

Vibration-based Monitoring for Transverse Cooperative Working Performance of Assembled Concrete Multi-Girder Bridge: System Design, Implementation and Preliminary Application

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Abstract

The hinge joints of the assembled concrete multi-girder bridge (ACMGB) are the main components of transverse transmission of loads and the decline of their transmission capacity would lead to the degradation of the transverse cooperative working performance (TCWP) of the multi-beam system. As the key working performance of this kind of bridges, its severe degradation would result in the single-girder-bearing phenomenon inducing bridge collapse. In order to detect TCWP degradation timely, this paper proposes a vibration-based monitoring scheme of the ACMGB, where structural vibration information are collected by accelerometers and dynamic strain gauges at the bottom of girders in mid-span. These monitoring data are then used for identifying transverse mode shape of the multi-beam system and its derivated index in order to evaluate TCWP. This scheme has been implemented on three adjacent ACMGBs in Tongji Road Viaduct in Shanghai and the preliminary analysis of monitoring data proves the effectiveness and rationality of the monitoring scheme proposed in this paper, which has the potential of the TCWP evaluation.

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1. Introduction

The assembled concrete multi-girder bridge (ACMGB), a common kind of medium and small span bridge, is widely used in China [1], South Korea[2], Japan[3] and USA[4]. For example, the concrete hollow core slab bridge[5],
5 the adjacent box - beam bridge [6] and the concrete bulb-tee bridge[7] can be classified as the ACMGB. Its universal construction method is to erect pre-cast concrete girders on site firstly, then construct hinge joints (shear keys) or diaphragm, and finally pour integrated deck pavement to ensure the whole structure working monolithically. Therefore, hinge joints, girders and deck pavement
10 are the main components of the superstructure of this type bridges, as shown in Fig. 1 where reinforcement and accessory structures are ignored. Given its advantages over cast-in-place concrete bridges, ACMGB can be seen as an early but still popular practice of superstructure construction of accelerated bridge construction (ABC) in USA[4, 8].

15 From the viewpoint of design, the mechanical properties of ACMGB should be similar to that of an orthotropic concrete slab or grillage system. In other words, when a vehicle acts on one of the girders, the adjacent girders should also bear part of the load through the load-transmitting function of transverse joints, which is defined as transverse cooperative working performance (TCWP).
20 TCWP prevents the single-girder-bearing phenomenon, as shown in Fig. 2. However, after years of engineering practice, the ACMGB usually suffers from longitudinal reflective cracking, leakage as well as prestressed strand corrosion of the girders. It's believed that the cracking of hinge joints is the main cause of other diseases in the structural perspective, while vehicle load and environment
25 conditions are the external cause of them. After damage of the hinge joints develops, or to say, degradation of TCWP, the non-uniform load distribution

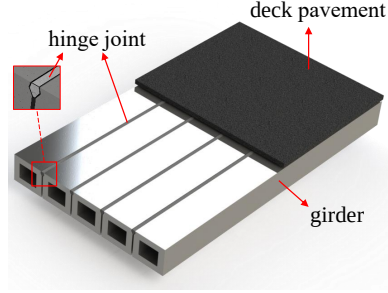


Fig. 1. Schematic diagram of main components of ACMGB superstructure

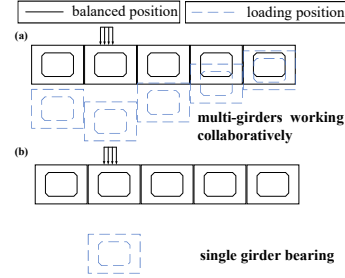


Fig. 2. Schematic diagram of working state of ACMGB in cross-sectional perspective

problem between girders under vehicle load is intensified and the deck pavement has to play a bigger role on the transverse force transimission which may results in longitudinal cracks in the deck pavement. Also, the cracked hinge joint would be vulnerable to the moisture of rains leading to prestressed tendons corrosion, and water stains at the bottom of girders. Obviously, after the occurrence of the above diseases, the TCWP of the ACMGB and resistance of girders would deteriorate. Therefore, the cracking of hinge joints can be regarded as a sign of deteriorated TCWP which may induce the single-girder-bearing phenomenon and even failures of the ACMGB[9–11].

At present, studies on the cracking cause of hinge joints have developed from the early insufficient strength of hinge joints under vehicle loads [12] to environmental conditions, like temperature and humidity[13], and cumulative fatigue damage[14]. Correspondingly, the design solution includes new hinge joint structures[6], transverse prestressing[15], the increase of pavement layer thickness, and the recent use of UHPC material[16]. The application of UHPC to hinge joints seems to be a state-of-the-art solution[17], but it's still in the early stage of promotion and the improvement of the long-term performance of hinge joints has not been fully and widely verified in the industrial community.

Consequently, in addition to improving hinge joint performance in terms of design, it is more important for ACMGBs in service period to identify hinge joint transmission function in time, evaluate TCWP, and carry out appropriate hinge

joint maintenance. The hidden hinge joints assessment methods are divided into four ones: visual inspection, load test[18], nondestructive testing[19, 20] and structure health monitoring (SHM)[21–26]. The first three methods above can hardly indentify the early disease in hinge joints in time. Since the SHM technology takes full use of the sensors installed on the structure to continuously measure of the structure response, it can overcome the shortcomings of the above three methods, and has the potential of realizing early damage detection of hinge joints and reliable evaluation of TCWP.

The transverse force transfer mechanism of ACMGB is not only reflected in the transverse distribution of quasi-static effect between girders, but also included in the vibration response’s transverse distribution. For the former one, random vehicle’s self-weight is a interference factor for TCWP evaluation so it’s difficult to directly apply quasi-static monitoring information for assessing TCWP. To overcome this problem, the authors analyzed a large quantity of quasi-static information obtained from long-term monitoring to extract an convergent index in a statistical sense, and indirectly realized the monitoring of the TCWP of the ACMGB[22]. In order to achieve statistical stability, this method require calculation of a large amount of data, so the real-time performance is poor. Since the modal components in the structural vibration response are almost independent of the external excitation[27], it’s convenient to realize the independent identification of the TCWP through vibration response monitoring of the ACMGB, without considering the random influence of vehicle load. This technology route relies on the identification of modal parameters, so big data seems not to be necessary. Besides, small computation and good real-time performance should be its advantages.

Considering all this, this paper starts from the simplified dynamic model of the ACMGB of which modal analysis principle is studied and on this basis a vibration-based monitoring scheme for monitoring and evaluating the TCWP of ACMGBs is proposed (section 2). Next, an implementation on an actual ACMGB is introduced (section 3). Then, Through the preliminary analysis of the monitoring data, the feasibility of the proposed scheme are revealed (section

4). Lastly, some problems and challenges in the further application are discussed
 80 (section 5). This paper can be seen as a preliminary study on the vibration-
 based TCWP monitoring route of the ACMGB.

2. Vibration-based lateral cooperative performance monitoring scheme

2.1. Monitoring placement scheme

In the previous work[23], it has been proved that, under certain assumptions,
 85 the modal vibration of ACMGBs can be analyzed by means of a multi-beam
 model connected by distributed springs. In this section, first of all, the modal
 analysis process of the multi-beam model is given. Secondly, the TCWP inver-
 sion route of ACMGB based on the modal parameter-derived index is estab-
 lished and the corresponding sensor placement scheme of the TCWP monitoring
 90 system is put forward.

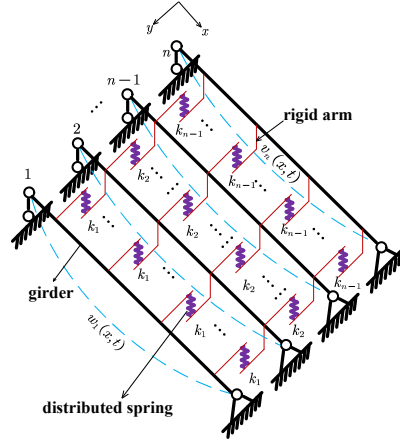


Fig. 3. Distributed springs-connected multi-beam model of ACMGB

Considering the mechanical properties of the ACMGB structure, the follow-
 ing assumptions are made:

- Every beam is a uniform Euler-Bernoulli beam, ignoring the torsional effect, and each beam has only vertical displacement degree of freedom.

- 95 • The massless springs are longitudinally continuous distributed and only
transfer vertical shear force between adjacent beams. The TCWP between
the single beams is characterized by the stiffness of the distributed spring.
- Each beam in longitudinal direction is simply supported.
- The cross-section characteristics and material properties of each beam
100 are constant in longitudinal direction and the stiffness per unit of each
distributed spring is also constant in longitudinal direction.

The schematic diagram of the simplified distributed spring-connected multi-beam model is shown in Fig. 3. The direction x is longitudinal and the direction y is transverse. The stiffness per unit of i th distributed spring is k_i and v_i is the
105 vertical displacement of the i th beam. Note that the virtual rigid arm in Fig. 3
is used to simulate the transverse transfer path of vertical shear force, which is
an infinite, massless element of stiffness.

The modal analysis process of distributed spring-connected multi-beam system is as followed: Firstly, execute time and space variables separation of displacement. Then assume global mode shape $\Phi_v(j)$ is a product of mode shape
110 for single beam system $\varphi_j(x)$ and transverse proportion coefficient vector $\mathbf{a}_j$
to separate longitudinal and transverse space variables. Thirdly, using the analytical characteristics of simply supported Euler-Bernoulli beam system, the
solution of the j th modal parameters of the distributed spring-connected multi-
115 beam system can be transformed into the following matrix eigenvalue problem

$$\mathbf{K}_j \mathbf{a}_j = \omega_j^2 \mathbf{M} \mathbf{a}_j \quad (1)$$

ω_j is natural frequency for mode j of the multi-beam system, matrix $\mathbf{K}_j$
and $\mathbf{M}$ are stiffness matrix and the mass matrix regarding multi-beam system

respectively, given by

$$\mathbf{K}_j = \begin{bmatrix} \tilde{\omega}_k^2 + \kappa_1 & -\kappa_1 & 0 & 0 & \cdots & 0 \\ -\kappa_1 & \gamma_2 \tilde{\omega}_k^2 + \kappa_1 + \kappa_2 & -\kappa_2 & 0 & \cdots & 0 \\ 0 & -\kappa_2 & \gamma_3 \tilde{\omega}_k^2 + \kappa_2 + \kappa_3 & -\kappa_3 & \cdots & 0 \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & 0 & \cdots & -\kappa_{n-1} \\ 0 & 0 & 0 & 0 & \cdots & \gamma_n \tilde{\omega}_k^2 + \kappa_{n-1} \end{bmatrix} \quad (2)$$

$$\mathbf{M} = \begin{bmatrix} 1 & 0 & 0 & 0 & \cdots & 0 \\ 0 & \mu_2 & 0 & 0 & \cdots & 0 \\ 0 & 0 & \mu_3 & 0 & \cdots & 0 \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & 0 & \ddots & 0 \\ 0 & 0 & 0 & 0 & \cdots & \mu_n \end{bmatrix} \quad (3)$$

$\tilde{\omega}_k$ is the corresponding modal frequency of the longitudinal base function or to say the longitudinal mode shape of a single beam, γ_n is the stiffness of the n th beam, κ_{n-1} is the stiffness of the $(n-1)$ th distributed spring, and μ_n is the mass of the n th beam. The above parameters are all dimensionless.

It can be further assumed that the stiffness and mass of each beam is identical, which means $\gamma_i = 1$, $\mu_i = 1$, $i = 2, 3, \dots, n$, and $\mathbf{K}_j = \tilde{\omega}_k^2 \mathbf{I} + \mathbf{K}$, hence the eigenvalue problem (1) can be transformed into

$$\mathbf{K} \mathbf{a}_j = \lambda \mathbf{a}_j \quad (4)$$

$$\mathbf{K} = \begin{bmatrix} \kappa_1 & -\kappa_1 & 0 & 0 & \cdots & 0 \\ -\kappa_1 & \kappa_1 + \kappa_2 & -\kappa_2 & 0 & \cdots & 0 \\ 0 & -\kappa_2 & \kappa_2 + \kappa_3 & -\kappa_3 & \cdots & 0 \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & 0 & \cdots & -\kappa_{n-1} \\ 0 & 0 & 0 & 0 & \cdots & +\kappa_{n-1} \end{bmatrix}, \lambda = \omega_j^2 - \tilde{\omega}_k^2$$

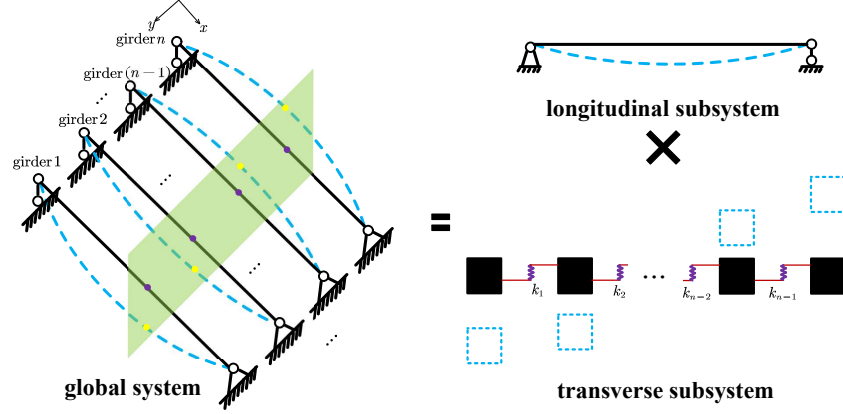


Fig. 4. The global system is decomposed along longitudinal and transverse direction into two types of subsystems. The green quadrilateral in the global system represents a specific cross-section plane. The purple point is the intersection of the plane and the balance position of girders. The yellow point is the intersection of the plane and a certain mode shape of the global system.

It can be seen that the value of the transverse participation coefficient vector $\mathbf{a}_j$ is only determined by $\mathbf{K}$ and its order j depends on the magnitude ordering of ω_j . The stiffness matrix $\mathbf{K}$ is only related to the stiffness of the distributed spring and can be interpreted as a n identical mass block system connected by $n - 1$ springs. In other words, the spring stiffness of the mass block system decide $\mathbf{a}_j$. At this point, the distributed spring-connected multi-beam global system can be equivalent to the product of the transverse subsystem and the longitudinal subsystem, as shown in Fig. 4. The longitudinal subsystem is a simply supported single beam describing the longitudinal characteristics of each girder, while the transverse subsystem is a spring-connected mass block system which represent the characteristics of hinge joints. $\mathbf{a}_j$ is the mode shape of the transverse subsystem and is defined as transverse mode shape for its novelty. The according modes are named as transverse modes. The detailed qualitative analysis and parameter analysis of the transverse mode shape can be found in the previous work[23].

As mentioned above, the stiffness of the distributed spring is used to char-

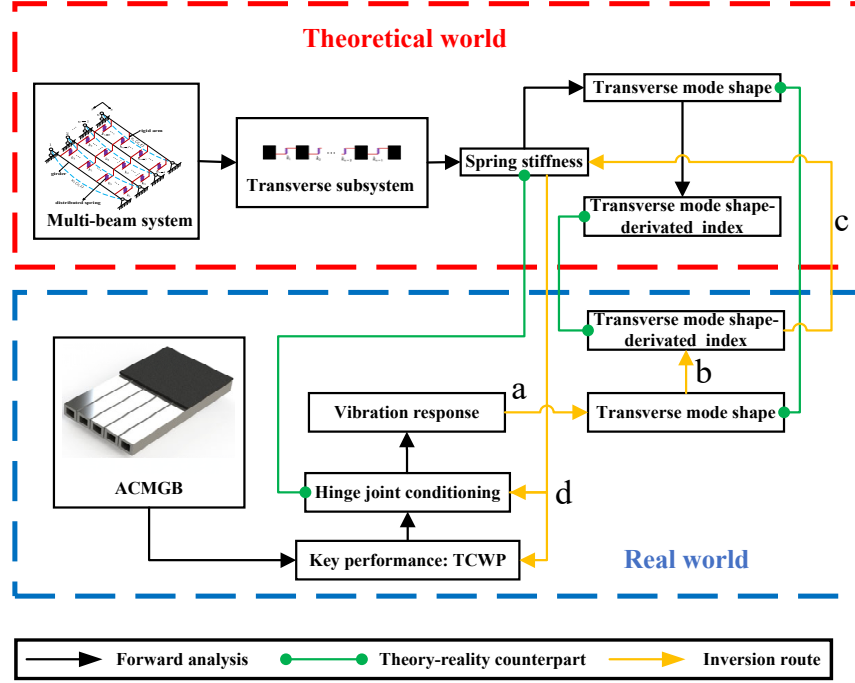


Fig. 5. Technical route diagram: a represents modal identification, b stands for index extraction, c means index characterizing, and d indicates diagnosis and evaluation.

acterize the conditioning of hinge joints and is further linked with the TCWP.

140 The transverse mode shape contains the information of the transverse subsystem or to say the spring stiffness's information. Therefore, the index derived from the transverse mode shape can be used to represent TCWP. The forward analysis logic and TCWP inversion route are shown in Fig. 5. Inside it the theory-reality counterpart presents the connection between the theoretical word

145 and the real world which justify the rightness of inversion route in concept.

To identify transverse mode shape, arranging sensors on any cross-section to measure the vertical vibration information is reasonable. However, considering that the ACMGB is generally of relatively large stiffness, the problem vibration signal-noise-ratio (SNR) may be low and the amplitude of midspan section is

150 often higher than other sections, so arrangement of sensors at the bottom of the girders in midspan section is a more concrete and suitable scheme.

2.2. Choice of sensors for vibration information collecting

For vibration monitoring of civil engineering structures, the most commonly used sensors is the accelerometer. In addition, the dynamic strain gauges is also an option. In general, with appropriate sampling frequency, strain data of bridge structures contains four components[28–33]. According to different time scales, they are static component induced by dead load, temperature-induced component, quasi-static component resulted from static effect of vehicle load and vibration component caused by vehicle load dynamic effect.

Since the longitudinal normal strain at the bottom of girders is mainly controlled by the bending of the girders while the longitudinal subsystem shows the bending vibration of the girders when the multi-beam system vibrates, the longitudinal normal strain of at the bottom of girders must contain abundant vibration information. Furthermore, the lateral arrangement should reflect the vibration characteristics of the transverse subsystem. Therefore in this study, vibration information of ACMGB comes from both accelerometers and strain gauges. The vibration component of strain is further used to identify the transverse strain mode shape and then utilized for structure performance evaluation.

3. Real bridge implementation of vibration monitoring system

Built in 2001, the Shanghai Tongji Road Viaduct has been in safe service for 19 years. Because of its heavy or even overloading traffic load which is mostly composed of container trucks, its TCWP has been greatly degraded. Apparent diseases, longitudinal cracks in the deck pavement (to see Fig. 6a) and water leakage in hinge joints is easy to be found. Exactly, in its service period, the pavement and hinge joints have undergone repairing and strengthening of many times, but due to no suitable solution to the heavy load problem, these diseases recurred again and again. In the summer of 2016, three adjacent spans, K92, K93 and K94, are chosen to be monitoring span where vibration-based TCWP monitoring system proposed above implemented, as is shown in Fig. 7a.

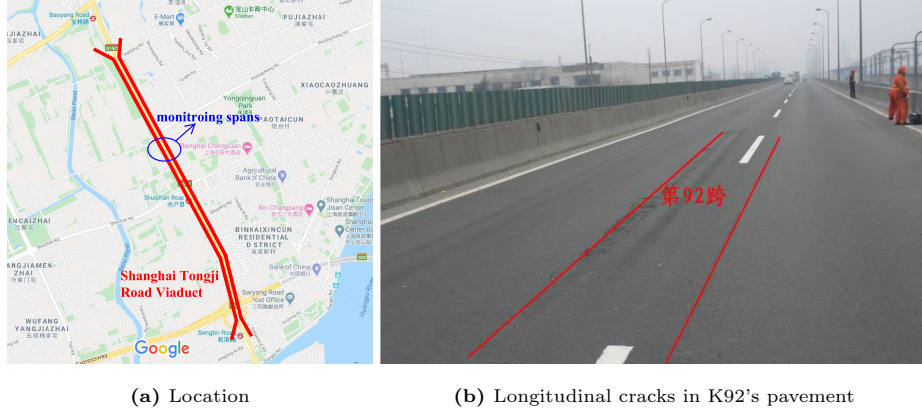


Fig. 6. Shanghai Tongji Road Viaduct

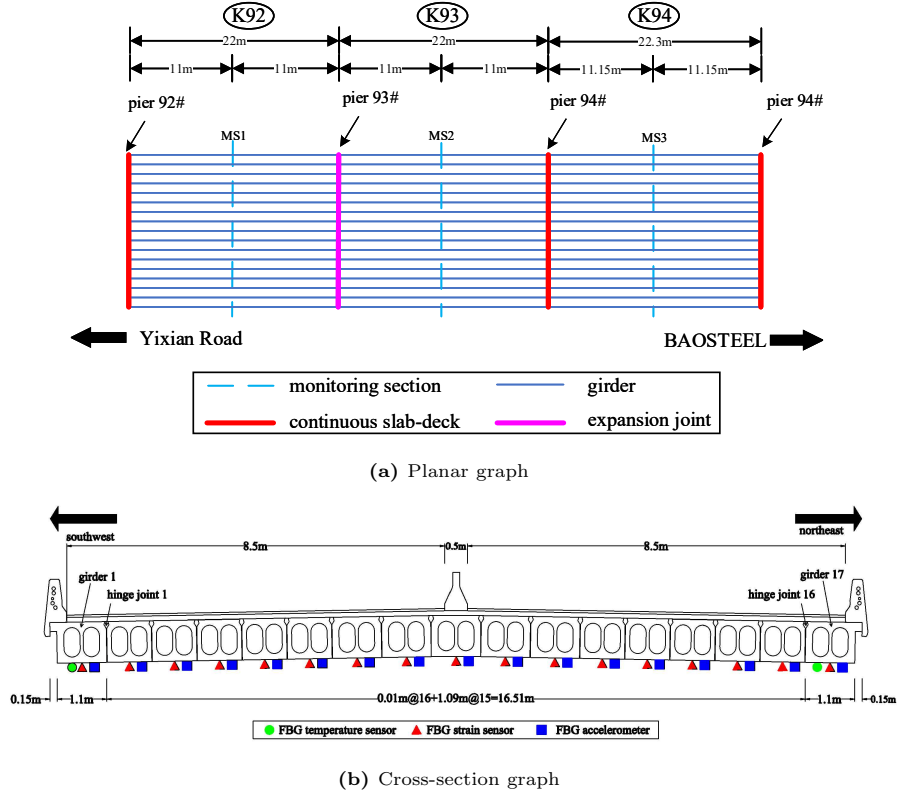


Fig. 7. Schematic diagram of sensor placement

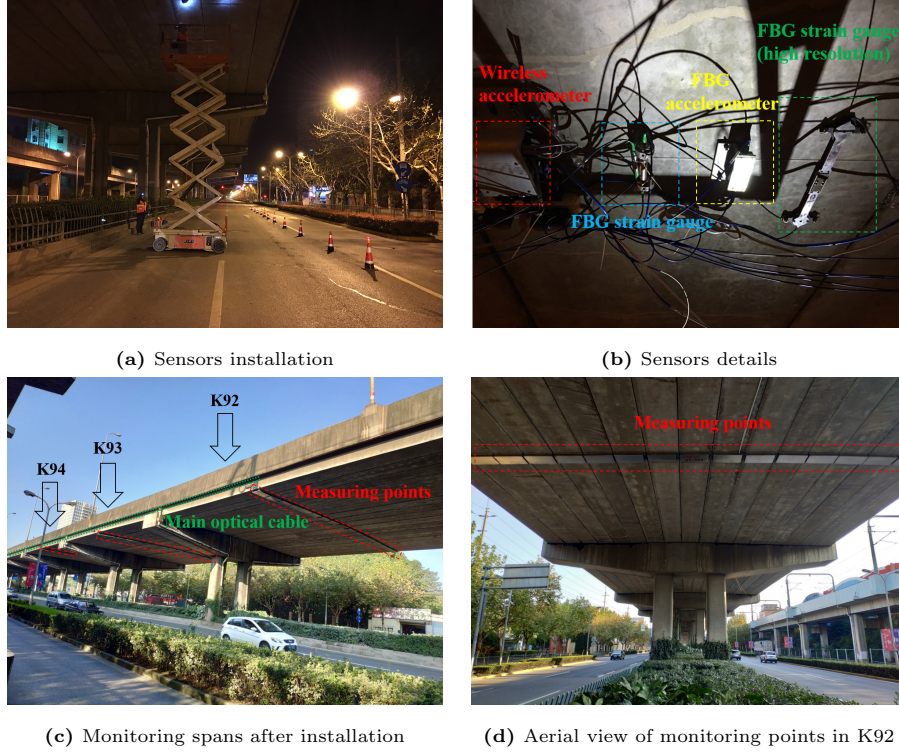


Fig. 8. Monitoring system implementation graph

180 The selected spans' transverse layout is 0.5m barrier+8.5m lane+0.5m median+8.5m lane+0.5m barrier, so the total width is 18.5m. 17 pieces of slab girder of 0.9 m height is connected by pavement and hinge joints to form multi-beam system. Except that the K92 and K93 are entirely isolated by the expansion joint in the decks, the structure in the rest of the supporting location
185 are simply supported with continuous deck details, shown in Fig. 7a. In order to facilitate the follow-up evaluation, from southwest to northeast, girders is numbered as girder 1 to girder 17 and hinge joint number rule is similar (to see Fig. 7b).

The Fiber Bragg Grating (FBG) accelerometers and longitudinal normal strain gauges are arranged at the bottom of each girders in the midspan section
190 of three spans, as shown in Fig. 7. For studying the effect of environmental

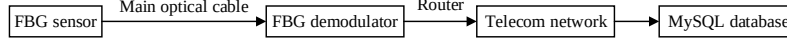


Fig. 9. Monitoring system topology graph

temperature on the monitoring index, FBG thermometers are also installed at the side girders. All the sensors in the midspan section share a main cable guaranteeing good synchronization of multi-channel measurement. The optical
 195 signals of different spans' sensors are also connected by the main cable and are further converted into electrical signals through FBG demodulator with a sampling frequency of 50Hz. After that, the electrical signals are transmitted into the telecom network by the router, and are finally transmitted to the server for storage. The monitoring system implementation diagram and topology diagram
 200 are shown in Fig. 8 and Fig. 9, respectively.

4. Preliminary analysis of vibration monitoring data of K92

The preliminary analysis of monitoring data starts with the time-frequency feature analysis of measured vibration response (i.e., data quality assessment) and identification of modal parameters follows. At the end there would be a tentative performance assessment using transverse mode shape-derived indexes.
 205

4.1. Time and frequency domain analysis of measured acceleration and strain response data

In order to truly reflect the time-frequency characteristics of the two type of measured response data, raw data are analyzed without preprocessing like
 210 filtering. Unless noted specially, the data analyzed are taken from the FBG acceleration and strain monitoring data of K92 on July 1, 2018. In the case of single-channel data analysis or to say time domain analysis, the corresponding monitoring data of girder 15 is used while in the case of multi-channel data analysis or to say frequency domain analysis, the monitoring data of all girders
 215 are taken.

4.1.1. Time domain features

To better analyze the time domain features of the two types of data at different scales, scales of hours, minutes and seconds are selected to study the time domain features.

220 As is known to all, the acceleration measurement in time domain should present symmetry oscillations form around a certain baseline[27], which is also a typical time domain feature of vibration. It can be seen from Fig. 10 that the three -scale time domain diagrams of the acceleration at the bottom of K92 show obvious symmetric oscillations. Note that due to the measurement setup
225 problem, the acceleration data has a baseline close to 105mg.

Compared with acceleration data, strain data is more complex, or in other words, contains more abundant information. It can be seen from Fig. 11 a that the most evident component of strain data in hour-scale is the changing baseline or trend term under the action of dead load and temperature. The biggest
230 difference between it and the above acceleration data's is that, in hour-scale, the trend term of strain is presented as nonlinear offset, while the acceleration's is linear drift. Take the strain data of 5min around 12:00 as an example to observe the time domain feature in minute-scale, as shown in Fig. 11 b. Due to high traffic flow density at noon, the strain data in minute-scale includes many
235 peaks with burring. The shape of each peak is similar to the shape of the bending moment influence line of the simply supported beam bridge, which is mainly the quasi-static response generated by moving vehicles. Furthermore, a section of data of 16s is selected to observe the features in second-scale, as shown in Fig. 11 c. It is evident that in second-scale, more high-frequency oscillating components
240 can be observed riding above the slowly changing baseline in the strain data. The oscillating component is related to the vibration response of the structure while the slowly changing baseline corresponds to the quasi-static response. As for such oscillatory components, some existing studies regard them as noise, and ignore them by filtering or signal reconstruction. Of course, this depends on the
245 research purposes. However, it is undoubtedly a waste of valuable monitoring

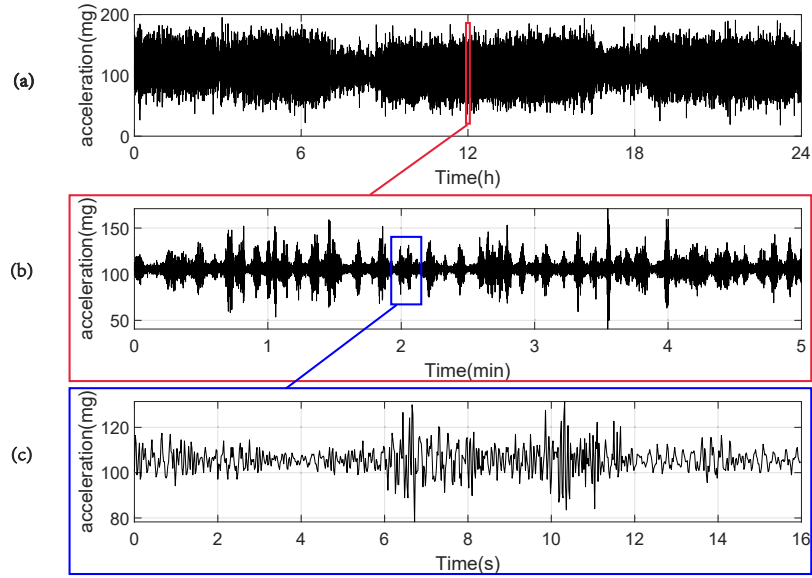


Fig. 10. Time domain diagrams of acceleration at different scales: (a)hour (b)minute (c)second

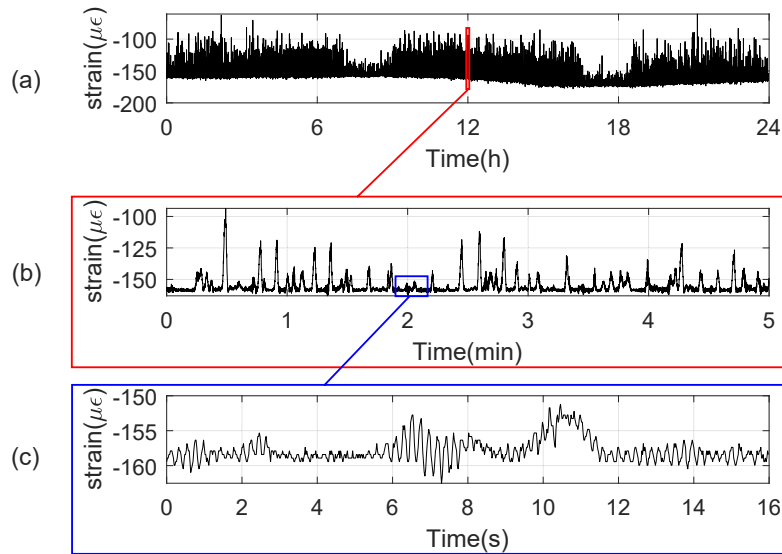


Fig. 11. Time domain diagrams of strain at different scales: (a)hour (b)minute (c)second

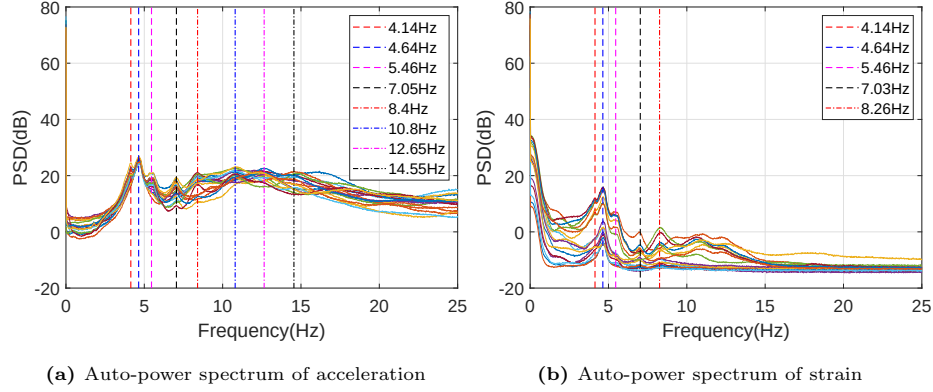


Fig. 12. Frequency domain feature figure of monitoring data: Each solid line of different color represents the auto-power spectrum of a channel's data while the vertical dotted lines and dash-dotted lines suggest the location of peaks identified manually.

information to overlook the details of high-frequency oscillatory signals.

To sum up, the acceleration data and strain data should include vibration information. When the system is excited, the acceleration and strain would be synchronously excited, so if the acceleration oscillation is apparent, the strain should also contain vibration component. This can be found by comparing Fig. 10 c and Fig. 11 c.

4.1.2. Frequency domain features

In addition to oscillations in time domain, the vibration behavior of mechanical systems is also featured by the power distribution in frequency domain. But ACMGBs belong to the category of medium and small bridges, and their global stiffness and mass is often larger than that of composite bridges of the same span. Therefore, weak-excitement[34] is indeed a common problem. Fig. 12 shows the auto-power spectrum of acceleration and strain data, which is estimated by Welch's method. The adopted window function is a hanning window with a length of 4096 points and its segment overlap rate is 50 %. Besides, the FFT calculation points are 4096.

As can be seen from Fig. 12, due to the limited exciting level of the vehicles for ACMGBs, the spectrum peaks from measured data are not as abundant as

those obtained in the laboratory vibration test[35], regardless of acceleration or
265 strain. Although the excitation effect is not very good, a number of peaks are
still apparent and the modal frequencies can still be identified by peak picking
manually.

Comparing the strain and acceleration auto-power spectrum, the lower fre-
quency bounds of the vibration components of both are about 4Hz and the
270 peak locations of power spectrum have good consistency. Of course, the accel-
eration data have a richer high-frequency vibration component in the range of
9-15 Hz. This phenomenon can be explained as follows: under the Euler beam
hypothesis, the longitudinal strain is proportional to the second order partial
derivative of the deflection curve with respect to space, so the frequency domain
275 characteristic is consistent with the deflection. Moreover, acceleration can be
transformed into deflection by a low-pass system filtering with an amplitude of
 $1/\omega^2$, therefore the quasi-static and trend term of the strain are particularly
obvious in the frequency domain, while the high-frequency components would
be masked by noise due to small power spectral density.

280 Three components can be clearly seen in the Fig. 12b. The static component
with the maximum amplitude stays at 0Hz, the quasi-static component is be-
tween 0 and 4Hz with a second-largest amplitude and vibration components is
in the range of 4-9hz. Therefore, according to the features of different scales (or
different frequency bands) of each component of the strain, a rough component
285 separation method is FFT bandpass filtering.

4.2. Operational modal identification based on frequency domain decomposition method

As mentioned above, the main technical route of this paper is output-only
health monitoring and its corresponding modal identification technology is often
290 referred to as operational modal analysis (OMA) technology. OMA usually as-
sumes that the random environment excitation (mainly vehicle load) over long
time is presented as white noise[36], it mainly depends on the second order
moments of the structure under white noise as 'theoretical proxy'[37]. The 'the-

oretical proxy' estimating and identifying modal parameters from 'theoretical
 295 proxy' by parametric or nonparametric method are two main steps for OMA.
 Depending on taking the correlation function or power spectrum as 'theoretical
 proxy', OMA can be divided into time domain method and frequency domain
 method and the representatives of the two kinds of methods are stochastic sub-
 space identification (SSI)[38] and frequency domain decomposition (FDD)[39],
 300 respectively. SSI can realize more automated modal identification, but its pa-
 rameter selection is always a difficult problem and choosing large model order
 and identifying the real mode by stable diagram is a widely-used solution. In
 the case of low SNR, it tends to identify many false modes. Besides, large
 calculation time is its another shortcoming. Though FDD needs manual peak
 305 picking, its robustness is easy to guarantee. Considering that only preliminary
 data analysis is conducted and big data analysis is not necessary in this paper,
 FDD with less influence from noise is adopted.

4.2.1. Principle of frequency domain decomposition method

Under white noise excitation, the power spectrum density matrix of the civil
 engineering structures, $\mathbf{G}_{yy}(i\omega)$, can be expressed by Eq. (5), where N is the
 number of modes, d_r is a scalar constant, λ_r is r th pole, Ψ is the mode shape
 matrix of which each column is r th mode shape ϕ_r ($r = 1 \cdots N$). The super-
 script $*$ and T represents the complex conjugate and transpose, respectively.
 Besides, $diag(\cdot)$ means the diagonal matrix operator while $Re(\cdot)$ denotes the
 real-part operator. Also note that i here stands for the imaginary unit.

$$\mathbf{G}_{yy}(i\omega) \approx \sum_{r=1}^N \left(\frac{d_r \phi_r^* \phi_r^T}{i\omega - \lambda_r} + \frac{d_r \phi_r^* \phi_r^T}{-i\omega - \lambda_r^*} \right) = \Psi^* diag \left(2Re \left(\frac{d_r}{i\omega - \lambda_r} \right) \right) \Psi^T \quad (5)$$

It can be observed that Eq. (5) have a similar form as matrix singular value
 310 decomposition (SVD)[40]. Therefore it's possible to pick peaks from the mea-
 sured vibration signals' singular value spectrum using manual and automated
 algorithms[41] to identify modal frequency while the corresponding singular vec-
 tors is taken as mode shape. Generally, the first singular value is considered in

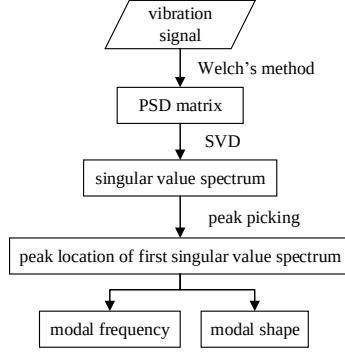


Fig. 13. FDD flowchart

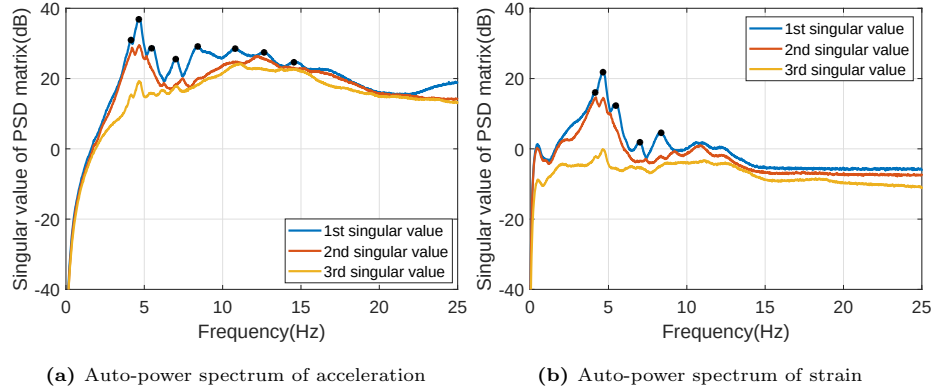


Fig. 14. Singular value spectrum of monitoring data: black points are peaks picked manually

this process. In this paper, FDD flowchat is shown in Fig. 13.

315 4.3. Modal frequency and mode shape results

The singular value spectrum of K92's acceleration and strain data collected on July 1st in 2018 is shown in Fig. 14. By picking peaks in the first singular value spectrum, modal frequencies of the displacement mode (DM) and strain mode (SM) are identified and listed in Table 1. The number of strain modes
320 identified is obviously less than that of displacement modes, but the consistency of the displacement mode's and the strain mode's modal frequencies indicates the possibility of mining vibration information in the strain data.

Table 1. Modal frequencies of K92’s identified displacement modes and strain modes

Mode		1st	2nd	3rd	4th	5th	6th	7th	8th
$\omega(\text{Hz})$	DM	4.15	4.65	5.47	7.01	8.41	10.80	12.65	14.55
	SM	4.15	4.65	5.46	7.01	8.36	—	—	—

It should be pointed out that the above singular value spectra of acceleration and strain are based on Welch power spectrum density matrix by multi-segment averaging of one day’s data. If the vehicle-passing event is taken as the sign of data collection, the size of collected data would be much less and the estimated power spectra also contain more burring, which is adverse to the later modal identification. Therefore, the authors believes that although the excitation level of the ACMGB is not high under the ambient actions, as long as enough operational state data are collected, even the strain’s vibration information with low SNR can be recognized and further used for the characterization of key performance.

The corresponding transverse displacement mode shapes (TDMSs) and transverse strain mode shapes (TSMSs) in K92 are shown in Fig. 15 and Fig. 16.

It can be seen that, not taking mode 1 into account, the 2nd-7th TDMS of K92 is very similar to the theoretical qualitative mode shape of the simplified transverse subsystem model:

- The second TDMS of K92 presents an approximate horizontal line, which has a good consistency with rigid body mode shape (a horizontal line) of the simplified transverse subsystem. The difference originated from the not exactly same the stiffness and mass of each girder of K92, which is not completely complied with the assumption of the simplified model.
- The other six mode’s TDMS is similar to harmonic shapes and the harmonic order rises with the increase of the mode order, which is consistent with the non-rigid body mode shape of the simplified transverse subsystem.

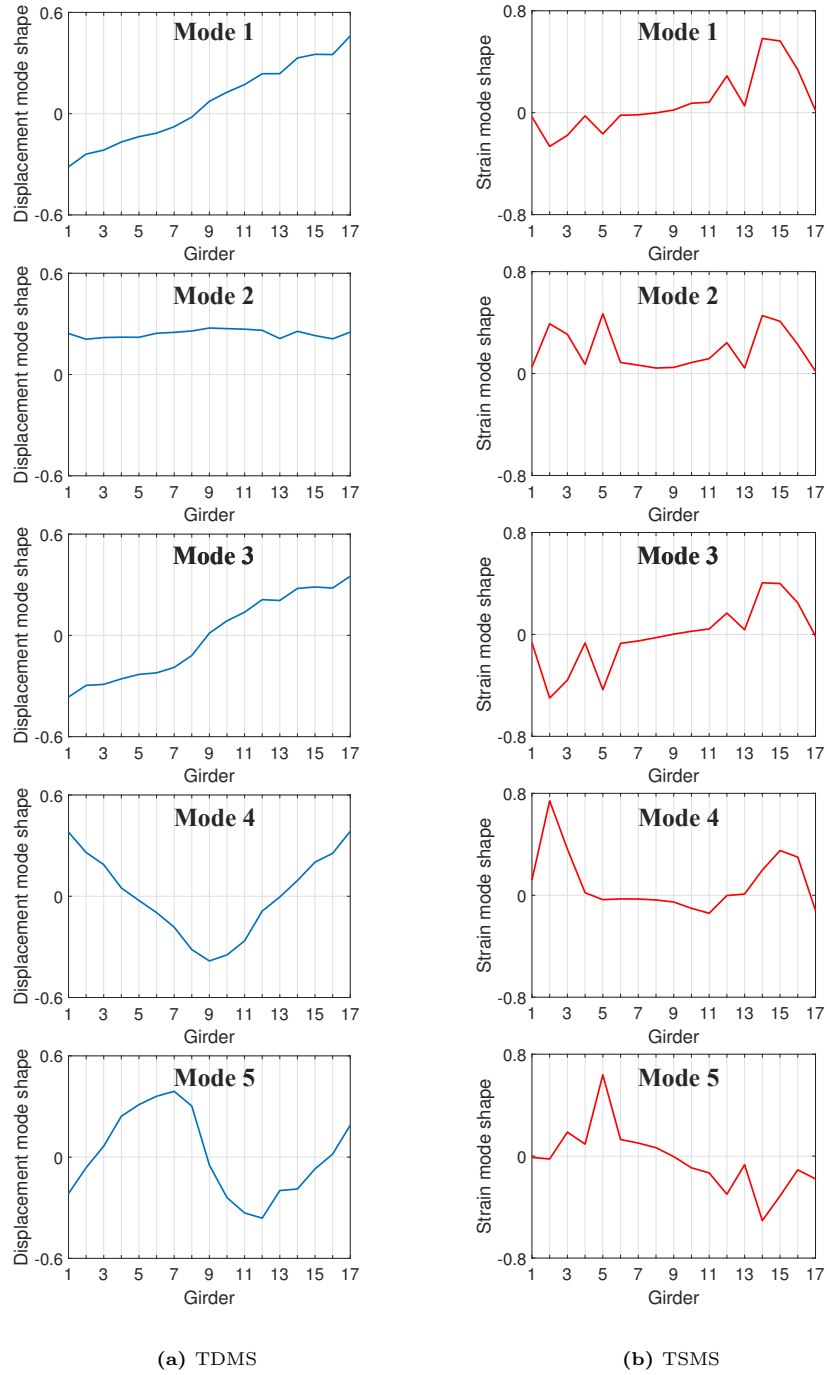


Fig. 15. Comparison TDMS and TSMS of the first five modes in K92

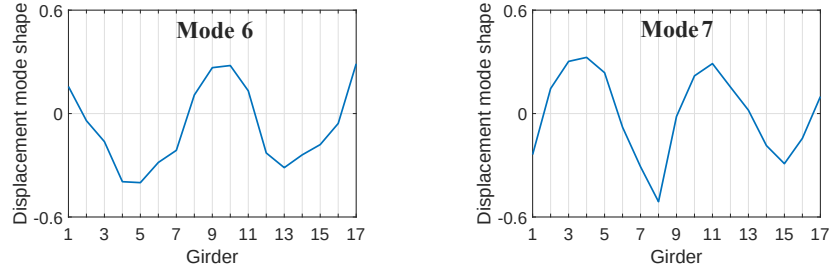


Fig. 16. 6th and 7th TDMS in K92

In fact, the transverse rigid body mode corresponds to the vertical bending mode of the ACMGB, while transverse non-rigid body modes correspond to torsion modes. Given that the vertical bending mode is mainly controlled by the behavior of a single beam, the transverse rigid body modal frequency and mode shape are not sensitive to hinge joint damage, that is, the spring stiffness decrease in the transverse subsystem model.

By the above analysis, the concept of transverse subsystem for the ACMGB proposed in this paper is to some extent proved to be in line with the actual situation. However, the 1st TDMS at 4.1hz appears to be similar to that of the 3rd TDMS, which is inconsistent with the transverse subsystem theory of the simplified model. This issue would be discussed in section 5.1.

In the authors' previous work[23], it's deduced that there is one-to-one correspondence in shape between TSMS and TDMS of the multi-beam system according to the transformation relationship between euler beam displacement and strain. However, as observed between Fig. 15a and Fig. 15b, the first five mode's TDMSs are seemingly different from the corresponding TSMSs. If comparing identified TSMS with the ideal transverse mode shapes (harmonic shapes) of the simplified transverse subsystem, the discrepancy is more serious.

Some similar situations exist in strain or curvature modal testing [42] and the reason for them is that the strain measurement is sensitive to local stiffness. If the local stiffness of different strain measuring points are of great difference, it's easy to find some outliers relative to the baseline in strain mode shape, which have a great influence on the understanding of strain mode shapes and

370 their further use. Actually, for strain measuring points arranged as we proposed above, the local stiffness may be distinct, which not only can be attributed to low construction quality, but also be due to random size effect of concrete material. Therefore, it is an objective fact that the identified TSMSs shown in Fig. 15b present abnormal shapes due to the existence of outliers. Considering these, if
 375 removing outliers and comparing the identified TSMSs and TDMSs, there are some similarities between them at the baseline level. For example, for mode 2 and 3, ignore the outliers at girder 2, 3, 5, 12, 14 and 15 and the obtained baseline will show an approximate horizontal line and an anti-symmetric harmonic shape.

Of course, there is another possible reason for the abnormal shape of the
 380 identified TSMSs in Fig. 15b, that is, they are results of spurious modes. In order to falsify this possibility, more strain modes were identified from the strain data of the following six consecutive days (July 2-7, 2018). The variation of them relative to the first day's is studied and the mode shape difference was measured by modal assurance criterion (MAC). See Eq. (6) for MAC calculation,
 385 where $\phi_{i,A}$ represents the i th element of mode A , n indicates the total element number of A 's mode shape, and the mode of B another mode shape that is compared with the mode shape of A . MAC value varies from 0 to 1, with 0 for no correlation (huge difference) and 1 for full correlation (no difference).

$$\text{MAC}_{A,B} = \frac{|\sum_{i=1}^n \phi_{i,A} \phi_{i,B}|^2}{\sum_{i=1}^n (\phi_{i,A})^2 \cdot \sum_{i=1}^n (\phi_{i,B})^2} \quad (6)$$

Fig. 17 shows the MAC results of TSMSs identified from the next six days'
 390 data and the first day's. Obviously, these 6 days of TSMSs are very similar to the first day's, as most MAC values are greater than 0.9, so it can be considered that the TSMS identified on July 1, 2018 is relatively stable. Even though they are far from the ideal TSMSs, they do represent the tranverse characteristics of operational strain to some extent and are not spurious.

395 4.4. Preliminary evaluation of K92 based on transverse mode shape

For preliminary evaluation, only short-term (1 day) monitoring data are used. According to the previous study[23], modal absolute difference (MAD)

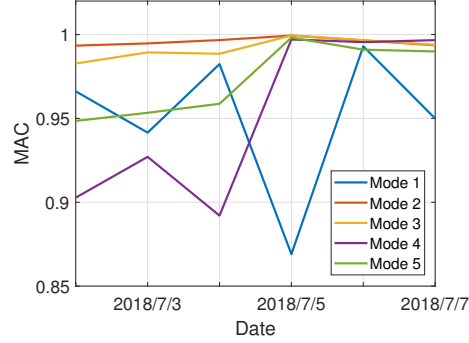


Fig. 17. MAC between the first day's TSMSs and the next six days'

index is an appropriate choice, which is defined by Eq. (7). In essence, MAD captures the difference of vibration amplitude and phase between adjacent girders, on which the degradation of TCWP caused by hinge joint damage is inferred based. Therefore, locating outliers on MAD is the main step of evaluation. The previous modal parameter identification results have shown that the mode 1 is not consistent with the theory of this paper and the previous work while the mode 2 is the 'rigid body mode', which is not sensitive to hinge joint damage. Besides, TSMSs have outliers on baselines. Considering all this, in this section, the 3rd-7th TDMS are utilized for MAD calculation and preliminary TCWP evaluation.

$$MAD_j(i, i+1) = |\phi_{i,j} - \phi_{i+1,j}| \quad (7)$$

The MAD results are shown in Fig. 18 and outliers are listed in Table 2. Because the transverse modes have different sensitivity to hinge joint damage, the selected outliers are also different. It should be noted that, since there are additional masses on the girder 1, 9 and 17 from the accessory structures (the barriers and median), the related MAD may present outliers which are excluded as outliers in this paper.

In order to indirectly verify the feasibility of MAD method, a field survey was conducted in K92 on November 23, 2018. and the girder bottom picture is shown in Fig. 19. It can be observed that there were evident marks of water

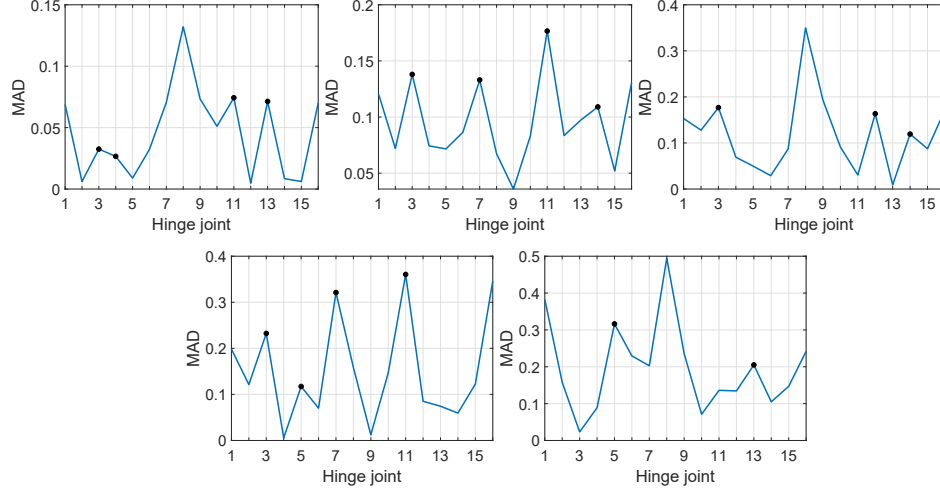


Fig. 18. MAD of 3th-7th TDMS. The black points indicate identified outliers and show the TCWP degradation locations of preliminary evaluation.

leakage in different length under hinge joint 3, 4, 5, 6, 9, 11, 12, 13, 14 and 15, which are considered of obvious damage, and the corresponding TCWP had deteriorated to different degrees. Therefore, most of the selected MAD
420 outliers can correspond to the damage of the actual hinge joints, indicating that the proposed MAD method has the potential to evaluate the TCWP of the ACMGB.

The preliminary application also shows that the MAD method has the following problems in practice which are common in damage detection based on
425 vibration index :

- The inconsistency of the results of multi-mode MAD outliers makes it more difficult to accurately locate the damage of hinge joints.
- Although most hinge joint damage can be located, the damage degree is difficult to assess.

430 The purpose of this paper mainly lies in studying the TCWP evaluation of ACMGB, i.e., the damage detection of hinge joints. In addition to the transverse cooperative working system, the evaluation of the performance and damage

Table 2. Outliers identified from MAD

Mode	Hinge joint number of outliers
3	3,4,11,13
4	3,7,11,14
5	3,12,14
6	3,5,7,11
7	5,13



Fig. 19. Field picture of K92 on November 23, 2018. The blue line denotes locations of damaged hinge joints causing evident water leakage.

of the girder is also of great importance to the ACMGB management. As mentioned above, the ideal transverse strain 'rigid body mode shape' should be close to the horizontal line. If there are evident outliers from the horizontal line, they can be regarded as results of the corresponding girder stiffness changing. The 2nd TSMS does contains many outliers, mainly at girder 2, 3, 5, 12, 14, 15 and it's reasonable to infer that the stiffness of these girders have decreased significantly.

5. Discussion

5.1. Rationality of the simplified model for qualitative analysis

As previously stated, most of the identified transverse mode shapes are basically the same as those of the transverse subsystem of the simplified model. However, it seems that the transverse mode shapes of mode 1 are difficult to be explained by it, and may be identified as the spurious mode. Here we provide a possible explanation from the view point of the simplified model's rationality: In fact, the transverse subsystem model proposed in this paper is simplified. Only unidirectional linear displacement spring is used to connect the multi-beam system while the actual hinge joints should also have the effect of transmitting bending moment, i.e., there should also be torsional springs. Therefore, the proposed transverse subsystem would have a mode of integral rotation (also a 'rigid body mode') in which the torsional springs between girders would not work at all and the according transverse mode shape would appear an oblique line, to which the identified 1st TDMS is highly close.

5.2. Comprehensive utilization of various information in strain

As analyzed in section 4, it is clear that there is more than one component in the strain of the ACMGB, and its quasi-static information can also be used to evaluate the TCWP[22]. By utilizing various information in strain comprehensively, the quality of evaluation should be enhanced. However, it seems that how to realize strain multi-component coseparation in an accurate

and automated way is still a unsolved problem. Although there have been a lot of attempts to separate strain components[28, 32, 43], but according to the author’s preliminary testing, some of these methods are still far from the pareto optimal solution. It’s still difficult to achieve high quality of coseparation of the
465 quasi-static and vibration components.

5.3. *Necessity of long-term analysis*

The data analysis in this paper is still of the category of short-term or small data analysis and only the index in no need of the baseline can be used. Although this method has’t reached a satisfactory level due to some factors, it still
470 shows some applicability and provides useful enlightenment for future research, i.e., long-term monitoring and analysis of indexes. Long-term continuous monitoring makes it possible to attain massive indexes of different time, and the corresponding analysis method is called long-term analysis method[44–48]. The analysis of long-term monitoring data mainly evaluates the structural perfor-
475 mance by identifying the trend of changeing indexes in the background of big monitoring data, which is considered to be a more robust data utilization technology route. The above long-term analysis ideas would provide better tools for this paper’s goal realization and demonstrate necessity for research.

5.4. *Physical concept orientated middle and small span bridge monitoring system design* 480

Consndiering limited management funds, the monitoring system of medium and small bridges is often not equipped with a large number of sensors of various types like the one of long-span bridges. However, the main structures of them are relatively simple and the disease patterns are also comparatively clear.
485 Therefore, the corresponding monitroing system design should begin with disease pattern mining to find out the specific key performance. Then, the key performance is modeled in mechanical system which should not only be simplified enough to model key performance but also be of enlightenment to indexes

construction. Next, depending on how to identify the indexes, the sensor place-
490 ment, selection and other hardware details are determined. Last but not the
least, the index analysis based on monitoring data are designed. The ACMGB
monitoring system proposed in this paper can be served as an solid example
and the above four steps are concise summary for section 2 and 3.

6. Conclusions

495 This paper first introduced the distributed spring-connected simply-supported
multi-beam model for the modal analysis of the ACMGB, finding that if assum-
ing each beam to be identical, the global vibration system can be divided into
simply-supported single beam subsystem and spring-connected mass block sys-
tem along the longitudinal and transverse direction. Besides, the mode shapes
500 of transverse subsystem were defined as the transverse mode shape, which is
completely determined by the spring stiffness. Inspired by this, this paper pro-
posed an TCWP monitoring and evaluation scheme based on the transverse
mode shape-derived index which is identified from vibration information col-
lected from sensors arranged at the bottom of girders in midspan. Moreover,
505 the vibration information can be sensed by accelerometer and dynamic strain
gauge. This vibration-based TCWP monitoring system is implemented in Tongji
Road Viaduct and through the preliminary analysis of the short-term vibration
monitoring data of it, the purpose of evaluating the TCWP of the ACMGB is
realized. The concrete conclusions are as follows:

- 510 • The measured longitudinal normal strain of girder bottom contains abun-
dant information and its vibration information can be used to identify the
transverse strain mode.
- The transverse displacement mode identified by acceleration data are qual-
itatively consistent with those of the simplified transverse subsystem, prov-
515 ing the rationality of the simplified model to some extent.

- The MAD index based on the transverse displacement mode shape has the potential to evaluate the TCWP of the actual ACMGB, while the transverse strain mode shape can provide additional information about the girder degradation.

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