SHM-based Correlation Study of Trainload-induced Response in Tsing Ma Bridge

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Abstract
Structural Health Monitoring (SHM) technique has gained increasing attention in the areas of mechanical, aerospace and civil engineering over the past decades. Reliability analysis based on real-time SHM data can provide more timely and accurate assessment of present and future conditions of structures. Many researchers studied the reliability of bridges based on the field measurement data. However, most of the past researches focused on the reliability of certain component of bridges, such as the main cables, the suspenders, the stiffening girders and so on. Few literatures could be found on the reliability of entire bridge system. One of the obstacles is the correlation of stochastic responses of different components which is required for the computation of system reliability is usually missing. Simple assumptions of fully-correlated or uncorrelated cannot reflect the reality of bridges and may considerably compromise the accuracy of the analysis results. In light of this research need, this paper presents a correlation study of train-induced strain responses in a cable suspension bridge based on a comprehensive SHM system installed on the Tsing Ma Bridge in Hong Kong. Eight pairs of components, including four pairs of diagonal members and four pairs of bottom chords of longitudinal truss in two different sections were chosen in the correlation study. The correlations between peak strain responses of members in the same section, and members of different sections are discussed in details. Different correlation levels have been observed. The results of this study would shed light on the reliability analysis of the entire bridge system in future.

Key words: Correlation analysis, Cable stayed bridge, SHM, Strain

1. Introduction

With the rapid development of the economy and technology, many long span suspension bridges have been designed and constructed throughout the world in recent years, such as the Akashi Kaikyo Bridge and Innoshima Bridge in Japan(1), the Great Belt Bridge Halsskov in Denmark(2), the RunYang Yangtze River Bridge in China(3) and so on. Some of them are Rail-cum-Road bridges, such as the Tsing Ma Bridge in Hong Kong(4), and the Tianxingzhou Yangtze River Bridge in Wuhan(5). Bridges are critical links of transportation and railway networks. Any damage or collapse of a bridge not only results in loss of property and human fatalities but also has severe effects on the regional economy(6). There is a growing need for researchers to assess the reliability of bridges. Unfortunately, because of the uncertainty and randomness in loadings, material properties, and deterioration models of structures and so on, it is very challenging for researchers to get accurate reliability of structures.

Structural Health Monitoring (SHM) technique has been gaining considerable attention in mechanical, aerospace and civil engineering over the past few decades. SHM can provide objective and accurate assessment of existing condition and prediction of future trends based on collected data. A long-term reliable SHM approach could be used to evaluate the condition of the structure in a timely and accurate manner, describe the time-dependent civil infrastructure performance, follow abnormal structural behavior and deterioration processes at the very early stage of structures and provide information useful for structural repair and retrofitting. In this regards, SHM technique can considerably improve the accuracy of conventional reliability techniques. Many researchers have used the SHM data to analyze the reliability of bridges and their components in the past decades(7), (8), (9). However, Most reliability-based analyses have focused on the reliability of the individual structural components, rather than the whole structural systems(6), (10). Few literatures could be found on the reliability analysis of the whole bridge(10), (11), (12), (13). One of the obstacles is the correlation of stochastic responses of different components which is required for the computation of system reliability is usually missing. Simple assumptions of fully-correlated or uncorrelated cannot reflect the reality of bridges and may considerably compromise the accuracy of the analysis results.

In light of this research need, this paper presents a correlation study of trainload-induced strain responses in a cable suspension bridge based on a comprehensive SHM system installed on the Tsing Ma Bridge in Hong Kong. Eight pairs of components, including four pairs of diagonal members and four pairs of bottom chords of longitudinal truss in two different sections were chosen in the correlation study. The correlations between peak strain responses of members in the same section, and members of different sections are discussed in details. Different correlation levels have been observed. The results show that there is a highly linear relationship between the strain peaks of bottom chords in the same section. The correlations of strain peaks of the diagonal members on the same side are significant, while the correlations of the strain peaks were poor if diagonal members are on different sides of the bridge even though they are in the same section. However, both the strain peaks of the diagonal members and the bottom chords in the longitudinal truss in two different sections have relatively weak correlation.

2. Tsing Ma Bridge

The Tsing Ma bridge is a cable suspension bridge carrying a railway inside the bridge deck which links Tsing Yi island on the east to Ma Wan island on the west over Ma Wan Channel. It is a critical transportation link between Hong Kong International Airport and Hong Kong's vibrant business center. The bridge has a main span of 1,377 m and an overall length of 2,160 m (as shown in Figure 1). It is a rail-cum-road bridge with double bridge decks. The upper deck is 41-m wide and 7.643-m high, and carries six lanes of highway traffic; the lower level contains two rail tracks and sheltered carriageways for maintenance access and as backup for traffic in case of severe typhoons striking Hong Kong (as shown in Figure 2). The east and west towers are 206-m high prestressed concrete towers. The two main cables with a diameter of 1.1m are accommodated by the four saddles located at the top of the bridge towers. The anchorages are gravity structures resting on the underlying rock(4). The bridge deck is a hybrid steel structure continuing between the two main anchorages. The deck is supported by totally 94 suspenders to allow a sufficiently large navigation channel. It should be noted that the Tsing Yi side span and the Ma Wan side span are not symmetrical with respect to the bridge midspan due to the highway layout requirement. The bridge deck is composed of a number of steel cross-frames which are connected together by two longitudinal steel trusses and the upper and lower stiffened steel plates (as shown in Figure 2). The cross-frames are 4.5 m apart along the longitudinal direction. Each longitudinal steel truss consists of upper chords, bottom chords and diagonal members. Figure 2 also shows bracing systems in horizontal planes of the upper and lower decks.

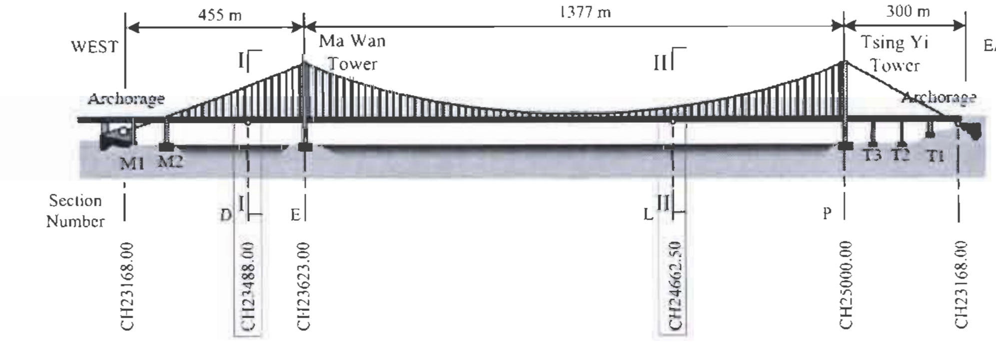

Fig. 1 Elevation of Tsing Ma bridge.

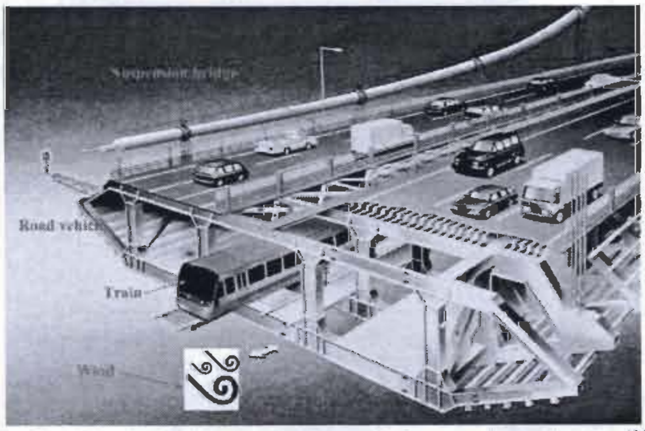

Fig. 2 Tsing Ma bridge under railway, highway and wind loading[14].

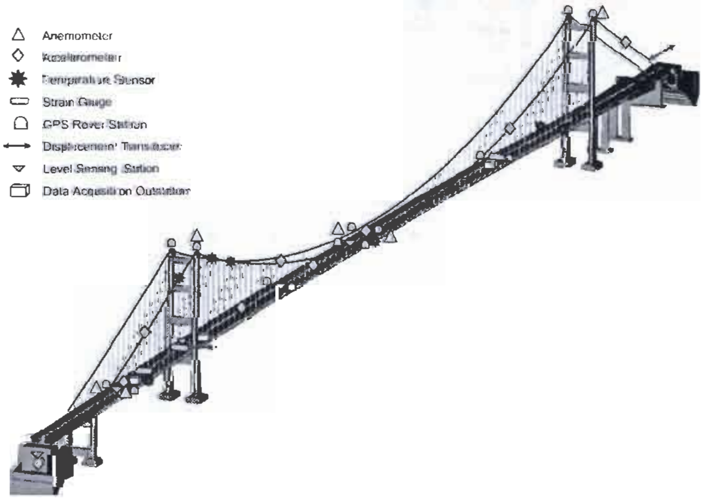

Fig. 3 Layout of the sensor and data acquisition systems[15].

3. Sensor Arrangement and Data Collection

3.1 SHM System on Tsing Ma bridge

A comprehensive SHM system, termed "Wind And Structural Health Monitoring System" (WASHMS), has been implemented on Tsing Ma Bridge by the Highways Department of the Hong Kong Special Administrative Region in 1997. The SHM system comprises 300 sensors of different types, including anemometers, temperature sensors, servo-type accelerometers, weigh-in-motion sensors, global positioning systems (GPS), level sensing stations, displacement transducers, strain gauges and CCTV video cameras. Such structural monitoring system has been continuously monitoring the loading conditions (e.g. wind, temperature and traffic loads) and bridge response since 1997.

Among 300 sensors, there are 110 dynamic strain gauges installed at different sections along the longitudinal direction. Four representative deck sections with strain gauges are located at the side span deck (Chainage (CH) 23488.00), Ma Wan Tower (CH 23623.00), three quarter point of the main span deck from the Ma Wan Tower side (CH 24662.50) and rail track (CH 24664.75), respectively. Figure 4 indicates the arrangement of strain gauges on truss chords located in the section CH 23488.00 and CH 24662.50. Three different types, namely, single, pair and rosette strain gauges, are installed. The details of tag numbers of strain gauges can be found in reference (14).

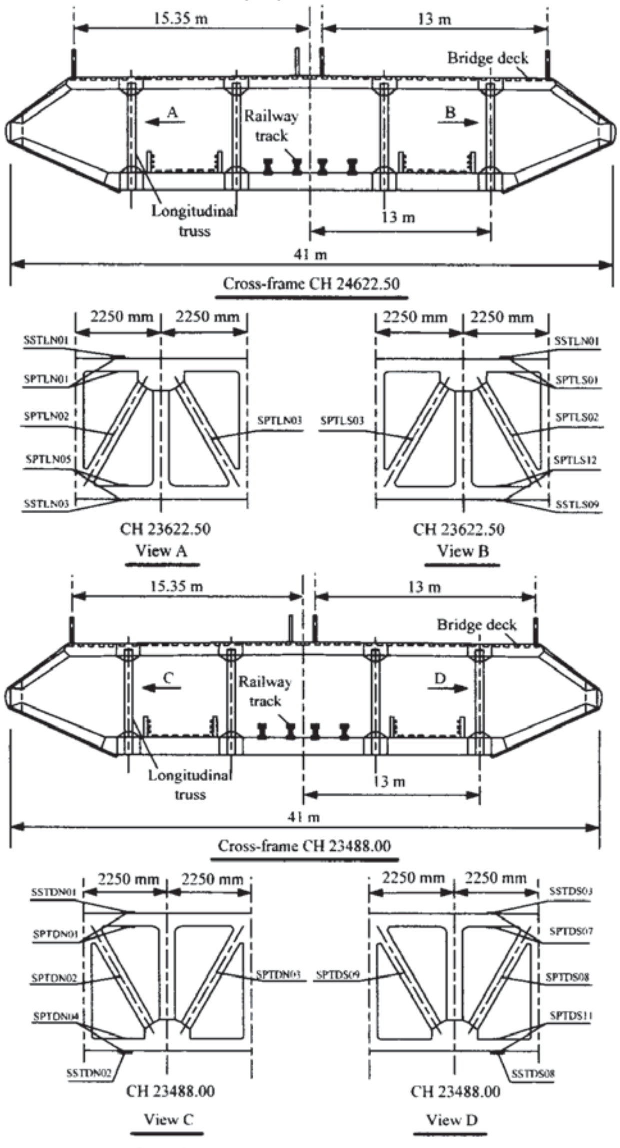

Fig. 4 Arrangement of strain gauges in two sections: CH23488.8 and CH24622.50.

3.2 Data Acquisition

Monitoring data of the loading and bridge responses are incessantly collected by the WASHMS. A huge amount of strain data have been collected over the past 14 years. The Tsing Ma bridge are normally subjected to combined action of multiple types of loadings, including railway loading, highway loading and wind loading, etc. However, the strain response of a certain component in the bridge is often dominated by one or two types of loadings. For example, the strain of chords in the longitudinal trusses is mainly induced by trainloads, while the strain of upper cross beams in cross-frames is affected by both trains and highway vehicles. The high complexity and randomness of highway traffic loads renders very low spatial correlations among those components greatly affected by highway traffic loads. Therefore, those components in longitudinal trusses are selected in this study, including four pairs of diagonal members and four pairs of bottom chords of longitudinal truss in the section CH 23488.00 and CH 24662.50. The locations of strain sensors are shown in Figure 4. The trainload-induced strain responses collected by the sensory system are analyzed. These components are not subjected to sever local stress concentration induced by train wheels. According to the records provided by MTR Corporation Hong Kong Limited, two kinds of trains run on the bridge. The daily trainload distribution is quite regular except for some extreme weather conditions. Only one-day strain response data (from 00:00:00 to 23:59:59 on 2 November 2005) are used in this study. However, the recorded data in a much longer period are also available, and more analyses will be carried out in future study.

4. Results and Discussion

4.1 Pre-processing of Data

The original strain data need to be pre-processed in order to obtain more reliable strain measurement, and it is composed of the following steps: (1) abnormal data due to a system interruption or strain gauge malfunction are removed; and (2) strain responses induced by temperature and dead load are removed. Since the frequency range of the strain responses due to temperature fluctuation is very low, the frequency less than 1/3600 Hz is filtered by wavelet analysis(14). Figure 5 shows the strain measurement of strain gauge SP-TDN-04 on 2 November 2005 before and after the filter. All the strain measurements are processed before the formal analyses.

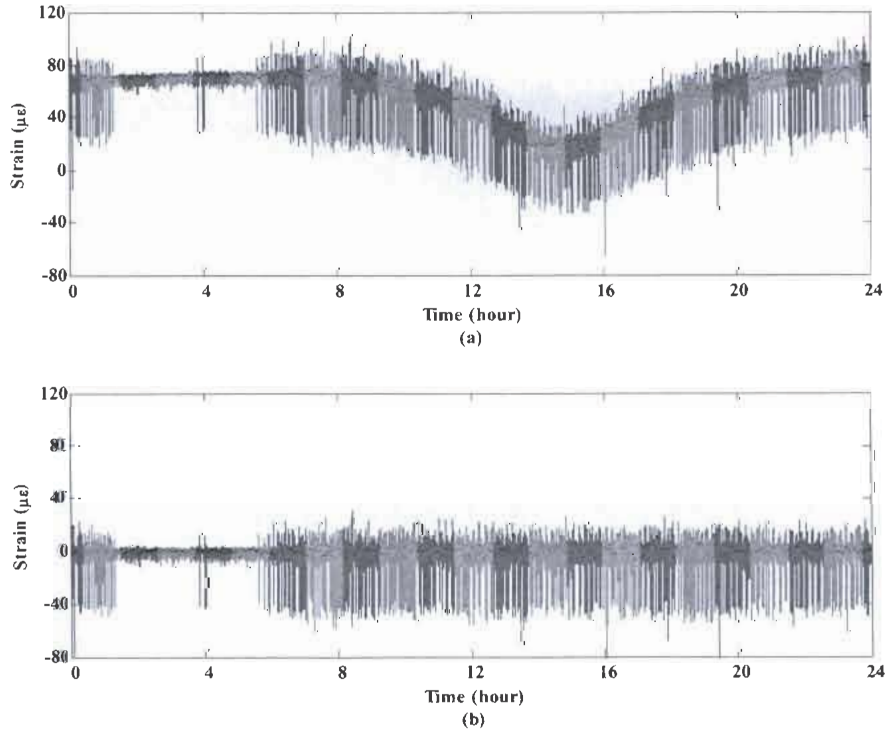

Fig. 5 Measured strain time histories of SPTDN04: (a) original data; (b) filtered data.

4.2 Statistics of peak strains

After pre-processing, the strain measurements are analyzed in terms of the correlations between peak strains of different components. Programs are developed to identify the peak values of strain measurements induced by trainload and compute their statistical distributions. Figure 6 shows the identified peak responses of strain gauge SP-TDS-04. By comparing with the train schedule, it is found that although the actual strain response is influenced by multiple loadings (e.g. highway loading, railway loading and wind loading), the dominant peaks are mainly induced by the railway loading. The stain data at the other locations are also analyzed in the same way.

4.3 Correlation analysis and discussion

Due to the movement of trains, the peak strains at different sections caused by the same train occur at different time. The correspondence among the strain peaks of different components is first identified through a program. In this procedure, the false-identified strain peaks are removed to make the vectors of peak values have the same length. In particular, two trains running in opposite directions may pass the bridge simultaneously. As a result, only one dominant peak can be observed at some locations, while there are still two peaks at the other sections. These special situations are also captured in the identification procedure.

Figure 7 shows the strain peaks of SS-TDN-02 before and after deleting non-corresponding peaks during one-hour period, from 20:00:00 to 20:59:59 on 2 November 2005. Comparing with Figure 6, it is found that the vectors of peak values of SP-TDN-04 and SS-TDN-02 are of the same length after the pre-processing.

Figure 8(a) plots the relationship of peak strains of two strain gauges at the same section—SP-TDN-04 and SS-TDN-02. A strong correlation between these strain gauges can be observed in the figure. Figure 8(b) plots the relationship of peak strains of two strain gauges—SP-TDN-02 and SP-TLN-03, both of which are diagonal components but located in two different sections which are 1133.7m apart from each other. Compared with Figure 8(a), Figure 8(b) shows much weaker correlation between the strain gauges of SP-TDN-02 and SP-TLN-03.

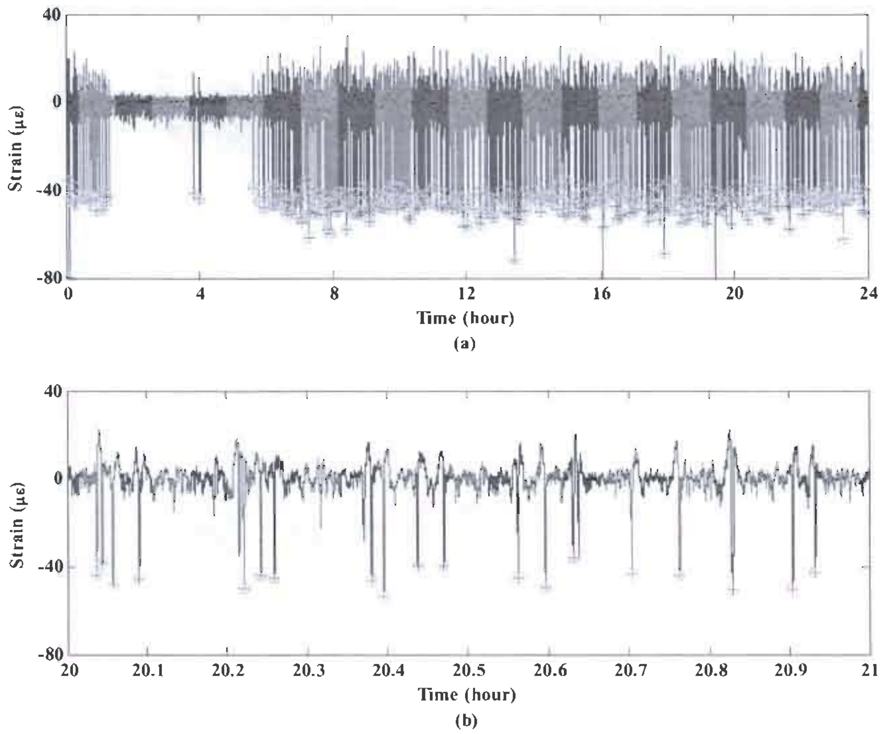

Fig. 6 Statistics of strain peaks(SPTDN04) on 2 November 2005: (a) one day duration from 00:00:00 to 23:59:59; (b) one hour duration from 20:00:00 to 20:59:59.

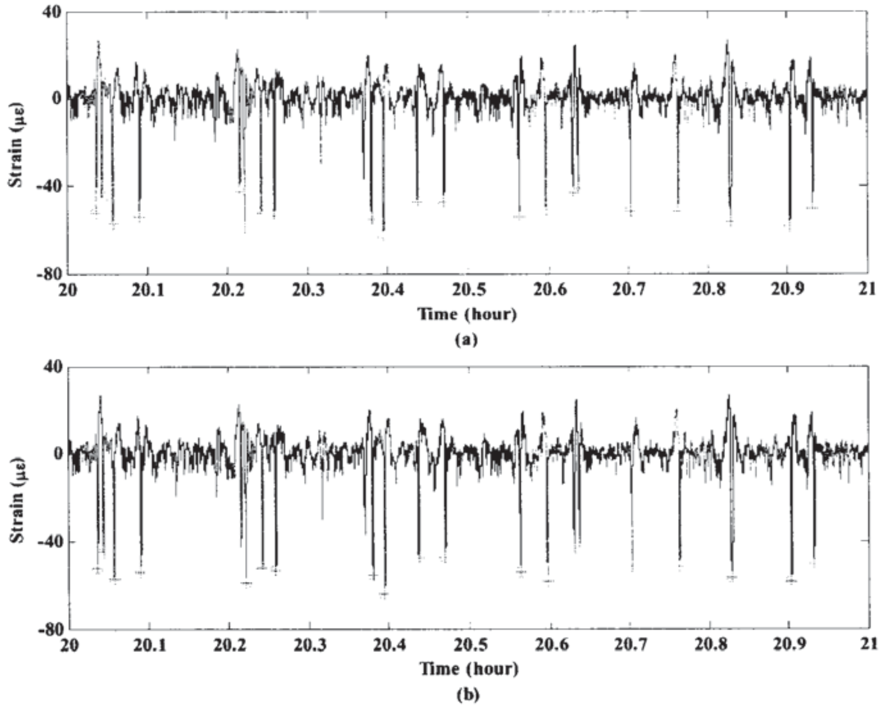

Fig. 7 Strain peaks of SS-TDN-02 during the period from 20:00:00 to 20:59:59 on 2 November 2005: (a) before deleting non-corresponding peaks; (b) after deleting non-corresponding peaks.

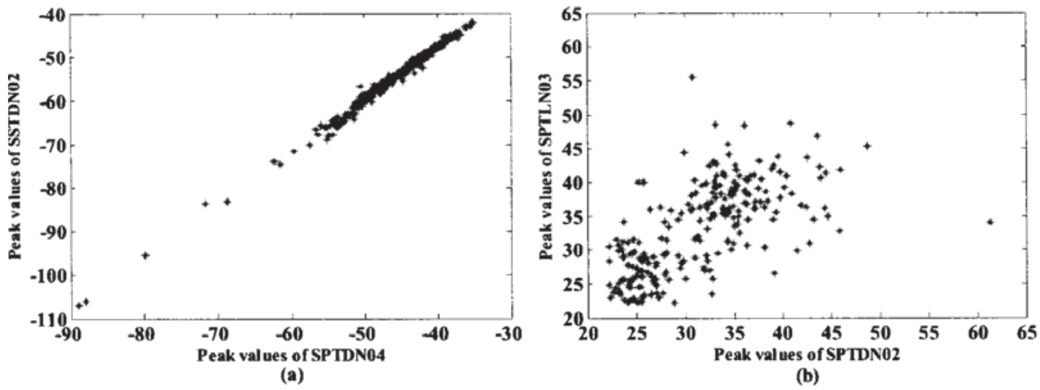

Fig. 8 Correlation analysis of strain measurements: (a) SS-TDN-02 and SP-TDN-04; (b) SP-TLN-03 and SP-TDN-02.

Correlation coefficient ρ which is a measure of the degree of linear interrelationship between two variables is used to analyze the correlations of peak strains response in this paper. The correlation coefficient is defined as[16]

$$\rho = \frac{\mathrm{cov}(\mathbf{X}, \mathbf{Y})}{\sqrt{\mathrm{var}(\mathbf{X})\,\mathrm{var}(\mathbf{Y})}} \tag{1}$$

where ρ is the correlation coefficient; **X** and **Y** are random variables respectively, and refer to the peak strains in this study; 'var' means the standard deviation of random variables; the symbol 'cov' means the covariance of two random variables

$$\mathrm{cov}(\mathbf{X}, \mathbf{Y}) = E\big[(\mathbf{X} - E[\mathbf{X}])(\mathbf{Y} - E[\mathbf{Y}])\big] \tag{2}$$

where E means the average value of random variable.

As described before, eight pairs of components, including four pairs of diagonal members and four pairs of bottom chords of the longitudinal truss in two different sections were analyzed. Table 1 presents the correlation coefficients of peak strain responses in the same section. As shown in table 1, the correlation coefficients of the strain peaks of bottom chords are almost equal to one, representing a highly linear relationship between the strain peaks of bottom chords. For diagonal components, the correlation coefficients are also close to one when they are on the same side of the bridge. However, the correlation coefficients of strain peaks decrease greatly when the diagonal members are on two different sides (north and south), and a poor correlation is observed even though two diagonal members are symmetrical about the longitudinal axis. The likely explanation is when a train is running on one track, the induced peak strains of components on two sides are different.

Table 2 gives the correlation coefficients of peak strains in two different sections which are 1133.7m apart. Unlike those in the same section, both the peak strains in diagonal members and in the bottom chords of the longitudinal truss show a relatively weak correlation between two different sections, especially the peak strains in bottom chords which show very poor correlation.

Table 1. Correlation coefficients of strain measurements in the same section

Section	On the Same side				On Different side			
	Diagonal member		Bottom chord		Diagonal member		Bottom chor	
	Sensor Number	ρ	Sensor Number	ρ	Sensor Number	ρ	Sensor Number	
4488.00	SPTDN02	-0.9804	SSTDN02	0.9953	SPTDN02	-0.2771	SPTDN04	0.
	SPTDN03		SPTDN04		SPTDS08		SPTDS11	
	SPTDS09	-0.9816	SSTDS08	0.9936	SPTDN03	-0.2522	SSTDS08	0.
	SPTDS09		SPTDS11		SPTDS09		SSTDN02	
4662.50	SPTLN02	-0.9798	SSTLN03	0.9959	SPTLS02	-0.5152	SPTLS12	0.
	SPTLN03		SPTLN05		SPTLN02		SPTLN05	
	SPTLS02	-0.9798	SSTLS09	0.9936	SPTLS03	-0.5085	SSTLS09	0.
	SPTLS03		SPTLS12		SPTLN03		SSTLN03	

Table 2. Correlation coefficients of strain measurements in different section

Position	Direction			
	North		*South*	
	Sensor Number	ρ	Sensor Number	ρ
Diagonal member	SPTLN03	0.6365	SPTLS02	0.5043
	SPTDN02		SPTDS09	
	SPTLN02	0.6235	SPTDS08	0.1863
	SPTDN03		SPTLS03	
Bottom chord	SPTLN05	0.0510	SPTLS12	0.0622
	SPTDN04		SPTDS11	
	SSTLN03	0.0425	SSTLS09	0.2247
	SSTDN02		SSTDS08	

5. Conclusion

This paper presents a correlation study of peak strain responses in a cable suspension bridge using field strain measurement data obtained from a comprehensive SHM system. Eight pairs of components, including four pairs of diagonal members and four pairs of bottom chords in the longitudinal trusses in two different sections were analyzed in the case study. The peak values of field strain measurements were identified and used in the correlation analysis. The results show that there is highly linear relationship between the peak strain responses of bottom chords in the same section. The correlations of peak strains in those diagonal members on the same side are significant, while the correlations of the peak strains are poor when diagonal members are on the different sides of the bridge even though they are in the same section and symmetrical about the longitudinal axis. Unlike the results in the same section, both the peak strains of diagonal members and the bottom chords in two different sections far apart have relatively weak correlation. In particular, the peak strains of the bottom chords have very poor correlations. More field strain measurements need to be investigated to obtain the overall variation law of the correlations of trainload induced responses in the cable suspension bridge. Numerical simulation based on finite element method will also be attempted in future study.

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