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Research on Bridge Structural Health Assessment Based on Finite Element Analysis

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Abstract: In view of the content of bridge condition assessment and health monitoring, this paper is based on the finite element simulation analysis. The uncertain finite element model updating method based on sequential optimization strategy is studied, and the uncertain modal parameter data obtained by health monitoring system are applied to upgrade the uncertain finite element model of cable-stayed bridges, which provides a more accurate finite element model for subsequent reliability analysis. Firstly, the finite element dynamic analysis of the main span structure of the bridge is carried out, and the natural frequencies and modes are obtained. Then the measured natural frequencies of the structure are obtained by estimating the power spectrum of the dynamic monitoring data, and the theoretical values are compared with the measured ones. The dynamic characteristics of the modified two-stayed bridge finite element model are verified by the load test results. The results show that the modified finite element model can simulate the dynamic characteristics of the actual structure well. Most of the measured and calculated displacement increments were within the margin of error. The error is within 5%, which can accurately reflect the true stress state of the structure. The uncertainty model based on the sequential optimization strategy is simple and can be applied to the uncertainty of the finite element model of the actual bridge structure.

Keywords: bridge data; finite element simulation; health monitoring; state assessment

1 INTRODUCTION

Monitoring the structural safety, integrity and durability of important large bridges to ensure their safe and normal operation has become an important issue of great concern. It is well known that besides the impact of sudden accidents, earthquakes and other natural disasters, the accumulation of damage near the integral joint of steel bridge structure and the brittle fracture therefrom are the main causes of bridge damage [1]. Therefore, the long-term state assessment of existing large bridges under operational loads should be the main objective of safety and durability monitoring of such structures. In addition, a large number of existing railway bridges in China are mainly steel structures, many of which have entered the later stage of their design service life, and some have obvious hidden dangers [2, 3]. In the environment of rapid development of national economy and continuous increase of railway speed, the safety, fatigue life and ultimate load evaluation of these bridges have become urgent problems to be solved.

In recent years, the application of structural health monitoring system on large and important bridges has provided an excellent opportunity to break through the bottleneck of the development of civil structure analysis. Structural health monitoring system provides the real response of the bridge under the condition of on-line operation, and provides a reliable basis for model identification of the bridge structure. However, some bridge structural models are still simple "fishbone" shape, which inevitably makes the results of finite element structural analysis far from the measured results [4]. Literature [5] proposed that the dynamic damage diagnosis technology of long-span bridge structures based on dynamic parameter test is difficult to effectively apply to structural state early warning and evaluation [5]. The bridge health monitoring system data has not been fully applied, and even formed a situation of "massive garbage data" [6]. With the application of structural health monitoring system in large and important bridges, more and more attention has been paid to the finite element modelling and analysis of bridge structures, and the means of modelling tend to be "similar" to the structure entity of bridges [7]. This finite element structural model has been used to calculate the dynamic characteristics of bridges and the results are quite consistent with the measured results, but the analytical model for structural dynamic response is still far from the measured results [8]. Modelling methods tend to be "similar" to bridge structural entities in structure. The welded area of welded members can only be simplified as a joint, which cannot reflect the local stress concentration effect in these areas and the fatigue accumulation caused by local stress concentration. This shows that even the three-dimensional finite element model of the solid bridge which is similar to the bridge structure in geometrical structure cannot satisfy the analysis of the state identification and fatigue assessment of the bridge structure [9, 10]. However, it is impossible and unnecessary to simulate precisely the structure of the solid entity.

Through a brief review of the research status of health monitoring and condition assessment of bridge structures, this paper further clarifies the understanding of key theoretical and technical issues in structural health monitoring and condition assessment. For the full play of the massive data of bridge structure health monitoring, this paper reveals the changes of environmental effects, structural responses and cumulative damage effects of bridges during service, serves the maintenance and operation management of bridges, and feedbacks and improves the design of bridge structures. Other aspects have important academic value and engineering application value.

2 KEY THEORY AND TECHNOLOGY OF BRIDGE STRUCTURAL HEALTH ASSESSMENT

With the application of structural health monitoring system in large and important bridges, more and more attention has been paid to the role of finite element modelling and analysis of bridge structures [11]. The author believes that the finite element modelling strategy of long-span bridge structure should be based on the goal of finite element analysis. Different objectives can lead to different modelling strategies and different models can be obtained. For the purpose of bridge design, the finite element analysis only needs to establish a relatively simple finite element model. As long as the results of calculation and analysis tend to be conservative, the design goal can be achieved. For the purpose of structural health monitoring and condition assessment, the finite element model needs more precise finite element model to get accurate calculation and analysis results [12]. Otherwise, it may be "lost by a thousand miles" and fail to achieve the goal of structural health monitoring and condition assessment. Even in the process of structural health monitoring and state assessment, there are different requirements for structural finite element model. The following will specifically illustrate the different modelling strategies for different objectives in the process of structural health monitoring and state assessment.

2.1 Dynamic Characteristics of Bridges by Point Arrangement Scheme

In order to establish an effective bridge structure health monitoring system, it is necessary to design a reasonable and efficient sensor placement scheme for monitoring purposes and needs. At present, the health monitoring of bridge structure is mainly based on the overall detection method of structure, which is mainly based on the modal analysis of vibration experiment. The state of the structure can be judged by analyzing the dynamic mode change or structural stiffness change related to the dynamic characteristics of the structure. The validity of the sensor lies mainly in the merits of the modal experiment results [13]. Therefore, the location and number of sensors are very important to the experimental results. In addition, in the monitoring of dynamic response of bridge structures, in the design of point layout scheme, it is necessary to fully understand the position of key components, relative dangerous sections and dangerous points of bridges under typical loads.

(1) Objectives and technical requirements

The aim of finite element analysis of bridge structure is to accurately calculate the overall dynamic characteristics of bridge structure, such as frequency and mode shape, so as to preliminarily determine the measuring points according to the calculated mode shape. The internal force and stress distribution of bridge structures under typical loads are calculated accurately so as to determine the key components such as high stress zone, high fatigue stress zone and vulnerable parts. The corresponding finite element model must satisfy the following technical requirements: in order to effectively calculate the dynamic characteristics of the bridge structure, the internal force and stress distribution of the main components and cables in each part of the bridge under the design dead load must be analyzed more accurately.

(2) Solutions and strategies

In order to achieve the above objectives and technical requirements, the established finite element model should fully reflect the spatial geometry and topology of the bridge, the spatial distribution of mass and the physical and mechanical properties of the bridge material. The dynamic characteristics of the bridge are related to the mathematical discreteness, geometric configuration and material mechanical properties of the model, but according to the existing numerical experiment experience, the influence of geometric configuration plays an important role. Therefore, the spatial geometric topological configuration reflecting the structure should be put in the first place when establishing the finite element model. Secondly, the preliminary model should be revised according to all available measured data.

2.2 Finite Element Simulation of Structural State Assessment

After the establishment of the bridge structure health monitoring system, continuous on-line monitoring can be carried out on all parts of the bridge structure, resulting in a large amount of monitoring information. Using this monitoring information to assess the health status of bridge structures and detect possible damage is not only the information post-processing process of structural health monitoring, but also the main process of health monitoring. At present, there are many key theoretical and technical problems to be solved urgently in this process. Among them, establishing a finite element model of dynamic response of bridge structure aiming at structural state assessment to analyze and calculate the working state of bridge structure is the key to make full use of structural health monitoring information to achieve damage detection and state assessment [14]. In addition, because the installation process of sensors is often limited by the bridge structure, the location of sensor installation is not necessarily the dangerous part of the structure, such as the welded joints of various types of components in the bridge often have a high probability of fatigue damage. However, it is very difficult to deploy sensors in these locations, so it is necessary to establish a more refined finite element model to reflect the local stress concentration distribution of key components in bridge structures.

(1) Objectives and technical requirements

The objective of finite element analysis of bridge structure aiming at structural state assessment is to accurately calculate and analyze the dynamic response of bridge structure under actual applied load and the hot stress distribution of key components in bridge structure. On this basis, fatigue damage cumulative process and other types of failure process are further simulated and analyzed. In order to achieve this goal, the corresponding finite element model should meet the technical requirements: in addition to the dynamic characteristics of the finite element model should meet the requirements in the structural stress simulation, but also include the key components of the welding, stress concentration at the joints. In the boundary condition of the structure, the accurate simulation of the actual operating load of the bridge should be included.

(2) Solutions and strategies

In order to achieve the above objectives and technical requirements, the urgent problem to be solved in structural simulation is that the difference between the full scale and the local damage detail scale is very wide. Its characteristic element size is in millimetre scale. Although the damage process occurs in the details of local components, the cause of the damage - environment and working load acts on the whole bridge structure. At the same time, because the local damage originates from the most disadvantageous parts of the structure, the overall structural analysis is inevitable in the process of structural damage analysis. Obviously, it is impossible to analyze such structures in the same scale space as other damage analysis such as automobile and mechanical structures, whether in terms of calculation capacity, or practical operability. The disparity between the full scale of structure and the detail scale of local damage in civil structures makes it difficult to simulate and analyze the evolution and development process of local defects and damage considering the actual existence of structures in structural damage and failure analysis. The common problems existing in damage and failure analysis of long-span structures need to be studied theoretically and computationally. For this reason, the academic idea of establishing multi-scale structural damage analysis model for long-span structures has been put forward, and research has begun under the support of relevant projects of the National Natural Science Foundation of China.

For the finite element simulation of bridge structure dynamic response for the purpose of structural state assessment, the following two key problems must be further solved on the basis of finite element simulation of structural dynamic characteristics: Firstly, in the process of structural modelling and analysis, the defects occurring in the most disadvantageous parts of the structure and their evolution process, as well as their effects on the structural response, are appropriately considered [15]. The second is to establish the moving load model of serving bridges based on the on-line measured information of bridge loads. This problem will be put on hold for the time being because it has gone beyond the scope of this paper. The first way to solve the above problem is to establish a multi-scale structural analysis model. For the description of the whole or different parts of a large span structure, the applicable theory and scale range will depend on the description purpose and the main object of investigation. When building multi-scale models from three aspects: full scale, local component scale and damage detail scale, the object of analysis concerned by each model scale, the applicable theory and the characteristic length of finite element should be different, as described in Tab. 1.

Table 1 Analytical objects, applicable theories and characteristic length of finite element elements concerned by various model scales

Model scale	Unit characteristic length / cm	Object of stress analysis	Applicable theory
Full-scale structure	50 ~ 100	Internal force of component	structural mechanics
Local component scale	5 ~ 10	Stress distribution in components	Mechanics of materials, elastic and plastic theory
Meso-scale	0.1 ~ 0.5	Hot spot stress distribution	Macro and micro damage theory

In addition, in order to simultaneously carry out the stress analysis and local damage analysis of the whole structure, the models of different scales should be able to link up and calculate. The processing of the convergence area is the key to the successful convergence and effective calculation of the model. It is expected that there will be many choices in connection modes, such as constructing special connection elements, calculating "transfer" by means of internal force equivalent or energy equivalent, and so on.

3 BRIDGE REAL-TIME HEALTH MONITORING BY DYNAMIC ANALYSIS AND MEASUREMENT

3.1 Finite Element Dynamic Analysis

In theory, all structural systems have distributed mass and have infinite degrees of freedom. However, except for some simple structures, most engineering structures will be extremely difficult to calculate as infinite degrees of freedom. Therefore, discretization method is often used to treat structures as finite degrees of freedom systems. The vibration equation of a finite multi-degree-of-freedom system is:

$$\boldsymbol{M}\ddot{X}+\boldsymbol{C}\dot{X}+\boldsymbol{K}X=P\left(t\right) \tag{1}$$

Formula $\boldsymbol{M}$ is mass matrix, C is damping coefficient matrix, $\boldsymbol{K}$ is stiffness matrix, $\boldsymbol{X}$ is displacement matrix, $P(t)$ is interference force matrix.

The natural frequencies and modes of structures can be derived from the undamped free vibration equation.

$$\boldsymbol{M}\ddot{X}+\boldsymbol{K}X=0 \tag{2}$$

If the system oscillates in a cylinder, then

$$\left(\boldsymbol{K}-\lambda^2\boldsymbol{M}\right)A=0 \tag{3}$$

In the formula, λ is angular frequency and A is mode shape. The natural frequency and corresponding mode shape can be obtained from the formula.

The finite element dynamic analysis of 190 m main span of Qianjiang Fourth Bridge is carried out with ANSYS software, and the theoretical white vibration frequency is obtained. The model uses three-dimensional beam element to simulate truss arch ribs, wind braces and all chords and webs. In addition, tied beams, arch rib beams, pier columns, arch columns, cross beams and end beams are also simulated by three-dimensional beam elements. The bridge deck system is simulated by grillage system, and its mass and section characteristics are converted according to the design drawings, which can better reflect the bending and torsion characteristics of the bridge deck; the suspender is simulated by rod element [16]. There are 7920 units in the main span.

The boundary conditions are as follows: (1) The arch foot is regarded as a rigid body, and the end of the arch rib, tied beam and end cross beam are subordinated to the arch foot in the direction of six degrees of freedom; the cross beam of the arch rib, the cap beam of the arch pillar, the cap beam of the pier pillar and the upper deck are connected by sliding hinges. (2) When the lower suspender passes through the upper suspender beam, its transverse and longitudinal displacements are constrained. In the model, the relative degrees of freedom are constrained by the coupling of nodes. (3) The whole 190 m span superstructure is a self-stable system, and the external constraints are simply supported. The three-dimensional displacement constraint is applied to the supporting points of the two arch feet on the cover beam of the left pier, and the vertical and transverse degrees of freedom are restrained at the right pier to release the longitudinal degrees of freedom of the bridge. The small support at the bottom of end beam is simply supported.

The first 30 order frequencies and corresponding modal characteristics are obtained by finite element dynamic calculation as shown in Tab. 2.

Table 2 Finite element dynamic calculation of 190 m main span structure

Order NO.	White vibration frequency (Hz)	Mode description
1	0.29369	Symmetrical lateral bending of arch ribs and upper deck
2	0.59543	Antisymmetrical vertical bending of arch ribs and upper and lower decks
3	0.63289	Symmetrical lateral bending of arch rib and lower bridge surface
4	0.71728	Symmetrical lateral bending of arch ribs and superstructure
5	1.1655	Antisymmetrical lateral bending of arch ribs and superstructure deck
6	1.3797	Symmetrical vertical bending of arch ribs and upper and lower bridge surfaces
7	1.3852	Antisymmetrical lateral bending-torsion coupling of arch rib and upper deck
8	1.6821	Symmetrical lateral bending-torsion coupling of arch ribs and upper deck
9	1.6987	Antisymmetrical vertical bending-torsion coupling of arch ribs and upper and lower decks
10	1.7072	Antisymmetrical lateral bending of lower deck
11	1.9315	Symmetrical vertical bending of arch ribs and upper and lower bridge surfaces
12	1.9765	Symmetrical lateral bending of arch ribs and upper deck
13	2.2355	Antisymmetrical vertical bending of arch ribs and upper and lower decks
14	2.2788	Antisymmetrical lateral bending of arch ribs and upper deck
15	2.3365	Antisymmetrical lateral bending-torsion coupling of arch ribs and upper and lower decks
16	2.3199	Antisymmetrical Vertical Symmetrical Bending-torsion Coupling of arch rib and upper and lower deck side
17	2.5044	Symmetrical vertical bending of arch ribs and upper and lower bridge surfaces
18	2.6135	Symmetrical vertical bending of arch ribs and upper and lower bridge surfaces
19	2.6341	Antisymmetrical lateral bending of arch ribs and upper deck
20	2.8381	Antisymmetrical vertical bending of arch ribs and upper and lower decks

3.2 Dynamic Data Analysis

The health condition of bridge structure is monitored and evaluated by testing signals, i.e. extracting various features from the signals collected by sensors, detecting structural parameters, condition monitoring and damage diagnosis, etc. When the sensor system can provide a large number of signals about the structure performance, in order to identify and locate the damage, an effective and reliable signal analysis and processing method is needed.

Bridge health monitoring data mainly consist of two parts, static data and dynamic data. Static data include suspender and pre-stressing cable tension, arch foot strain, tied beam bottom deflection, pier displacement. Dynamic data include arch rib and bridge deck acceleration, vault wind speed and temperature. How to effectively analyze these data is of great significance to bridge damage detection and safety evaluation. Dynamic data can be analyzed in time domain and frequency domain, while static data can only be analyzed in time domain. There are many methods for monitoring signal analysis and processing, among which Fourier transform is the most classical. It transforms the signal from time domain to frequency domain and decomposes the original signal into a weighted combination of orthogonal trigonometric functions.

Discrete sequence Fourier transform DFT and inverse transform IDFT formula:

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

$$X(n)=\frac{1}{N}\sum_{n=0}^{N-1} X(k)\mathrm{e}^{j\frac{2\pi}{N}nk},\ 0 \le (N-1) \tag{5}$$

where $x(n)$ is the original discrete sequence and $X(k)$ is the Fourier transform sequence.

Several points should be paid attention to when using DFT for spectrum analysis:

1) Sampling frequency should satisfy Nyquist sampling rate, i. e. $f_N \ge 2F_h$, in which F_h is the highest frequency of the signal to be analyzed and f_S is the sampling frequency. So the interval should satisfy $T \le \frac{1}{2F_h}$.
2) The duration t_p of the sample signal is $t_p = NT = \frac{N}{f} = \frac{1}{\Delta F}$, and ΔF is the frequency resolution of spectral analysis.
3) The number of points N of the discrete sequence is $N \ge \frac{2F_h}{\Delta F}$.
4) There is leakage and fence phenomena in DFT spectrum analysis, but the effect of these phenomena will be weakened when the window function is processed or the sequence length is increased.

In the actual analysis process, the calculation speed of DFT is relatively slow. When the sequence length is long, it takes a long time for the computer. Therefore, FFT (Fast Fourier Transformation) is generally used. For the environmental random excitation vibration test method, we directly use Fourier transform to analyze the harmonic components and frequency response characteristics [18]. The effect may not be good. If the self-power spectrum of monitoring data is used to replace the frequency response function, the peak value will be more obvious. The autocorrelation function formula of sequence:

$$R_{xx}(\tau)=E\left[X(t)X(t+\tau)\right] \tag{6}$$

The self-power spectral density function formula of the sequence:

$$S_{xx}(f) = \lim_{T\to}\frac{1}{T}\left|\widehat{X}_T(f)\right|^2 \tag{7}$$

$\widehat{X}_T(f)$ is a Fourier transform of $X(t)$.

Monitoring signal processing is a complex task, which requires us to master the profound theory of signal processing and compile complex programs to complete. Now there are powerful mathematical tools, commonly used mathematical software is MATLAB, which includes a lot of toolboxes, that can easily carry out signal processing. The signal must be preliminarily processed before spectral analysis.

1) The beam health monitoring signal is affected by the environment, so it is necessary to filter the signal. When analyzing the monitoring data directly, it is found that the current signal with the frequency of 50Hz is very strong, and the monitoring information of the structure is covered by it. Because the low-frequency part of structural vibration is more meaningful to the research, when analyzing the dynamic data of bridge real-time health monitoring, the digital low-pass filter is used to filter the signal. In this paper, the elliptic filter in MATLAB is used for filtering to obtain the required information.
2) The monitoring data are discrete sequences and are finite in length, so there is leakage phenomenon. The effective way to solve this problem is to add window function, which is handled by Henning window.
3) In spectral analysis, attention should be paid to the appropriate sequence length, because when the sequence is too long, the fluctuation of the spectral curve will be very large and the resolution will be high. When the sequence is too short, the spectral curve will be smooth, but the resolution will be reduced.

In this paper, the PSD function in MATLAB is used to estimate the power spectrum. In the dynamic data of bridge real-time health monitoring, there are 8 items that need spectrum analysis: the vertical acceleration of the vault, the lateral acceleration of the vault, the vertical acceleration of the upper deck, the lateral acceleration of the upper deck, the vertical acceleration of the lower deck, the lateral acceleration of the lower deck, the lateral acceleration of the suspender in the middle span and the wind speed of the vault, respectively. The wind speed of the vault has three directions: x, y and z, which can be estimated by power spectrum, and the horizontal and vertical wind angles can also be analyzed.

Before analysis, data need to be preprocessed, including de-mean, low-pass filtering and so on. By drawing the power spectrum curve, we can find the resonance frequencies, which are the natural frequencies of the structure. The resonance bees in the power spectrum curves of vertical acceleration of vault and lateral acceleration of upper and lower decks are not obvious or relatively few. This may be due to the large structural rigidity in these directions and insufficient excitation force in the environment, so that the actual vibration is not obvious and the sensor cannot detect the signal. In these power spectrum curves, the most obvious resonance peak frequency is the suspender. Because of the small shaving of the suspender, it is easy to excite the vibration of the suspender under the excitation of wind and vehicle loads. In spectrum analysis, the data measured in 1 hour are used as units.

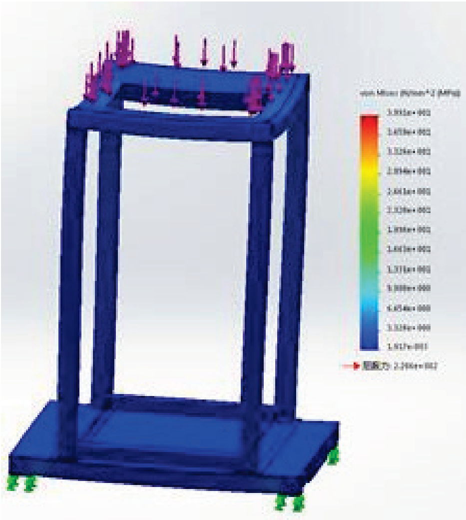
Under special climatic conditions, the vibration status of bridges is different from that of ordinary bridges, which is caused by different exciting forces. The frequencies of the resonance peaks also change slightly, mainly due to the coupled vibration between the structure and rainwater [19]. There is no vehicle load on the bridge, but the dynamic characteristics of the bridge under crowd load are different from other situations. When the upper and lower deck of the bridge act as a stand and the bridge is full of crowd loads, the frequency of resonance peaks is mainly concentrated in the range of less than 4Hz. Compared with other times, the number of resonance peaks is less.

4 EXPERIMENTAL VERIFICATION AND ANALYSIS

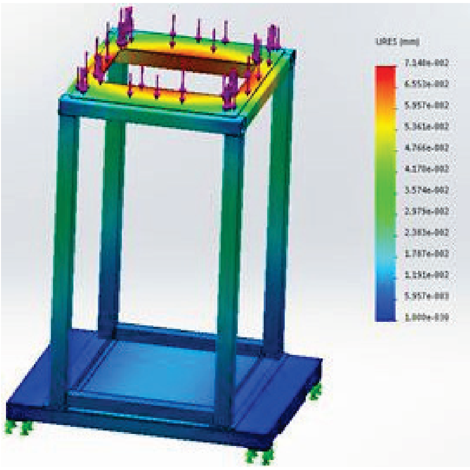
4.1 Bridge Tower Modeling

In the process of data processing and analysis of health monitoring data, it is found that the transverse vibration of bridge tower is often independent of the vibration of bridge deck (as shown in Fig. 1 (a)), such as the two-way accelerometer ACC.9 on the top of the main tower of Qianjiang Fourth Bridge. The acceleration data collected by the system are analyzed. The natural frequency component of the transverse bridge acceleration data in ACC.9 is 0.2760 Hz, which is not found in the longitudinal bridge acceleration data of the sensors ACC.1-8 and ACC.9. By modal analysis of acceleration data of a cable-stayed bridge in Tianjin, the same phenomenon is found. The transverse bridge frequency of the pylon is 1.255 Hz, which does not exist in the spectrum of acceleration measured on the bridge deck.

In order to find out the cause of this phenomenon, this section chooses the whole finite element model of Qianjiang Fourth Bridge and the model of the main tower without cable to carry out modal analysis. The finite element model is shown in Fig. 2 (a) and Fig. 2 (b). The results show that the first order transverse bridge vibration frequencies of the two finite element models are 0.2447 Hz and 0.2473 Hz, respectively. The calculated modes of the two models are basically the same. The effect of bridge deck and cable can be neglected. It can be concluded that this phenomenon is caused by the spatial arrangement of cable. Generally, in most of the bridge structures across the bridge, double cable planes are used, that is, the cables are located on both sides of the bridge deck structure. However, due to the space limitation of the main pylon of cable-stayed bridge and the stress characteristics of the cable, the angle of cable laying in space is small, and the single cable is basically in the same plane along the bridge, so the transverse vibration of the pylon has little influence on the vibration of the bridge deck. It should be noted that some modes in the transverse direction of the bridge tower are independent of the vibration of the bridge deck, but not all modes, such as some modes along the bridge that are the coupled vibration of the bridge deck and the bridge tower. Therefore, this paper can choose the main tower to do the finite element model updating alone. The aim of the updating is to simplify the finite element model updating problem of large bridge structure into a simple sub-structure model updating problem, which is independent of the vibration of bridge deck and cable-stayed cable, so as to improve the accuracy of model updating.



(a)

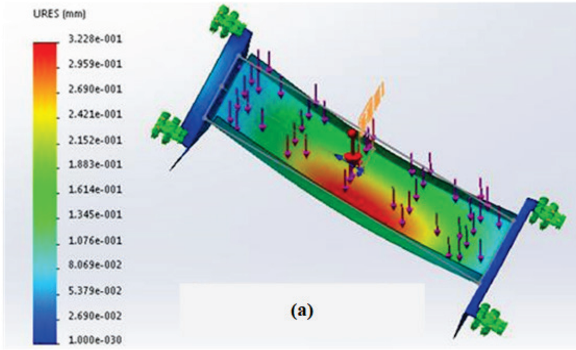


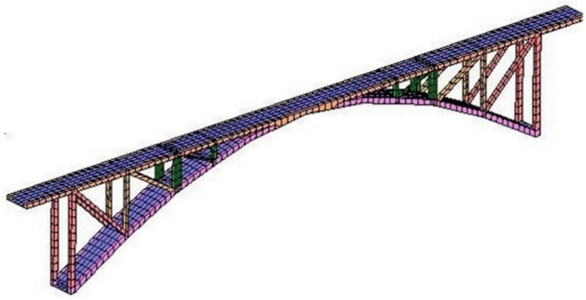
(b)

Figure 1 (a) Modal substructure characteristics of main tower of cable-stayed bridge, (b) Transverse bridge vibration of cable-stayed cables

The vibration of cable-stayed cables is also irrelevant to the vibration of bridge deck and pylon (as shown in Fig. 1 (b)). Cable-stayed cable is an important force-bearing component of cable-stayed bridge, but it has the characteristics of small flexural rigidity, light weight and poor energy dissipation. Some vibration of bridge is caused by vibration of bridge tower or deck, which is called parameter self-excitation. The parametric self-excitation of cable is a kind of large-scale divergent vibration, which occurs only when the bridge deck or tower vibrates at twice the frequency of cable. In the design of cable-stayed bridge, this vibration form must be avoided.

In order to eliminate the influence of local bending deformation of stay cables on the overall analysis, only one cable element (LINK10) is used to simulate each cable. Because the bridge deck and pylon are connected by cable-stayed cables, the influence of initial stress in cable-stayed cables should be considered. The initial stress in cable-stayed cables increases the stiffness of the bridge deck, which will affect the lateral vibration of the bridge deck (including vertical and lateral). For the boundary conditions of the bridge, the main girder and the middle tower are consolidated, the semi-floating support is at the side tower, and the bridge system restrains the vertical and transverse displacement of the bridge at the abutment and the auxiliary pier. The bottom of the pylon is consolidated and the spring element (COMBIN14) simulates the action of expansion joints and rubber bearings.





(b)

Figure 2 (a) Finite element model of main tower of cable-stayed bridge, (b) Finite element model of cable-stayed bridge

4.2 Structural Parameters

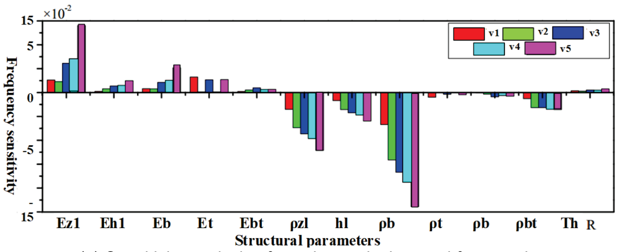
In this paper, the modal parameter analysis considering environmental factors is introduced, and the ANN model between environmental factors and modal parameters is established. The ANN model can be used to obtain the modal parameters of the structure in the closed environment, remove the influence of the main environmental factors, and obtain the probability distribution of the modal parameters caused by the randomness of the structural parameters and secondary environmental factors. Because the correlation between vibration modes and environmental factors is not very obvious, the vibration modes are taken as the mean of the test modes.

The comparison between the measured values and the calculated values of the initial model is shown in Tab. 3. From Tab. 3, it can be seen that the error between the calculated values of natural frequencies and the measured mean values ranges from −12.81% (TBK1) to −2.89% (V2). At the same time, the modal validation criterion (MAC) is used to match the identified modal and computational modal of the structure. The main reasons for the difference between theoretical and measured modal parameters are: (1) the error caused by discretization of continuous systems; (2) the uncertainty of structural geometry and boundary conditions; (3) the variability of material properties; (4) the error in the process of test and signal processing. It is generally believed that the measured modal data are more reliable than the theoretical modal data. In order to establish a sufficiently accurate dynamic model, the measured modal data are generally used in engineering to modify and adjust the theoretical model, so that some degree or some characteristics of the two models can reach a certain degree of consistency within the scope of user interest.

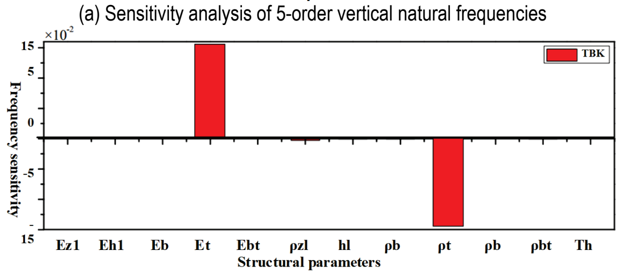
Table 3 Theory of high-way Bridge and measured free vibration frequency

Modality		Mode description	F_{ini} / Hz	MAC	Mean error
Main tower	TBK	First order transverse bending	0.2562	-	−12.92%
Deck	V1	First-order vertical bending	0.2622	99.80%	−5.92%
	T1	First-order torsional bending	0.4838	-	−7.53%
	V2	Second-order vertical bending	0.5185	99.70%	−2.98%
	V3	Third-order vertical bending	0.5921	95.70%	−6.97%
	V4	Fourth-order vertical bending	0.6728	94.30%	−8.16%
	T2	Second-order torsional bending	0.8386	91.80%	−4.41%
	V5	Fifth-order vertical bending	0.8867	86.90%	−8.01%

Ideally, all structural parameters including geometric parameters, elastic modulus, density and boundary conditions need to be corrected. However, if the revised parameters are much larger than the test results, the reliability of the finite element model will be reduced, so the revised structural parameters need to be screened. In theory, if a structural parameter has no or little influence on the objective function, they will not be selected as corrected parameters, so sensitivity analysis of structural parameters is needed when selecting corrected parameters. The variability of elastic modulus and cross-section area of stay cables is small, and the cable force has little influence on the overall dynamic characteristics of the structure. They are not used as correction parameters. Qianjiang Fourth Bridge is a newly built bridge [20]. It is assumed that the structural parameters of the main girders and pylons of cable-stayed bridges are uniform and do not change with the coordinate position. The connection between girder and cable-stayed cable, girder and pylon, girder and abutment, girder and auxiliary pier can be well controlled in the ideal state. Therefore, the above factors in the finite element model can be considered without modification. Considering the along-bridge restraint of Auxiliary Pier support, side tower support and expansion joint, the along-bridge restraint is simplified to spring restraint.



(a) Sensitivity analysis of 5-order vertical natural frequencies



(b) Sensitivity analysis of transverse bending frequency of bridge tower

Figure 3 Sensitivity analysis of natural frequency to structural parameters

Finally, the elastic modulus of the main girder (Es_l), the elastic modulus of the cross beam (Eh_l), the elastic modulus of the deck (E_b), the elastic modulus of the middle tower (E_t), the elastic modulus of the side tower (E_{bt}), the density of the main beam (ρ_{sl}), the density of the cross beam (ρ_{hl}), the density of the plate (ρ_b), the density of the middle tower (ρ_t), the density of the side tower (ρ_{bt}), the thickness of the bridge deck (T_h) are selected and the spring stiffness (R) of along-bridge constraints is simulated as a variable to be modified. The sensitivity analysis of the variables to the first five modes and the first transverse bending modes of the structure is shown in Fig. 3. From Fig. 3 (a), it can be seen that the higher order frequencies are more sensitive to the changes of structural parameters, and the structural natural frequencies are more sensitive to the mass parameters (such as ρ_{sl}, ρ_{hl}, ρ_b) than to the stiffness parameters (such as E_{sl}, E_{hl}, E_b).

For the transverse vibration frequency of the structure, only the E_t and ρ_t of the bridge tower contribute more to the change of the vibration frequency, while the other structural parameters have little effect on the change of the vibration frequency, which further verifies the characteristics of the substructure. The control interval of the structural parameters to be modified can be determined by the structural characteristics of the bridge, the estimation of the initial structural parameters and the construction drawings. For the elastic modulus of bridge tower, the variation range is 35%; for the elastic modulus of other components, the variation range is 25%; for the component density, its variability is relatively small, set to 5%; for the spring stiffness coefficient which simulates the along-bridge restraint effect, it is 50% because of its relative complexity.

4.3 Correction Result

The mean of structural parameters can be optimized by the flow chart shown in Fig. 4. The initial estimates of the structure are listed in the table below as the initial values of the above constrained optimization analysis. The convergence criteria of constrained optimization I and II and the whole optimization process are that the errors of test frequency and calculation frequency are less than 0.01 and 0.03, respectively. Generally speaking, the uncertainty of elastic modulus of concrete structure is much greater than that of density, so it can be considered that the correction results are meaningful. In this section, the standard deviation of modal parameters is used to construct the objective function. The standard deviation of structural parameters is corrected and listed in Tab. 4. The comparison between the measured and calculated values of random distributed parameters of natural frequencies is listed in Tab. 5. It can be seen from the table that the standard deviation of modal parameters is within 10%. The finite element model can better simulate the random characteristics of bridge structures.

Table 4 Initial and corrected values of standard deviation of structural parameters

Parameter	Initial	Lower limit	Upper limit	Correction value
σ_{Et}	0	0	0.06	1.282×10^{-3}
$\sigma_{\rho t}$	0	0	0.06	1.175×10^{-3}
σ_{Ebt}	0	0	0.06	5.021×10^{-3}
$\sigma_{\rho bt}$	0	0	0.06	2.689×10^{-3}
σ_{Ezl}	0	0	0.06	8.827×10^{-3}
$\sigma_{\rho el}$	0	0	0.06	1.439×10^{-3}
σ_{Ehl}	0	0	0.06	3.884×10^{-2}
σ_{hl}	0	0	0.06	1.335×10^{-2}
σ_{Eb}	0	0	0.06	4.489×10^{-2}
$\sigma_{\rho b}$	0	0	0.06	9.644×10^{-2}
σ_{Th}	0	0	0.06	1.166×10^{-2}
σ_{R}	0	0	0.06	0.144

Table 5 Comparing table for calculating and identifying natural frequencies

Modality		Identification	Finite element analysis	Error	
				Mean value	Standard deviation
Main tower	1(TBK)	N(0.2822,0.24X10-3)	N(0.2822,0.24X10-3)	−0.01%	0.04%
Deck	1(V1)	N(0.2748,0.41X10-3)	N(0.2748,0.41X10-3)	0.01%	0.01%
	2(T1)	N(0.5131,0.78X10-3)	N(0.5157,0.78X10-3)	0.45%	0.03%
	3(V2)	N(0.5284,0.48X10-3)	N(0.5356,0.47X10-3)	1.01%	3.91%
	4(V3)	N(0.6416,0.52X10-3)	N(0.6416,0.64X10-3)	−0.61%	−7.23%
	5(V4)	N(0.7311,0.68X10-3)	N(0.7053,0.65X10-3)	−2.73%	0.41%
	6(T2)	N(0.8773,0.76X10-3)	N(0.8833,0.66X10-3)	0.78%	1.17%
	7(V5)	N(0.9527,1.24X10-3)	N(0.9488,1.26X10-3)	−0.18%	0.04%

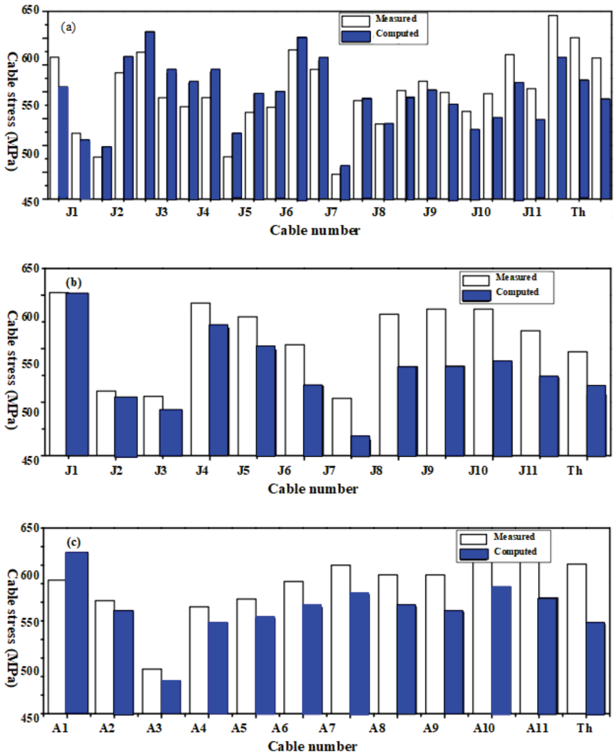


Figure 4 Comparisons of theoretical and measured cable stresses under dead weight

The change of cable force has a great influence on the static characteristics and alignment of cable-stayed bridges. Therefore, it is an important step to modify the finite element model and the dynamic characteristics of cable by modifying the initial stress of cable so that the finite element model can successfully simulate the measured cable force [21]. In this chapter, the measured cable forces are based on the load test data of the Institute of Highway Science of the Ministry of Communications. After correction, the error between the cable stress of the finite element model and the measured values is no more than 5.1%, while the error between the line shape of the bridge and the measured values is no more than 2.5 cm.

4.4 Results and Discussion

The analysis software uses the different processing methods described above to obtain different values of dead load displacement. For the above example, the free end deflection is the same as the structural mechanics formula. However, when the zero position mounting method is used, since the initial displacement of each node is zero, the final displacement value is the displacement increment from the start of its installation to the final state. For the above example, the free end deflection value is the deflection due to its weight after installation. The comparison between the calculated and measured values of the displacement increment is shown in Figure 5. It can be seen from the figure that compared with the measured displacement increment of the bridge structure under deterministic load. The corrected finite element model displacement increment calculation value is more accurate than before the correction. Most of the measured and calculated displacement increment errors are within 5%, and only the error at the center position is 8.82%. It can be seen that the modified finite element model can reflect the true stress state of the bridge structure as a whole.

The above discussion, whether it is displacement increment or modal parameters, is a reflection of structural static and dynamics as a whole. However, the failure of the bridge structure always starts from local micro-cracks. Therefore, it is necessary to perform local strain analysis on the bridge structure. Fig. 5 shows that the geometrical parameters of the cross-section of the prefabricated section of the bridge structure are small. However, the quality of the cross-bridge and the beam of the bearing is relatively large, so the variation is large. At the same time, the elastic modulus changes more than the density, so it is in the physical sense that the concrete elastic modulus has higher uncertainty than the density. The correction results are correct and reliable.

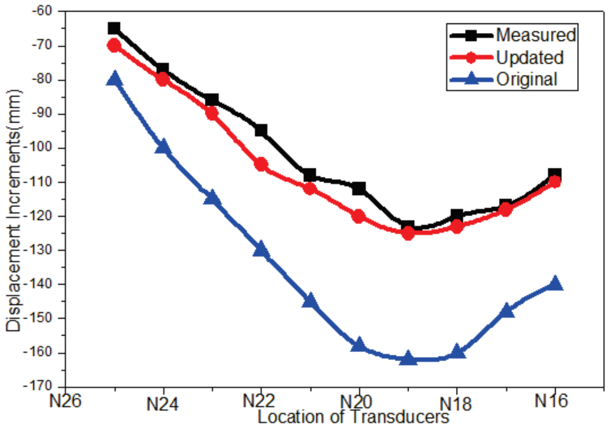


Figure 5 Finite element model calculation and measured displacement increment comparison chart

5 CONCLUSION

The efficiency of bridge structure health monitoring system mainly depends on the efficiency of data management and processing. The key to the development and effective application of structural health monitoring system is to study on-line detection and assessment method of structural damage based on health monitoring data and its implementation. The on-line detection and assessment system of structural damage should be established according to the principles of hierarchical in-depth, multi-level alarm and long-term tracking. Its primary goal is to visualize monitoring, quickly identify anomalies, and alarm at the first time. Then, the nature of abnormal information is analyzed and determined. The classification, location and assessment of structural damage are carried out. Secondary alarm is given to the damage location and extent, which provides guidance for the management department to carry out artificial flaw detection and make decision. In the process of analysis and evaluation, an accurate and effective finite element simulation is indispensable.

The contributions of this paper are as follows. Under the action of symmetric load, the strain increment of ANSYS is in good agreement with the measured value. Under the action of asymmetric load, the finite element model adopts the single main beam model, and the calculation results are basically consistent with the strain results under the symmetric load. The calculated value and the median error of the strain test on both sides of the bridge section are small, indicating that the finite element model can better simulate the strain effect of the bridge structure under vehicle load. This paper studies the uncertainty finite element model correction method based on sequential optimization strategy, and applies the uncertainty modal parameter data obtained by the health monitoring system to modify the uncertainty finite element model of the cable-stayed bridge. The finite element model can reflect the true stress state of the bridge structure as a whole. This provides a more accurate finite element model for subsequent reliability analysis. In addition, some key theoretical and computational problems related to finite element simulation and model identification for large-span bridge structures with state recognition are being studied. For example, it is suitable for the method and theoretical study of finite element model modification for large-span bridges with structural state recognition as the target.

In a word, the methods and theories of structural simulation aiming at health monitoring and condition assessment are still immature. There are many key theoretical and technical problems to be solved urgently, and some basic theoretical problems to be discussed urgently. Especially in the aspect of structural state inversion, there are many important theoretical problems. If not solved as soon as possible, it will limit the understanding of structural damage behavior and health status.

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