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Double-layer model updating for steel-concrete composite beam cable-stayed bridge based on GPS

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Abstract: In order to establish the relationship between the measured dynamic response and the health status of long-span bridges, a double-layer model updating method for steel-concrete composite beam cable-stayed bridges is proposed. Measured frequencies are selected as the first-layer reference data, and the mass of the bridge deck, the grid density, the modulus of concrete and the ballast on the side span are modified by using a manual tuning technique. Measured global positioning system (GPS) data is selected as the second-layer reference data, and the degradation of the integral structure stiffness *EI* of the whole bridge is taken into account for the second-layer model updating by using the finite element iteration algorithm. The Nanpu Bridge in Shanghai is taken as a case to verify the applicability of the proposed model updating method. After the first-layer model updating, the standard deviation of modal frequencies is smaller than 7%. After the second-layer model updating, the error of the deflection of the mid-span is smaller than 10%. The integral structure stiffness of the whole bridge decreases about 20%. The research results show a good agreement between the calculated response and the measured response.

Key words: steel-concrete composite beam; GPS; dynamic response; double-layer model updating

Theories and methods of model updating for civil engineering structures have been attractive areas of research within the engineering community both domestically and internationally, but few can be directly used in model updating for long-span bridges[1-7]. Nowadays, optimization method based on sensitivity analysis is commonly used for automatic bridge model updating[8-10]. The sensitivity-analysis-based model updating procedure generally includes the following aspects:

① Selecting measured frequencies and mode shapes as reference data;

② Selecting the updating parameters by considering the sensitivity of parameters to structural dynamic characteristics;

③ Selecting the range of updating parameters based on the physical significance;

④ Updating the initial model by modifying the updating parameters.

The sensitivity-analysis-based FE model updating method includes continuous FEM calculation and sensitivity analysis, and a large amount of computation should be done with the FEM software. Therefore, the main issue of most model updating methods in infrastructures lies in the connection between the updating program and the FEM software. A more effective model updating method for long-span bridges is imperative.

This paper aims to propose a new model updating method for large-scale cable-stayed bridges, and to put forward a stiffness degradation assessment for the whole bridge structure. The preliminary assessment can provide a theoretical basis for damage localization and maintenance of the bridge.

1 Basic Consideration for the Double-Layer Model Updating Method

The new updating process with particular attention paid to the consistency of each procedure includes five key phases: initial FE modeling, modal testing, systemic manual model tuning, GPS surveying and deep updating based on GPS, generally including the following details:

① Selecting the measured frequencies as the first-layer reference data.

② Choosing the updating parameters based on mechanical characteristics of bridge and environmental factors.

③ Analyzing the range of selected parameters based on the physical significance of updating parameters.

④ Systemically updating the initial FEM model by changing the updating parameters.

⑤ Selecting the measured GPS data as the second-layer reference data.

⑥ Updating the first-layer updated model by using the entire stiffness degradation method based on GPS data and preliminarily assessing the stiffness degradation of the whole bridge.

The effectiveness of the new approach is then confirmed by taking a composite beam cable-stayed bridge as an example.

2 FE Modeling for Updating

2.1 Simplification of bridge floor system

The Nanpu Cable-Stayed Bridge in Shanghai over the Huangpu River is taken as a case to verify the applicability of the proposed model updating method in civil engineering. Since the structure of the bridge is complicated, some simplifications are made to simulate the bridge structure according to actual situations: ① Relative slippage effects among steel main I-section girders, transverse steel beams and prefabricated concrete slabs are negligible; ② Strain distribution along the depth of a cross-section obeys the plain section assumption.

The single-girder beam element model, the double-girder beam element model and the triple-girder model are devel-

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oped to simulate the bridge deck[11-12]. The beam-truss element model can only satisfy the basic modal analysis and the static analysis according to different main girder cross-sections, but it fails to satisfy health monitoring. In this paper, both the shell element and the beam element are used to model the bridge deck. During the process, the pre-fabricated concrete slabs are simulated with shell elements, steel main I-section girders and transverse steel beams with beam elements, and the composite behavior between the slab and the girders is simulated by making use of vertical rigid links between corresponding nodes of the shell and beam elements. Fig. 1 and Fig. 2 show the configuration and simulation of the bridge deck structure.

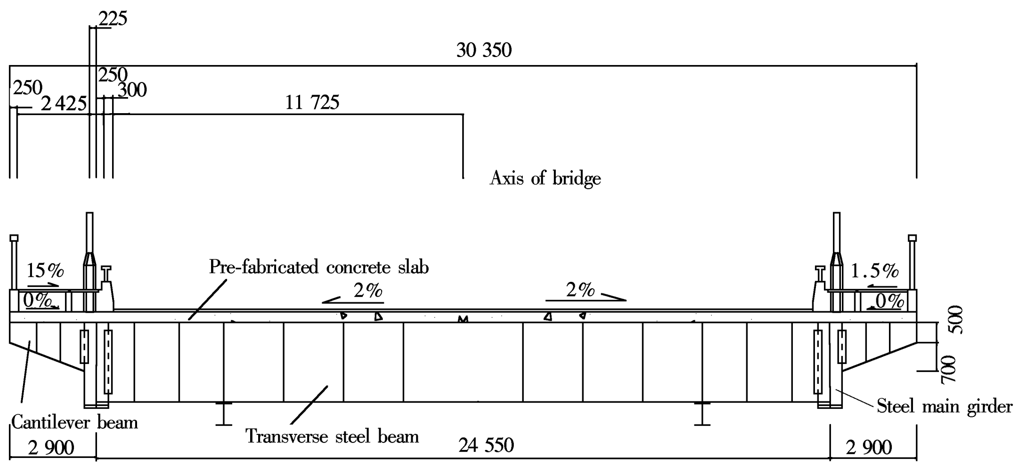

Fig. 1 General section of the girder (unit: mm)

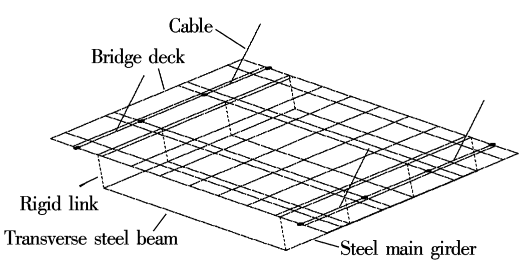

Fig. 2 Simplification of bridge deck system

2.2 Nonlinear analysis of FE model

Geometric nonlinear behavior of cable-stayed bridges originates from three primary sources: ① Cable sag effects; ② A combination of axial loads and bending moments for the girder and towers; ③ Large displacement. It is generated by the geometrical changes in the structure.

The approach used to account for the sagging of inclined cables is referred to as the equivalent modulus approach[13]. A single two-node catenary element is used to model each cable. The initial strain is set by taking the initial pretension in cables into consideration. The reference coordinate is located after deformation when a large displacement effect is taken into account, and the balance equation is established after deformation.

According to the modal analysis results shown in Tab. 1, a good agreement is shown between the calculated mode shapes and the measured ones for all the modes. For all the five modes, the mean D_f is 12.76%, and the range is from −0.6% to 24%. The standard deviation of D_f is 10.33%.

Tab. 1 Comparison of dynamic characteristics between model 1 and vibration test

Measured modes	f_m/Hz	f_c/Hz	MAC(deck only)	D_f/%
First lateral symmetric mode	0.370	0.395	0.991 3	6.8
First vertical symmetric mode	0.360	0.439	0.982 5	22.0
First vertical anti-symmetric mode	0.445	0.552	0.976 4	24.0
First torsional symmetric mode	0.525	0.586	0.882 6	11.6
First torsional anti-symmetric mode	0.665	0.670	0.796 3	−0.6

Note: f_m is the measured frequency; f_c is the calculated frequency; D_f is the frequency error.

3 First-Layer Updating Based on Structural Characterization

In this paper, updating geometries and physical parameters are chosen according to the construction process, the modeling process and the environmental factors, and the FE mode shapes are neglected since they are found to be relatively insensitive to minor changes in the model parameters. Instead, this paper focuses on the natural frequencies that are generally much more sensitive.

3.1 Variation in the mass of bridge deck

The masses of the following items are excluded from the model, which can be significant when combined: ① The web stiffeners on the longitudinal girders; ② The temporary construction works, e. g., safety barriers on the sides of the deck; ③ The mass of the wearing surface; ④ The cable anchorages above deck level.

The initial model is defined as model 1. The sensitivity of the initial model to an increase in deck mass of approximately 20% is investigated in the analysis of model 2 according to design drawings. The modal analysis results are shown in Tab. 2.

According to Tab. 2, the gap between the calculated frequency and measured frequency decreases. For all the five modes, the mean D_f is 8.6%, and the range is from −0.6% to 15.7%. The standard deviation of D_f decreases from 10.33% to 6.72%, and the range decreases. All these show that the additional mass has improved the model.

Tab. 2 Comparison of dynamic characteristics between model 2 and vibration test

Measured modes	f_m/Hz	f_c/Hz	D_f/%
First lateral symmetric mode	0.370	0.387	4.5
First vertical symmetric mode	0.360	0.410	13.9
First vertical anti-symmetric mode	0.445	0.515	15.7
First torsional symmetric mode	0.525	0.575	9.5
First torsional anti-symmetric mode	0.665	0.661	−0.6

3.2 Effect of grid density

A coarse mesh across the width of the deck is adopted in the initial model (model 1) for the sake of simplification in modeling. And the effect of the mesh resolution across the width is further examined. Model 3 is established by increasing the number of elements across the width of the deck, and the calculated results are shown in Tab. 3.

Tab. 3 Comparison of dynamic characteristics between model 3 and vibration test

Measured modes	f_m/Hz	f_c/Hz	D_f/%
First lateral symmetric mode	0.370	0.386	4.3
First vertical symmetric mode	0.360	0.438	21.7
First vertical anti-symmetric mode	0.445	0.549	23.4
First torsional symmetric mode	0.525	0.579	10.3
First torsional anti-symmetric mode	0.665	0.665	0

According to Tab. 3, the mean D_f decreases from 12.76% to 11.94%, and the range is from 0% to 23.4%. But the standard deviation of D_f increases from 10.33% to 10.37%. Therefore, the refined mesh across the width of the deck noticeably fails to improve the model.

3.3 Variation in the modulus of concrete

The elastic modulus for the concrete in the initial model 1 is based on 56-day strengths. The concrete should be exposed in the open air for six months to decrease the influence of creep and shrinkage on the bridge deck. According to the vibration test of Nanpu Cable-Stayed Bridge, field measurements were done 8 times from Oct. 1989 to Nov. 1991. The erect status of the pylon has been tested since Aug. 1990, which means that the age of the concrete varied with the height of the pylon and the length of the deck, and the concrete would have been stiffer than that assumed in the initial FE model. The variations in the elastic modulus over the extent of the cantilever were not investigated.

Model 4 is established by increasing the concrete modulus, and the calculated results are shown in Tab. 4. Although the mean D_f for all the modes increases from 12.76% to 17.54%, the standard deviation decreases from 10.33% to 7.2% and the range decreases from [−0.6%, 24%] to [7.3%, 24.9%]. The increased concrete modulus results in the higher frequencies, compared with the initial model, but they are supposed to be more accurate as they allow for the aging of concrete and reduce the standard deviation of D_f.

Tab. 4 Comparison of dynamic characteristics between model 4 and vibration test

Measured modes	f_m/Hz	f_c/Hz	D_f/%
First lateral symmetric mode	0.370	0.420	13.5
First vertical symmetric mode	0.360	0.443	23.0
First vertical anti-symmetric mode	0.445	0.556	24.9
First torsional symmetric mode	0.525	0.625	19.0
First torsional anti-symmetric mode	0.665	0.714	7.3

3.4 Effect of ballast on side span

Model 5 is established with the ballast on the side span under consideration, and the calculated results are shown in Tab. 5.

Tab. 5 Comparison of dynamic characteristics between model 5 and vibration test

Measured modes	f_m/Hz	f_c/Hz	D_f/%
First lateral symmetric mode	0.370	0.381	3.0
First vertical symmetric mode	0.360	0.438	21.7
First vertical anti-symmetric mode	0.445	0.550	23.6
First torsional symmetric mode	0.525	0.568	8.2
First torsional anti-symmetric mode	0.665	0.657	−1.2

According to Tab. 5, the mean D_f for all the modes decreases from 12.76% to 11.06%, the standard deviation increases from 10.33% to 11.11% and the range changes from [−0.6%, 24%] to [−1.2%, 23.6%]. Therefore, the consideration for ballast on the side span has not improved the model.

3.5 Final modifications

All of the above factors are taken into account in model 6. The calculated results are shown in Tab. 6.

Tab. 6 Comparison of dynamic characteristics between model 6 and vibration test

Measured modes	f_m/Hz	f_c/Hz	D_f/%
First lateral symmetric mode	0.370	0.373	0.8
First vertical symmetric mode	0.360	0.413	14.7
First vertical anti-symmetric mode	0.445	0.516	15.9
First torsional symmetric mode	0.525	0.581	10.7
First torsional anti-symmetric mode	0.665	0.694	4.4

Compared with model 1, the mean D_f decreases from 12.76% to 9.3%, the standard deviation decreases from 10.33% to 6.53%, and the range decreases from [−0.6%, 24%] to [4.4%, 15.9%], which demonstrates a marked improvement in the modeling and analysis.

4 Second-Layer Updating Based on GPS

Currently, GPS data are commonly used in bridge health monitoring for real-time warning. But few researches have been published on the connection between model updating and GPS data. In this paper, GPS data are taken as the second-layer reference data for basic model updating.

According to calculation results and engineering experience, the installation of a GPS on the bridge is shown in Fig 3. The antenna of the GPS receiver is mounted on the bridge handrail. The GPS reference station is placed on the top of a high building, which is far away from the bridge.

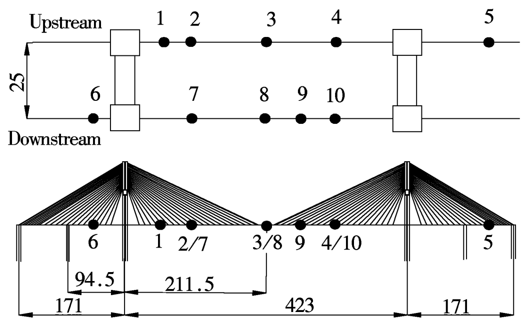

Fig. 3 Location of GPS (unit: m)

There are a large amount of data from many monitoring sites in a large monitoring system. It is difficult to visually inquire and analyze these monitored data, so the z-direction dynamic deflection of point 3 and point 8 are taken as reference data for investigating the dynamic change situation of deflection. The monitored results of points 3 and 8 are shown in Fig. 4 and Fig. 5.

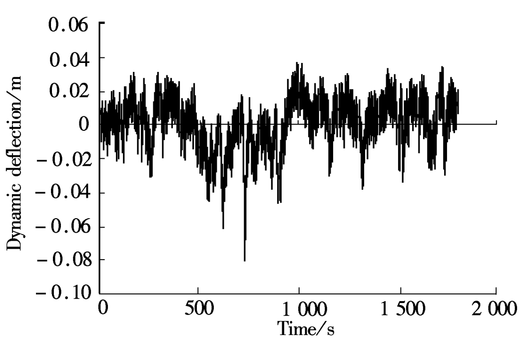

Fig. 4 Dynamic response time-history curve of point 3

The vehicle load is only taken into account for deep updating. Since the vehicles are not under control, the dynamic responses of each point do not have obvious regularity. Some simplifications should be made as follows: ① Consider the vehicle loads only but not the wind force and other environmental loads; ② Take the vehicle loads as dead loads, and consider the impact coefficient; ③ Consider the integrated action of vehicles only.

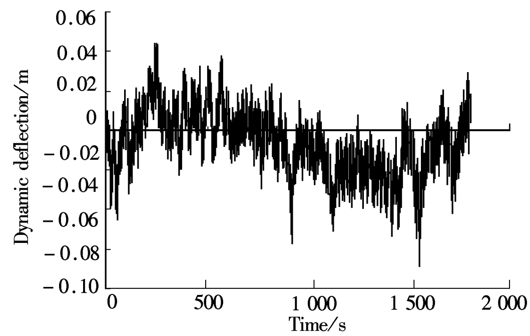

Fig. 5 Dynamic response time-history curve of point 8

The calculated and measured deflections are shown in Tab. 7. Some gaps exist between the calculated values and the measured values, and the absolute value of the measured deflection is usually larger than the calculated one.

Tab. 7 Comparison of deflection

Test point	Measured deflection/m	Calculated deflection/m
3	-0.080	-0.041
8	-0.075	-0.035

The stiffness of every part of the bridge decreases during the lifetime of the bridge, which leads to the change in the dynamic response of the bridge. The whole bridge structure is divided into four groups: tower, bridge deck, steel girder and cable. The degradation of flexural rigidity EI (E is the elastic modulus, and I is the sectional moment of inertia) of every group is taken into account for the second-layer model updating only. The calculated results are shown in Tab. 8.

According to Tab. 8, the mean D_f between the calculated peak and the measured peak is within 10%, which demonstrates the reliability of the updated model. The flexural rigidity of the whole bridge decreases by nearly 20% according to the second-layer updating results.

Tab. 8 Comparison of updated deflection

Test point	Measured peak/m	Calculated peak/m		
		Stiffness reduced by 10%	Stiffness reduced by 15%	Stiffness reduced by 20%
3	-0.080	-0.058	-0.071	-0.086
8	-0.075	-0.052	-0.065	-0.080

5 Conclusions

1) In the first-layer systemic manual updating, the updating geometrical and physical parameters are chosen with the construction process, modeling process and the environmental factors under consideration. The additional mass and the increased concrete modulus can improve on the initial model, whereas the refined mesh across the width of the deck and the consideration for ballast on the side span fail to improve the initial model evidently. Consequently, it is more reasonable to count the four factors totally so as to improve on the initial model.

2) After the second-layer model updating, the calculated dynamic response approximates the measured results. Taking the dynamic response as reference data for the second-layer model updating procedure can make the updated model applicable in the initial stiffness degradation assessment of the whole bridge structure.

3) Compared with other model updating methods, the physical significance is retained in the proposed method, and there is a good agreement between the dynamic characteristics of the updated model and the measured data.

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基于 GPS 的叠合梁斜拉桥双层模型修正方法

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摘要：为了建立实测动响应和桥梁健康状况的实质性联系，提出用双层模型修正方法修正叠合梁斜拉桥理论计算模型. 以实测固有频率为第 1 层修正目标，采用人工调节方法，在可靠范围内变化桥面板质量、网格划分密度、混凝土模量及边跨压重等参数修正模型的动力特性；然后以 GPS 实测动响应为第 2 层修正目标，通过逐步减小结构各向刚度 *EI* 模拟桥梁整体刚度损伤，采用有限元迭代的方法修正模型的挠度值. 以南浦大桥为例，对其进行有限元建模，并利用所提出的方法进行模型修正. 分析结果表明，经过第 1 层模型修正，频率误差的标准偏差小于 7%，比初始误差有明显改善；经过第 2 层模型修正，桥梁跨中挠度误差小于 10%，桥梁结构各向刚度 *EI* 下降约 20%. 研究结果表明理论计算结果与实测结果相吻合.

关键词：叠合梁；GPS；动响应；双层模型修正

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