probability, a one-dimensional partial differential equation in terms of the joint probability density function is set up (Li *et al*. 2006).

The aims of this paper are: (1) To examine the *RTK-DGPS* technique with frequency 1 Hz in bridge deck and towers movements monitoring, (2) To study the Zhujiang Huangpu Bridge safety under applied loads, (3) To examine *PDF* technique in bridge deformation study.

2. Bridge description and *GPS* information

The Zhujiang Huangpu Bridge is composed of a 705 m-long cable-stayed bridge and an 1108 m-long cable-suspended bridge. The width of deck and the height of towers for the Bridge are 34.5 m and 195.476 m, respectively. There were 8 *GPS* installed on the two bridges before August 15, 2009, including 7 rover stations and 1 base station. But after August 15, 2009, another 6 rover stations were installed on the two bridges and then there were 13 rover stations and 1 base station as shown in Fig. 1. The reference station refreshes the *RTK* correction messages for the *GPS* monitoring stations with a frequency of 1 Hz via the optical fiber communication system.

The outputs from *GPS* monitoring stations include monitoring point number and *RTK GPS* three-dimensional coordinates, *GPS* time, satellite status data, *GPS* receiver status data, and so on, and the raw *GPS* outputs, if required. The resonant frequency of a bridge is a very important parameter for bridge structure and safety analysis. To use the *GPS* data to estimate the bridge's spectrum and characteristic frequencies, the sampling rate of *GPS* monitoring stations, f_s, and the bridge resonance frequency, f_c, must satisfy the following criterion

$$f_s \geq 2f_c \tag{1}$$

The sampling ratio, f_s, is limited by the signal transmission ratio of *GPS*. The collected *GPS* data were respectively resolved from the rover stations and the base station. They represent the relative displacement changes (mm) to their own base points. A local Bridge Coordinate System (*BCS*) was established to be used in the analysis and evaluation of the observation data. The *X* data represent

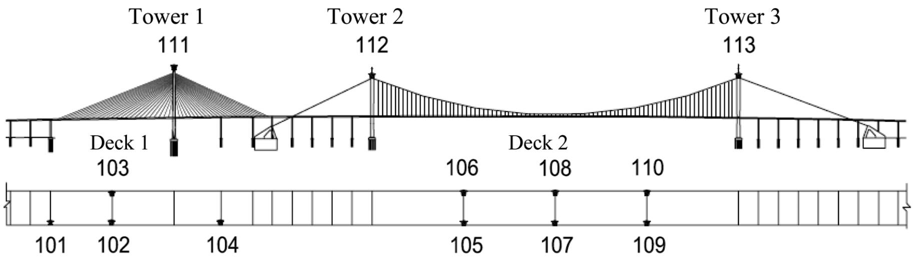


Fig. 1 Elevation and location of *GPS* receivers on Zhujiang Huangpu Bridge

the relative displacement changes along the longitudinal direction of the bridge, the *Y* data represent the relative displacement changes along the transverse direction of the bridge and the *Z* data represent the relative displacement changes along the altitude direction of the bridge.

3. Results analysis and applications

Before using the *GPS* data for analysis, the collected *GPS* data, which were received in a *WGS*84 coordinate system, were converted to three-dimensional coordinates relative to the bridge by coordinate transformation, for more information on coordinate transformation; reader may refer to Kaloop *et al*. (2009b). As errors, or “noises,” are unavoidable in *GPS* measurement for various reasons, the *GPS* data were then reconstructed smoothed by wavelet transform to remove any obvious errors as shown Fig. 2

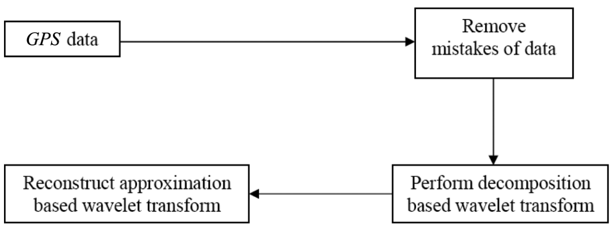
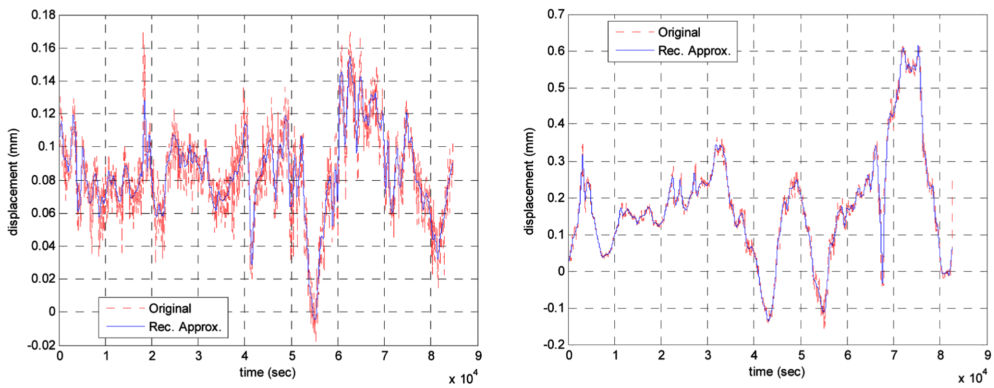


Fig. 2 Flowchart processing the *GPS* data collection



*Z*102 *X*111

Fig. 3 Signal processing of *GPS* signals for *Z* direction for point 102 and *X* direction for point 111

Collected time series of *GPS* data were carried out during September 24^{th} and 26^{th}, 2009. In the next subsection, the results of measurements those two days are discussed in details.

From Fig. 3, it can be seen that the signal errors of the deck stations is higher than that of the towers station. This indicates that the main source of *GPS* errors is Multipath error. It shows that the accuracy of the reconstruct approximation (*RA*) signals was increased by about 28% than that of the original signals. The *RA* signals can be used to analyze the movements of the bridge. In addition, the results show that wavelet methodology can separate different types of displacement information effectively. The similarity and dissimilarity between different types of bridges and different types of *GPS* can be clearly detected after separation (Wu 2008).

3.1 Movement of the towers

The two days observed data, according Fig. 4, from the three *GPS* rover antenna. The *RA* signals for the three towers of the bridge study are presented. Since the three towers (object points 111, 112 and 113) were observed at the same time intervals, it was possible to compare the movements of these towers. The evaluation of observations was performed with respect to *BCS*. Lateral movements of the tower 111 (Y111) show an increase starting from the initiation of the measurements until 3.5×10^4 (Sec.) and later on, a decrease is seen and then an increase again to decrease (a change of 0.7 mm) (Fig. 4). Lateral movements of the tower 113 (Y113) demonstrate similar movements (a change of 0.65 mm) (Fig. 4). Lateral movements of the tower 111 (Y111) show an approximate constant amplitude till 3.5×10^4 (Sec.) and later on, a decrease is seen and then increased (a change of 0.25 mm).The increase in the movement becomes significant at 7×10^4 (Sec.).

Similarly, it can be show that the lateral movements of the towers 111 and 113 change in a similar manner. The longitudinal movements of the towers 111 and 113 are almost similar to the lateral movements (Fig. 4). Also, the longitudinal movement of the tower 112 (X112) (a change of 0.17 mm) is observed in the reverse direction to that of the other towers. In addition, the time series plots in Fig. 4 show an overlay comparison between the results from the two days observations, and their differenced residuals. There is a strong correlation between the two measurements as evidenced by the correlation coefficient (R111 and R113=0.97 whereas R112=0.54). Changes in tower movements along the (*Z*) direction were insignificant and uncorrelated to variations in applied loads. From the comparison between observations of the presented days, it can be seen that the movements of towers in two directions were insignificant to variations of applied loads.

3.2 Movement of the deck

Studding the movements of the deck in both Lateral (*Y*) and longitudinal (*X*) directions show they occur almost in the same time interval directions (Figs. 5 and 6). The vertical (*Z*) movement occurs in the reverse direction to the lateral and longitudinal movements. As shown in Fig. 5 the maximum change for deck 1 is approximate 0.15 mm. Object points 107 and 108 are located in the middle of the bridge deck 2 and this is where significant changes in the vertical (*Z*) direction are expected. The observed measurement was 1.0 mm and 3.00 mm, respectively (Fig. 6). The maximum deformation changes in the longitudinal (*X*) and lateral (*Y*) directions were observed to be almost the same. Referring to Fig. 6, it is observed that the movements of the mid-point of bridge deck 2 in the lateral and vertical directions are not similar. While lateral movement is in the positive direction, and the vertical movement goes to the negative direction. The longitudinal movement, also, has

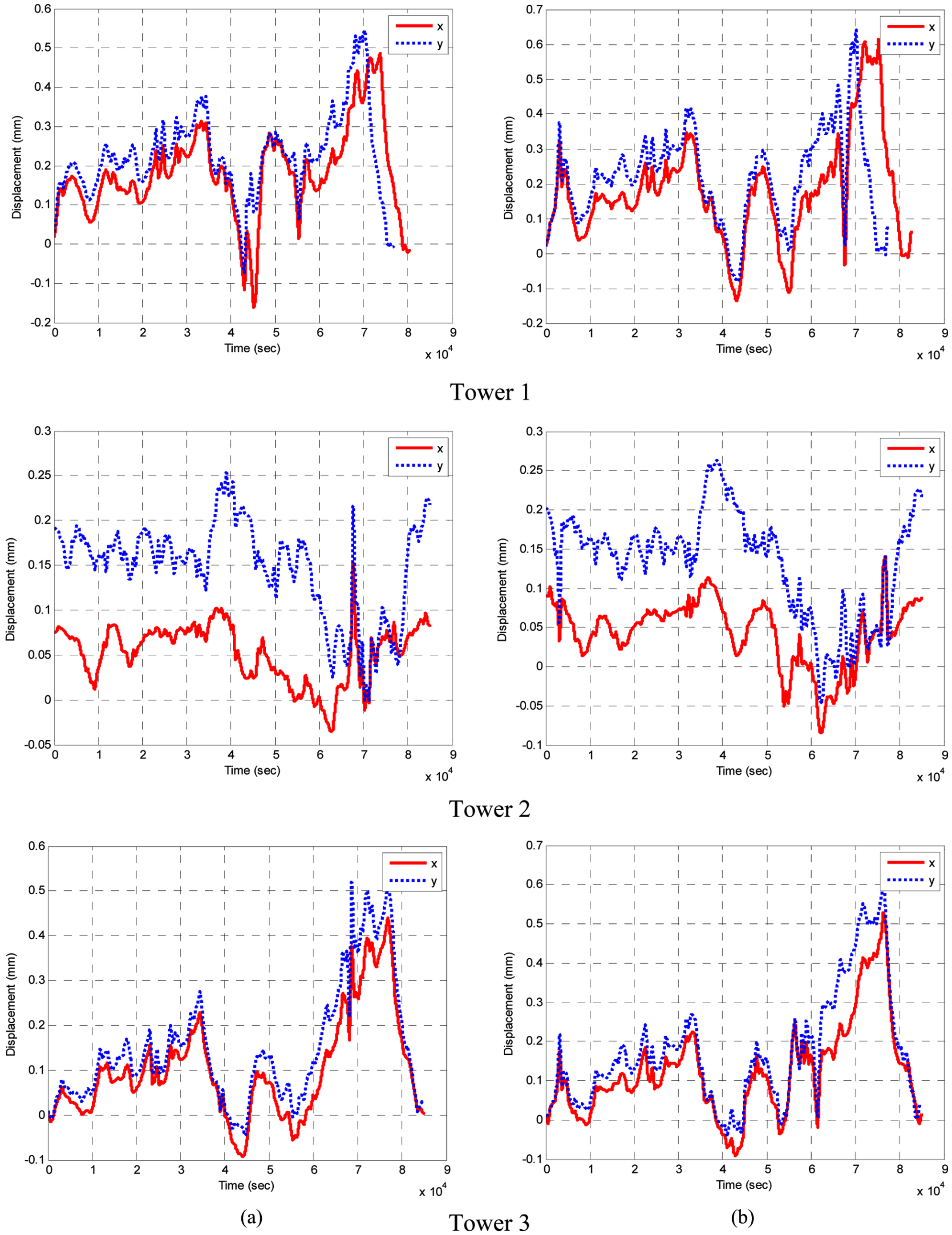
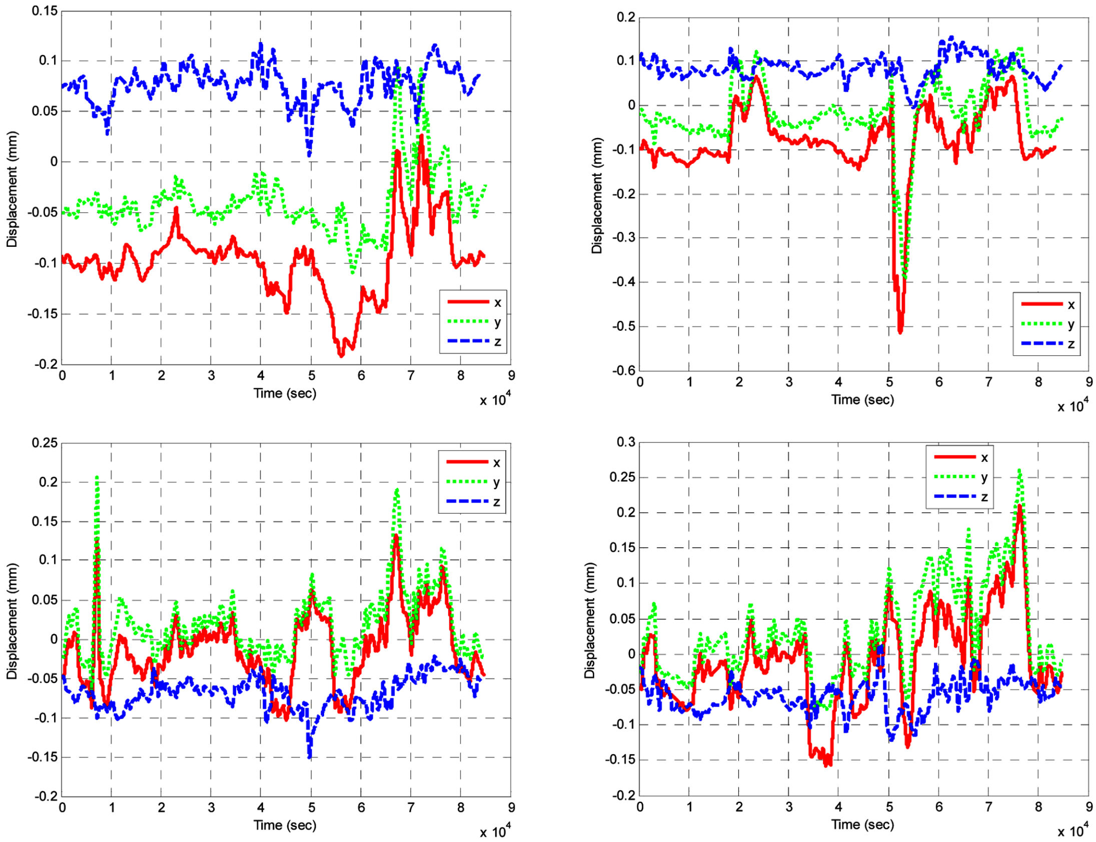


Fig. 4 The displacement of Towers for days (a) September, 24th and (b) September, 26th

similarities with the lateral and vertical movements. Yet, movements in the vertical direction are found to have more and sudden direction changes when compared with movements in the lateral and longitudinal directions. The lateral and longitudinal movements show stability within the observed

(a)

(b)

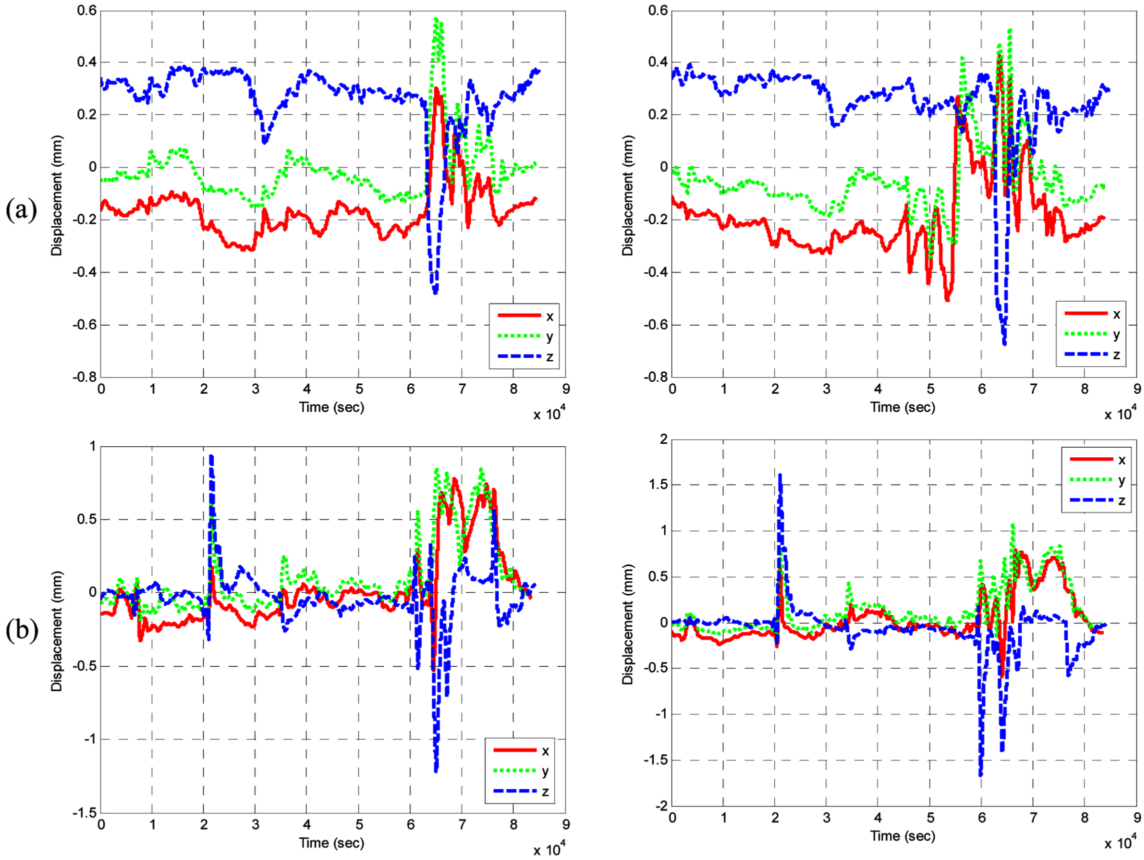


September, 24th September, 26th

Fig. 5 The displacement of Deck 1 for days (a) point 102 and (b) point 103

time intervals.

Changes in the lateral and longitudinal directions were found to be smaller and a maximum change occurred in the vertical direction due to traffic load. In Fig. 5, it can easily be seen that movements of the object points 102 and 103 as shown in Fig. 1 of the bridge deck 1 in the longitudinal (*X*) and lateral (*Y*) directions demonstrate a great similarity. Deck movement directions are the same and they changed more regularly with the time series. When the graphs are examined for the whole period of observation, it can be stated that changes in the longitudinal and lateral directions demonstrate a great consistency. The maximum change in the vertical direction of the object points 102 and 103 are 0.15 mm and 0.20 mm, respectively. Furthermore, the maximum changes in the longitudinal and lateral directions of the same points are harmonious with maximum changes of object points 107 and 108. From the previous studies Kuhlman (2001) and Kaloop *et al*. (2009b), it could be deduced that only temperature changes and traffic load had effects on the movement of the bridge deck. But due to the wind speed being low and constant in the course of observation, it was not possible to clarify its effects on the bridge graphically. In fact, wind speed is the most important factor that affects the bridge deck. Also, the time series plots in Figs. 5 and 6 show an overlay comparison between the results from the two days observations. There is a strong correlation between the two measurements as further evidenced by the correlation coefficient calculates.



September, 24th September, 26th

Fig. 6 The displacement of Deck 2 for days (a) point 107 and (b) point 108

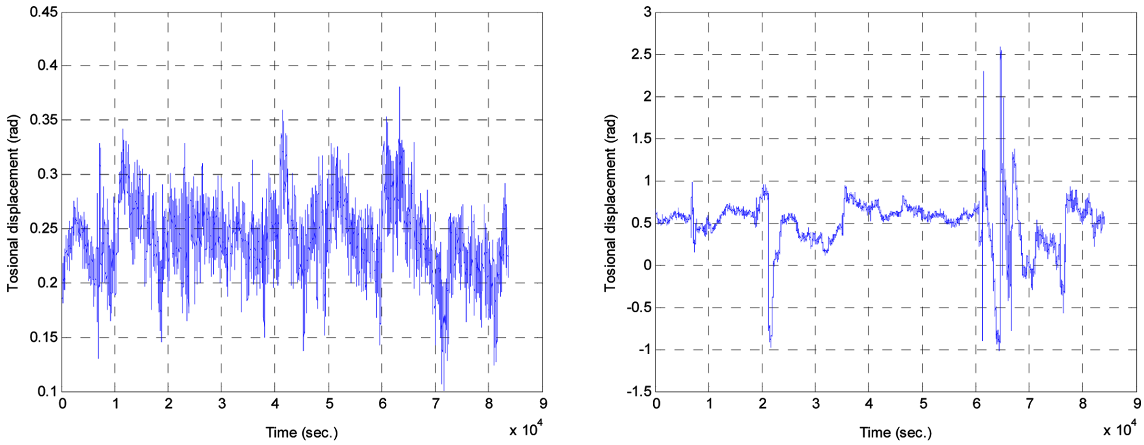
3.3 Torsional displacement analysis

The torsional displacement has been monitored and calculated with the following equation.

$$\theta = \arcsin\left(\frac{z-\bar{z}}{B}\right)\cdot\frac{180}{\pi} \tag{2}$$

where z and $\bar{z}$ are the vertical displacements at symmetrical stations (e.g., Stations 102 and 103 in Fig. 1), and B is the distance between the two stations (34.5 m). Fig. 7 shows the torsion displacement that is based on the data from Stations 102 and 103 for deck 1 and 107 and 108 for deck 2 (Fig. 1). The torsion displacement was measured to define completely the bridge's movement under wind and traffic load (Park *et al.* 2008).

From the Fig. 7, it can be seen that the range of torsion displacement for deck 1 and 2 is 0.25 and 3.5 radian; also, the mean value is 0.25 and 0.50 radian, respectively. These indicate that the length of deck is affected on the torsion values. Also, it reveals that the torsion displacement calculation of the bridge due to wind and traffic loads was ineffective on the bridge torsion determination.



(a) (b)

Fig. 7 Torsion displacement of bridge during applied loads (a) deck 1 and (b) deck 2

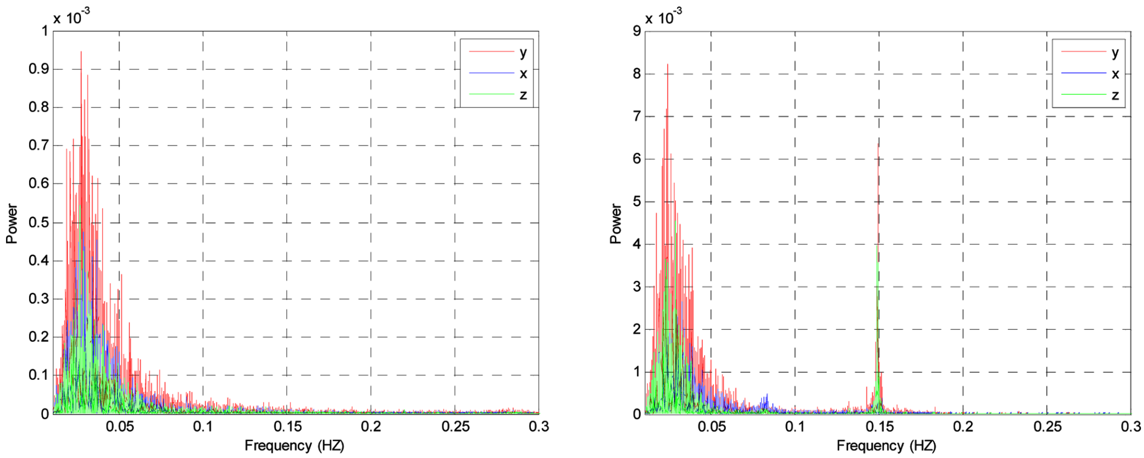
3.4 Bridge frequencies analysis

The displacements measured at the measuring stations can be used to calculate the bridge's spectrum using fast Fourier transform with the following equations

$$X(k) = \sum_{n=0}^{N-1} x(n)e^{-j(2\pi/N)nk} \qquad k = 0,1,...., N-1 \tag{3}$$

$$x(n) = \frac{1}{N}\sum_{k=0}^{N-1} x(k)e^{j(2\pi/N)nk} \qquad k = 0,1,...., N-1 \tag{4}$$

A power spectrum of a structure shows its main characteristic frequencies. The horizontal and vertical power spectrum of Zhujiang Huangpu Bridge is shown in Figs. 8 and 9. From which Figures, we can



(a) (b)

Fig. 8 The frequency of Deck bridge (a) deck 1 and (b) deck 2

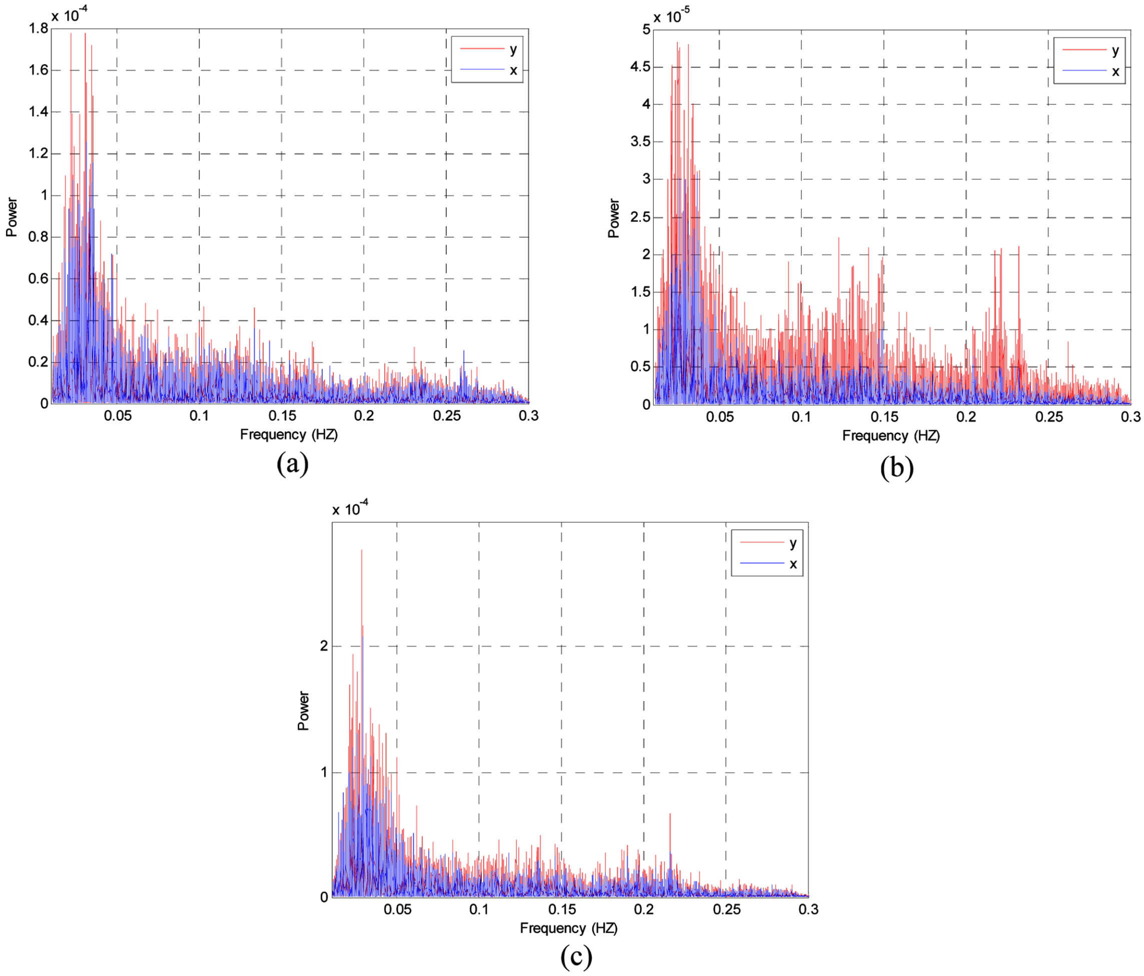


Fig. 9 The frequency of bridge towers (a) tower 1, (b) tower 2 and (c) tower 3

find that the first, second and third deck 2 oscillation frequencies, which are 0.027, 0.08 and 0.148 Hz, respectively. Characteristic frequencies are important parameters for bridge safety control. The characteristic frequencies of a structure are often be calculated using finite elements. However, it can also be done by using *GPS* data from the monitoring stations.

After the elimination of the linear trend component in the time series for movements of the bridge deck and towers in the longitudinal (*X*), lateral (*Y*) and vertical (*Z*) directions, frequency in the series were determined. The frequency calculate consists of the high frequency changes which occur over long-term movements as a result of high frequency, random or instantaneously changing loads affecting the bridge. The frequency components of the bridge deck and towers were calculated at intervals 0-0.3 Hz. The first-order differentiation procedure was applied to the series. Eq. (3) demonstrates the high-pass filtering property to determine high frequencies present in the series. By the use of such filters in the time series, noise components in the series can be reduced to one degree. To reduce the effects of spectral leakage, the series was multiplied by the Hanning Window Function (Kaloop and Li 2011). Later, transformation of the series from time domain to frequency domain was performed by *FFT* in Eq. (3) and power spectrums were calculated with Eq. (4). The calculated high frequency components of the Zhujiang Huangpu Bridge deck and towers are given in Table 1, Figs. 8 and 9.

From Table 1 and Fig. 9, it can be noticed that maximum powered high frequencies for movements of towers in lateral and longitudinal directions are the same; however, frequencies in lateral directions have higher powers. Besides, it can be seen that high frequency reactions of towers 2 and 3 in lateral direction and longitudinal direction are more intense in the 0.01-0.25 Hz interval, and high frequency reactions of tower 1 in the longitudinal direction are more intense in 0.01-0.15 Hz interval.

Table 1 and Fig. 8; on the other hand, shows that except for changes in the longitudinal, lateral and vertical directions of deck 1 and 2, frequencies of the bridge with maximum powers are nearly the same for all directional changes. Also, the frequencies for deck 1 and 2 in lateral directions have higher powers. High frequency reactions of the bridge deck 1 in the three directions were determined to be more intense in the 0-0.1 Hz interval and those for deck 2 in the three directions were determined to be more intense in the 0-0.2 Hz interval.

Furthermore, it was detected that the first mode frequency components of the movements of the Zhujiang Huangpu Bridge deck 1 and 2 in the lateral directions were found 0.027 and 0.023 Hz, respectively, whereas in the longitudinal and vertical directions were found 0.028, 0.033, 0.026, 0.028 Hz, respectively. As a consequence, the calculation of natural frequency of a structure is independent of other loads, for example, wind, earthquake and vehicle load, affecting the structure. However, it was stated in referenced Chatfield (1996) and Brownjohn *et al*. (1988) that the mode types and frequencies of suspension bridges could be obtained from the observation of temporary vibrations occurring as a result of wind and traffic loads and the Humber and Bosphorus Bridges were given as examples. Frequencies having maximum powers of bridge deck 1 were found 0.027 Hz (first mode) and the frequency having maximum power of bridge deck 2 were found 0.148 Hz (third mode). In addition, the preceding results suggest that the *GPS RTK* technology (either independently or in conjunction with other modal analysis tools) can provide accurate data for inherent modal analysis of such large structures as suspension bridges.

Based on the recorded *GPS* data from September 15 to 26, 2009, the estimated bridge characteristic frequencies are shown in Table 1.

The 0-0.2 Hz range, noise such a Multipath is contributing to all three components (Li and Peng 2004). So, in the study case the rate of *GPS* observation can not used to detect the natural frequency of the bridge. But the frequency calculation can be expressed the safety for the movements of bridge.

Table 1 Zhujiang Huangpu Bridge first mode frequencies results with *GPS* data

Date	Deck 1			Deck 2			Tower 1		Tower 2		Tower 3	
	X	*Y*	*Z*	*X*	*Y*	*Z*	*X*	*Y*	*X*	*Y*	*X*	*Y*
15 Sept.	0.029	0.0266	0.0227	0.0322	0.0248	0.276	0.0257	0.0257	0.0302	0.0318	0.0255	0.0221
22 Sept.	0.034	0.0281	0.0314	0.0291	0.0292	0.0232	0.0266	0.0265	0.0335	0.0319	0.0247	0.0261
24 Sept.	0.0366	0.0318	0.0279	0.0286	0.0272	0.0272	0.0258	0.0258	0.0309	0.0222	0.0332	0.0278
26 Sept	0.0333	0.0275	0.0249	0.0326	0.0235	0.0287	0.031	0.030	0.0288	0.0258	0.0293	0.0286

3.5 Stochastic applied loads response of the bridge movements

The principal of probability conservation can be desired as, during a conservative probability transformation process, within the state space, the increment of probability within a unit volume equals to the inflow probability that gets across this unit (Martin 2007, Ai *et al*. 2004). The dynamic equation of the non-linear structure can be written as

$$M(\beta)\ddot{V}+C(\beta)\dot{V}+K(\beta,V) = -M(\beta)\ddot{V}_g \tag{5}$$

Where β is the random parameter vector which represents the physical character of a stochastic structure, and its point probability density function (*PDF*) is $P_\beta(x)$. M,C are stochastic mass and damping matrices respectively which include random parameter with the rank $(n * n)$, n is the dynamic freedom degree (*DFD*); $[V\ \dot{V}\ \ddot{V}]$ are the displacement, velocity and acceleration vectors. $K(\beta, V)$ is the non-linear restoring force vector; $\ddot{V}_g$ is the acceleration vector of input dynamic loads. If the ordinary differential of state vector can be expressed as

$$\dot{Y} = G(Y,t) \tag{6}$$

Where $Y= [y_1, y_2,\dots, y_n]^T$ is the output observation vector and $G=[g_1, g_2,\dots, g_n]^T$ specifies which *DFD* of the system are observed. If $P_Y(y, t)$ is supposed as the *PD* of $Y(t)$, based on the principal of probability conservation, the *PD* evaluation (*PDE*) equation can be expressed as follow

$$\frac{\partial}{\partial t}P_Y(y,t)+\sum_{j=1}^{n}\frac{\partial}{\partial y_j}[P_Y(y,t)g_j(y,t)] = 0 \tag{7}$$

Lead in a response vector $U = [V^T, \dot{V}^T]$ the dynamic equation is then changed into a format of state equation including random parameters

$$U = A(U,\beta,t) \tag{8}$$

$$A = \begin{cases} \ddot{V}_g \\ -M^{-1}C\dot{V}-M^{-1}K-\ddot{V}_g \end{cases} \tag{9}$$

If $\dot{U} = \frac{\partial}{\partial t}U(\beta,t) = G(\beta,t)$, based on the above *PDE* equation, the point *PDF* of U and β will satisfy the following equation

$$\frac{\partial}{\partial t}P_{U_\beta}(u,u_\beta,t)+\sum_{i=1}^{n}\frac{\partial}{\partial u_i}[P_{U_\beta}(u,u_\beta,t)g_i(u,u_\beta)t] = 0 \tag{10}$$

Resolving the above equation with initial boundary value the joint *PDF* $P_{U_\beta}(u,u_\beta,t)$ can be calculated. After the integral with u_β, the *PDF* X_1(t) can be solved as

$$P_{X_1}(x_1,t) = \int P_{X_1}(x_1,x_\beta,t)dx_\beta \tag{11}$$

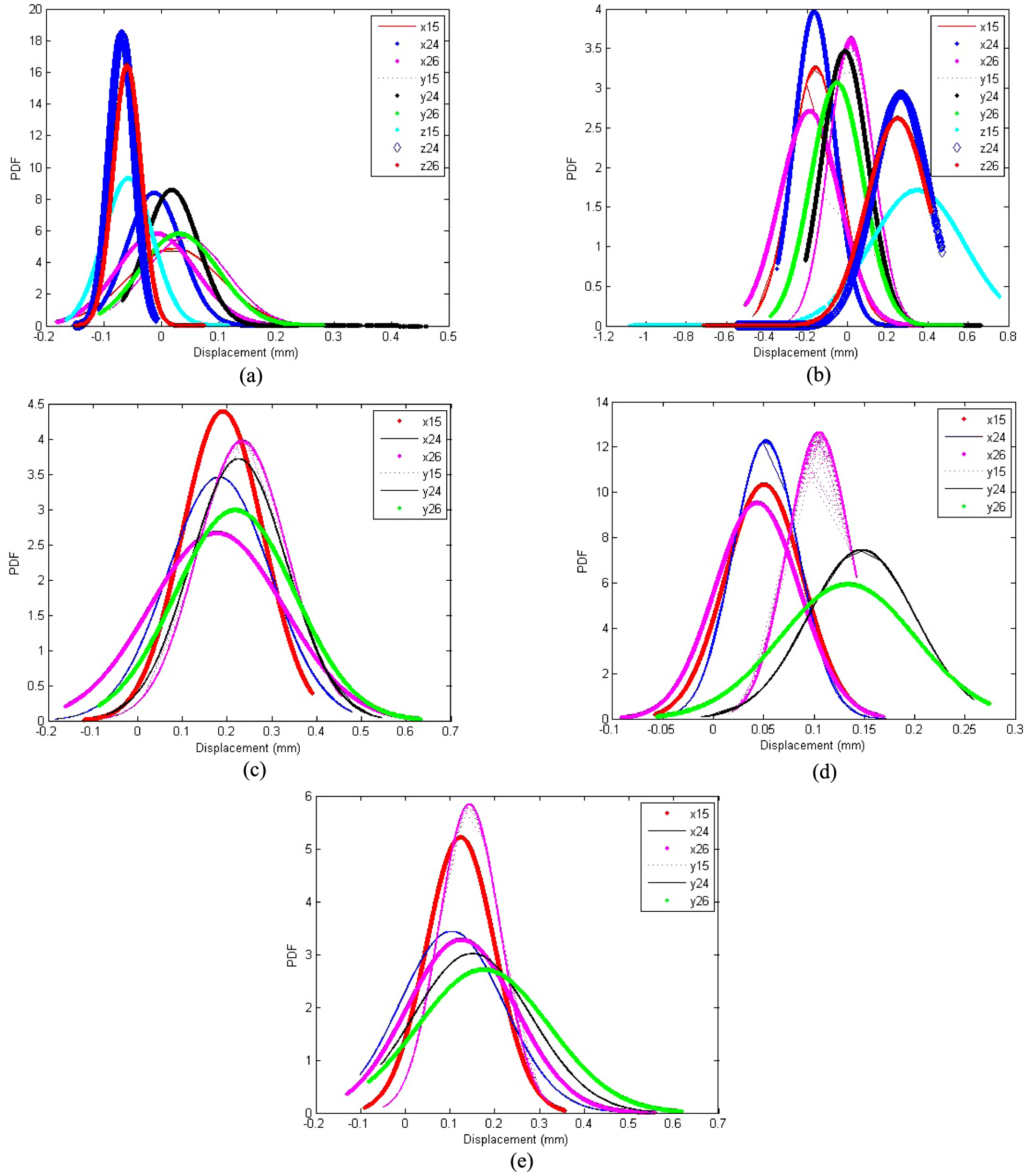


Fig. 10 the normal probability density movements of (a) deck 1, (b) deck 2, (c) tower 1, (d) tower 2 and (e) tower 3

Using *PDF* method, the *PD* of the stochastic point's deformation that evolves with time is calculated. Fig. 9 shows the evolving typical normal *PDF* curves at certain time instants, with the applied loads response.

In Fig. 10 shown is the probabilistic information of the bridge movements. Figs. 10(a)-(b) are the normal *PDF* (*NPDF*) of the probability density in the time intervals September 15, 24, 26 2009 for the Deck1 and Deck 2, respectively. From these figures, it is seen that the *NPDF* is quite regular for the *X*, *Y* and *Z*-directions. In general, it is irregular distribution type, if more than one or more peaks in the curves were shows for one curve, which implies that there exists obvious random bifurcation during the response process (Li and Chen 2004). In addition, the variance of the *PDF* against time is in a continuous way, in other words, the *PDF* is continuously differentiable with regard to time. So, the change of the probability density against time is obvious for the three dimensions of bridge decks. Also, it can be show that the curves were symmetrically for the *X*&*Z*-directions and *X*&*Y*-directions for Deck 1 and 2, respectively. In fact, the deviation from normal distribution means that the traditional second moments-based reliability analysis method may have significant error and should be refined to involve more precise probabilistic information (Li *et al.* 2006). The regularity means that the probability density changes greatly against time, of course mention that it is stationary. Also, it can be seen that the skew curve of *Y*-direction and *Z*-direction for the decks 1 and 2, respectively, were clearly. From the Fig., also shows that the movements of deck 2 were more stationary for the deck1 movements.

Figs. 10(c)-(e) are the *NPDF* of the probability density in the same time intervals for the Towers 1, 2 and 3, respectively. From these Figs., it is, also, seen that the *NPDF* is quite regular for the *X* and *Y*-directions. Also, it can be show that the curves were more symmetrically for tower1 and tower3 than for tower2. This revealed that the movements of tower 1 and 3 more than stationary of tower 2. in addition, it can be seen that the *GPS* signals for the (y15) contains errors, whereas the *PDF* curves in the three towers was contain some points outside the curve.

4. Conclusions

Monitoring the reactions of engineering structures under the effect of loads using appropriate observation devices is essential for maintaining safety and extending the lifetime of such structures. General information about the movements of a structure can be obtained using graphs to provide information about action and reaction quantities over time. This information will be even more useful when the data is drawn from very precise observations. However, in certain cases, although the quantities affecting the structures are well known, they cannot be measured, therefore, the structure can only be evaluated on the basis of the analysis performed for reaction quantities and in this situation time series analysis is a widely used tool.

In this study, both the form and functions of movements of deck and towers of the Zhujiang Huangpu Bridge, in lateral, longitudinal and vertical directions under affecting loads, were described in time and frequency domains by determining the trend, torsion, periodical of the series using the time series analysis and stochastic components response of the movements using probability density function (*PDF*).

From the time series graphs, it was concluded that the observed movements of the bridge deck and towers resulted from applied loads change. Using the symmetry of the bridge it was determined that the towers demonstrated similar movements under affecting loads, and movements of the bridge deck 2 in the vertical direction were much larger in size, when compared with longitudinal and lateral movements of the deck, whereas in contrast for the movements of the bridge deck 1. The lateral, longitudinal movements of the towers 1 and 3 changes were similar.

In the analysis of the frequency component, the maximum powered high frequencies of the decks and towers in the longitudinal and lateral directions are similar. The high frequencies of the towers 1 and 3 in the lateral direction and the longitudinal direction are more intense in the 0.021-0.034 Hz interval, and frequencies of the tower 2 in the same direction are more intense in the 0.024-0.23 Hz interval. High frequencies of the bridge deck 1 in the lateral, longitudinal and vertical directions were determined to be more intense in the 0.026-0.027 Hz interval. It was seen that except for changes in the lateral, longitudinal and vertical directions of mid deck 2 the frequencies (0.02-0.15 Hz interval) of the bridge deck with maximum powers are nearly the same for all directional changes. So, the proposed surveying techniques using *DGPS* and *RTK* (1 Hz) with long-time monitoring can provide valuable bridge deformation.

The instantaneous *PDF* and its evolution of responses of the stochastic structure are evaluated through combining deterministic dynamic analysis. Most importantly, the proposed method attains *PDF*, rather than second-order moments, of the responses. Some characteristics of the *PDF* are discussed. The *PDF* is varying against time, usually quite irregular with two or more peaks and far from those common used regular distribution types. This may need to be considered in dynamic reliability assessment. The basic idea of the proposed method is also suitable for nonlinear system where some special treatment different from the linear system is needed. The results show that the *PDF* of the non-linear dynamic responses evolve against time and are usually regular curves, near from normal distribution and other well-known distribution types. Finally, the bridge is safe under affect loads.

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