Research Article

Sensitivity Analysis of the Influence of Structural Parameters on Dynamic Behaviour of Highly Redundant Cable-Stayed Bridges

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The model tuning through sensitivity analysis is a prominent procedure to assess the structural behavior and dynamic characteristics of cable-stayed bridges. Most of the previous sensitivity-based model tuning methods are automatic iterative processes; however, the results of recent studies show that the most reasonable results are achievable by applying the manual methods to update the analytical model of cable-stayed bridges. This paper presents a model updating algorithm for highly redundant cable-stayed bridges that can be used as an iterative manual procedure. The updating parameters are selected through the sensitivity analysis which helps to better understand the structural behavior of the bridge. The finite element model of Tatara Bridge is considered for the numerical studies. The results of the simulations indicate the efficiency and applicability of the presented manual tuning method for updating the finite element model of cable-stayed bridges. The new aspects regarding effective material and structural parameters and model tuning procedure presented in this paper will be useful for analyzing and model updating of cable-stayed bridges.

1. Introduction

In the past decade the construction of cable-stayed bridges has increased and their span lengths are growing due to improvements in design and analysis technologies. However, the complex structural characteristics of long-span cable-stayed bridges cause difficulties in understanding their dynamic behavior and make them vulnerable to dynamic loadings from phenomena such as wind or earthquakes. In recent years, many experimental and analytical investigations have studied effective factors on the dynamic behavior of cable-stayed bridges, such as natural periods, mode shapes, and damping properties [1–5]. The sensitivity analysis is a promising way to provide a tuned analytical model and assess the actual dynamic characteristics of superstructures such as cable-stayed bridges.

Sensitivity analysis is a technique to determine the influence of different properties, such as boundary conditions, damping properties, material constants, and geometrical parameters, on the structural responses. A number

of sensitivity methods for model updating purposes have been proposed for different structures [6–12]; there have been successful applications of sensitivity-based updating technology for bridges in recent years. Cantieni [13] and Pavic et al. [14] were among the first to investigate the model updating of bridges using the sensitivity method. Mackie and Stojadinović [15] conducted a sensitivity study to identify the effect of abutment mass and stiffness on seismic demand for short- and medium-length bridges. For a complex structure with high degrees of indeterminacy, such as cable-supported bridges, model updating becomes difficult because it may inevitably involve uncertainties in many parameters, for example, material and geometrical properties, and boundary conditions. Most of the recent studies have applied iterative solutions for sensitivity-based model updating of cable-stayed bridges. Zhang et al. [16] implemented finite element (FE) model updating for a 430 m main span cable-stayed bridge in Hong Kong. The updating method was an iterative eigenvalue sensitivity-based approach with lower and upper bounds for the parameter values and the degrees

of uncertainty. Brownjohn and Xia [17] successfully applied iterative sensitivity-based model updating to the dynamic assessment of a curved cable-stayed bridge in Singapore. The results of previous studies demonstrate that during an iterative procedure, it is impossible to obtain a desirable result after only one or two iterations, and it is necessary to frequently adjust the tuning strategies for a successful updating. Although most of the proposed tuning methods are automatic processes, the reasonable approximations obtained by manual methods (i.e., engineering judgment) are required to obtain the best results. Daniell and Macdonald [18] successfully applied manual tuning to a 3D FE model of one balanced cantilever section of the Second Severn Crossing (SSC) cable-stayed bridge, using data measured from ambient vibration tests during its construction. Benedettini and Gentile [19] investigated the sensitivity of the natural frequencies to changes in uncertain parameters of a cable-stayed bridge in Italy with a 70 m main span through a manual sensitivity analysis. However, the application of the manual sensitivity-based model updating technology to long-span cable-stayed bridges is still a challenge to the civil engineering community.

This paper presents a manual iterative model tuning algorithm for updating the analytical model of long-span cable-stayed bridges. The sensitivity analysis is applied to identify the effect of different parameters on dynamic characteristics of the bridge, which provides a better understanding of the nature of the structural behavior of highly redundant cable-stayed bridges. The FE model of Tatara cable-stayed bridge in Japan is generated in ANSYS 12 [20], and the sensitivity of the bridge to structural and material parameters is investigated to understand their effect on the dynamic behavior of the structure. The new and important aspects of this study can be useful as a guide for the analysis and model updating of long-span cable-stayed bridges.

2. Model Updating Procedure Based on Sensitivity Analysis

Sensitivity analysis is a common way to find effective parameters for structural responses of cable-stayed bridges, such as static displacements, mode shapes, natural frequencies, or correlation values such as modal assurance criterion (MAC) values. Conducting a sensitivity study helps to update the FE model of existing bridges and understand the structural behavior for future design of cable-stayed bridges. In recent years, different methods have been proposed to apply sensitivity-based model updating to bridges. Most of the proposed methods are automatic solutions that iteratively update selected structural and material properties to improve the correlation of model responses and test results of the bridge. However, new investigations show that during the process, it is necessary to adjust the tuning strategies frequently based on engineering judgment for a successful updating. The automatic sensitivity-based updating algorithm for cable-stayed bridges was described previously by Zhang et al. [16]. This paper investigates a manual model updating algorithm based on sensitivity study to assess the dynamic behavior of the long-span cable-stayed bridges. The natural frequencies

of selected modes are considered to compare the results. The preference for the proposed method over other model updating methods results from the possibility of using manual tuning to obtain the most reasonable results. However, the automatic model updating is also possible using the presented algorithm by applying the iteration loops in ANSYS.

The order and the main features of the updating procedure are represented as follows.

The tuning procedure requires correct selection of updating parameters and reference responses to obtain the best match between FE and measured results. The following parameters ($P_i, i = 1, \ldots, n$) are selected for sensitivity analysis:

   (i) material properties: Young's modulus, Poisson's ratio, and mass density of the deck, towers, and cables;

   (ii) geometrical element properties: spring stiffness and beam cross-sectional properties;

   (iii) lumped properties: lumped stiffness (boundary conditions) and lumped masses.

The choice of sensitive parameters from the mentioned updating parameters is the most important aspect of the process, which requires wise engineering insights. The uncertain parameters must be selected to prevent meaningless results in the FE simulation. Furthermore, the number of updating parameters should be kept small, and such parameters should be chosen with the aim of correcting recognized uncertainty in the model and ensuring the data sensitivity to them.

The sensitivity study is a promising way to find the right parameters to update. A comprehensive sensitivity study of the parameters is presented in this paper using sensitivity coefficients. The sensitivity coefficient (S_{ji}) is defined as the rate of response change with respect to a parameter adjustment:

$$S_{ji} = \frac{\partial R_j}{\partial P_i}, \tag{1}$$

where R_j and P_i represent structural response and parameter, respectively. The subscripts are $i = 1, 2, \ldots, n$ for n parameters and $j = 1, 2, \ldots, m$ for m responses. Although some researchers [19] have considered the sensitivity coefficient as the change percentage in mode frequency per 100% change in the parameter, the changes in parameters should be small and restricted to lower and higher values to achieve reasonable responses and prevent physically meaningless updated results ($L < \delta P_i < H$).

The sensitivity of the natural frequencies to variations in different parameters of a long-span cable-stayed bridge is investigated in this study. The effective parameters (P^k) are chosen considering the sensitivity coefficients. As mentioned before, the change in parameters should be restricted to lower and higher values ($L < \delta P_i^k < H$). The tolerance (ε) is defined as the difference between the analyzed natural frequencies in each iteration (f_a^k) and the measured frequencies (f_m). The algorithm of the described procedure is represented in Figure 1. In the manual model tuning, the different bounds for parameters can be adjusted in each iteration,
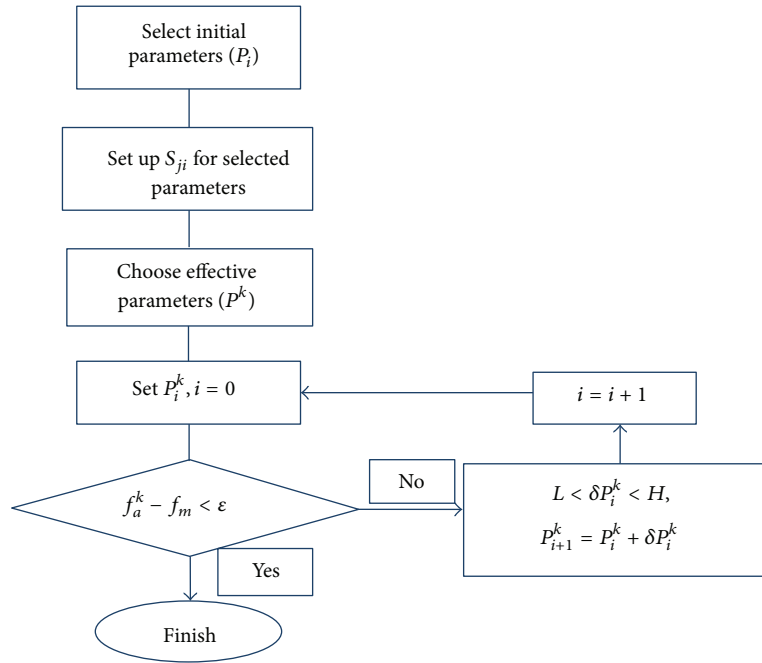
FIGURE 1: FE model tuning algorithm.

FIGURE 2: Tatara bridge.

based on engineering judgment. Moreover, choosing the changing parameters may have different priorities based on the parameter importance. The try- and -error procedure can be used for iterations. In the automatic procedure, the FE model updating starts with an iteration loop that considers reasonable tolerance for the results to converge. The manual model tuning results in better understanding of structural behavior and considers engineering considerations in long-span cable-stayed bridges.

The updated analytical models using the mentioned procedure can be used in the future for tasks such as health monitoring of the bridges.

3. Numerical Study

3.1. Bridge Description. The analytical FE model of Tatara Bridge is considered for simulations in this study. The Tatara Bridge (Figure 2) is completed in 1999 in Japan, with a main span of 890 m, and it is now the 4th longest cable-stayed bridge in the world. The bridge comprises a steel box deck 2.7 m deep with prestressed concrete (PC) girders serving as counterweights in side spans (Figure 3).

The bridge has two diamond-shaped steel towers with heights of 220 m. Figure 4 shows the schematic elevation view of the bridge. All 21 stay cables are arranged in a two-plane multifan shape and are stressed at the anchor points of the bridge deck. The stay cables have a polyethylene cable coating to resist rain vibration. Figure 5 shows the arrangement of the cables on the towers.

The bridge is modeled in ANSYS software based on the design information of the bridge. The effect of different structural and material parameters on the dynamic properties of the bridge is investigated from the analysis of a three-dimensional FE model.

3.2. Three-Dimensional FE Model of the Bridge. The geometric and inertial parameters, connections, and boundary conditions of the bridge are simulated in ANSYS software. The simplified three-dimensional FE model of the bridge is developed using elastic beam and link elements. The bridge deck is modeled by a single central spine with offset rigid links to accommodate cable anchor points (fishbone model). The BEAM4 elements are used to model the deck, towers, heads, and struts of the towers. The MPC184 elements are applied to model the rigid links, and MASS21 elements are used to include the mass of nonstructural members. The cables are modeled by tension-only LINK10 elements that have a stress stiffening capability. The cable pretensions are considered to ensure small deformations under the deck self-weight. Table 1
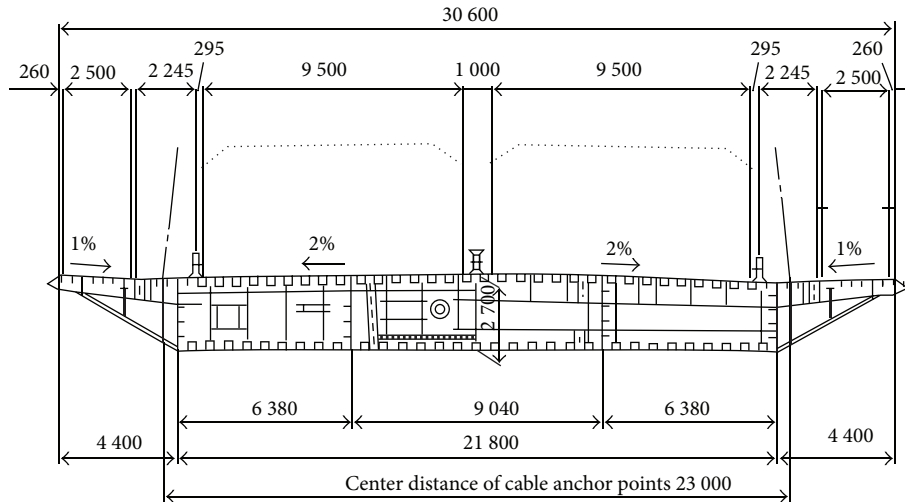
(a) Cross section of steel girder
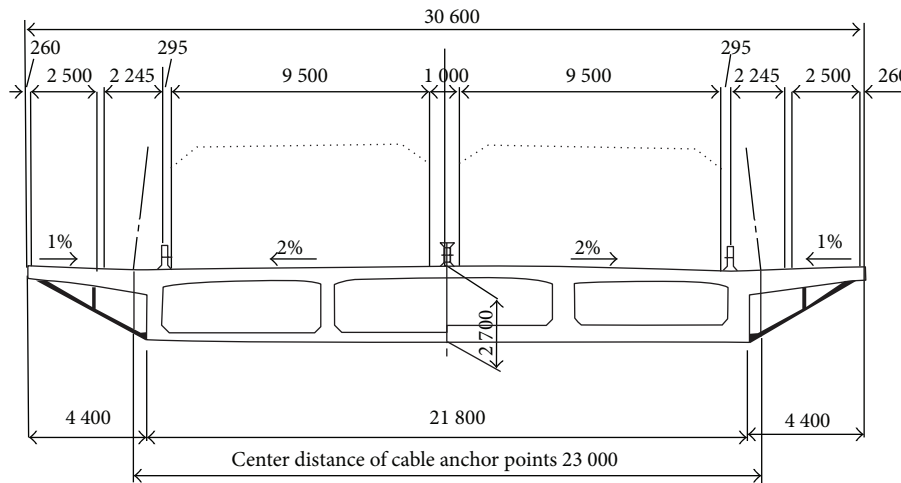
(b) Cross section of concrete girder

Figure 3: Main girder sections [21].

Table 1: Properties of structural members.

Structural members	Element type	Material	E (MPa)	ρ (kg/m³)
Tower	BEAM4, MPC184	Steel	2.10×10^5	7850
Rigid link of towers		—	—	
Girder	BEAM4	Steel	2.10×10^5	7850
Rigid link of girders	BEAM4	Concrete	3.00×10^4	2400
	MPC184	—	—	
Cable	Link10	Steel	2.00×10^5	7850

shows the properties and element types of the structural members that are used in the FE model of the Tatara Bridge.

The boundary conditions at the piers, at the base of the towers and at the connection of each abutment, are simulated in the FE model. The tower bases are considered as being fixed in all degrees of freedom. The end connections permit the end of the deck to rotate freely about the vertical and transverse axes. Rotation about the longitudinal axis (x) and two translational degrees of freedom at each abutment are fixed. Elastic bearings were used for vertical bearings on the tower-to-deck connections. Thus, the spring constant about 3.92×10^6 (N/m) is adopted for limit girder displacement in the direction of the bridge axis based on design information about the bridge. Figure 6 represents the boundary conditions assigned to the FE model of the bridge.

One of the important features of cable-stayed bridges is the dead load influence on the stiffness of the bridge. To include this influence, the static analysis under self-weight

Table 2: Comparison of natural frequencies (Hz) between field tests and FE modeling.

Mode no.	FE frequencies (HZ)	Deck component of motions			Tower component of motions			Measured frequencies	Frequency error (%)
		Vertical	Lateral	Torsional	Longitudinal	Lateral	Torsional		
V1	0.216	✓			✓			0.225	4.0
TL1	0.120		✓	✓		✓	✓	0.097	23.0
V2	0.270	✓			✓			0.263	2.7
TL2	0.320		✓	✓		✓	✓	0.248	29.0
V3	0.367	✓			✓			0.365	0.5
TL3	0.564		✓	✓		✓	✓	0.470	20.0

V indicates vertical modes; TL indicates torsional-lateral modes.
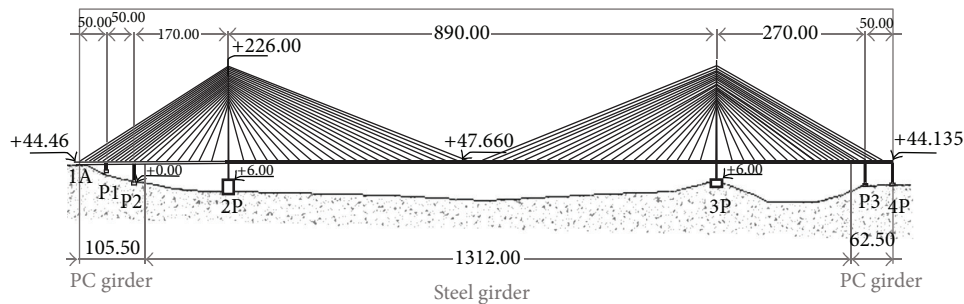

Figure 4: General arrangement of the Tatara cable-stayed bridge.

and cable pretension, in which all the structural members are prestressed, is performed before the modal analysis. The sag effect, the most important nonlinear effect, is considered to include geometrical nonlinearity in the static analysis. The results of recent studies show that nonlinearities are more significant in static analysis than dynamic analysis of the cable-stayed bridges and the most effective nonlinearity is the sag effect [4, 5, 22–25]. Following a nonlinear static analysis, the linear prestressed modal analysis is conducted to extract the natural frequencies and mode shapes of the initial FE model of the bridge.

3.3. Verification of FE Simulation Results with Field Test Results of the Bridge. The natural frequencies and mode shapes of the FE modal analysis should be verified with field test results to ensure that the structural parameters used in the FE modeling reflect the real dynamic characteristics of the bridge.

The field forced vibration test of the Tatara Bridge had been performed in 1998 when the construction of pavement was almost complete. The exciters vibrated the girder in the vertical direction to investigate three vertical bending vibration modes and in the horizontal direction to investigate three bending vibration modes in the normal direction of the bridge axis [26]. In the case of the Tatara Bridge, the test results are available only for first-mode frequencies. Thus, the results of FE modeling are verified with a few measured mode frequencies. However, the results of recent studies show that when the purpose of parametric updating is the response estimation of a long-span bridge under wind excitation, the verification of the lowest few vertical and lateral-torsional deck modes would be sufficient. It is conceived that the

response of a bridge can be quite accurately spanned by the lower modes [27]. Table 2 shows the comparison of FE calculated frequencies and identified frequencies from test results.

The other method available to compare the results of FE modeling with test results involves using the MAC value, which compares the ordinates of mode shapes and gives a value of unity for perfect correlation, while returning a value of zero for uncorrelated modes. The MAC values are not compared in this study with respect to lack of information about actual mode shapes from test results. However, the comparison of natural frequencies is considered to assess the sensitivity of the dynamic characteristics of the bridge to different parameters.

4. Model Updating Results

4.1. Sensitivity Analysis of the Parameters. Having established the basic model, the sensitivity of the natural frequencies is investigated for variations in material and structural properties of the deck, towers, and cables of the modeled bridge. Based on the natural characteristics of the bridge a maximum of 30% is considered for lower and higher change values of parameters.

The following properties are considered to investigate the sensitivity of the modal responses of the bridge:

(i) Young's modulus, self-weight (which is relative to mass density), and section properties of steel girders $(E_{d1}, \rho_{d1}, A_{d1}, I_{xx(d1)}, I_{zz(d1)}, I_{yy(d1)})$ and PC girders $(E_{d2}, \rho_{d2}, A_{d2}, I_{xx(d2)}, I_{zz(d2)}, I_{yy(d2)})$;
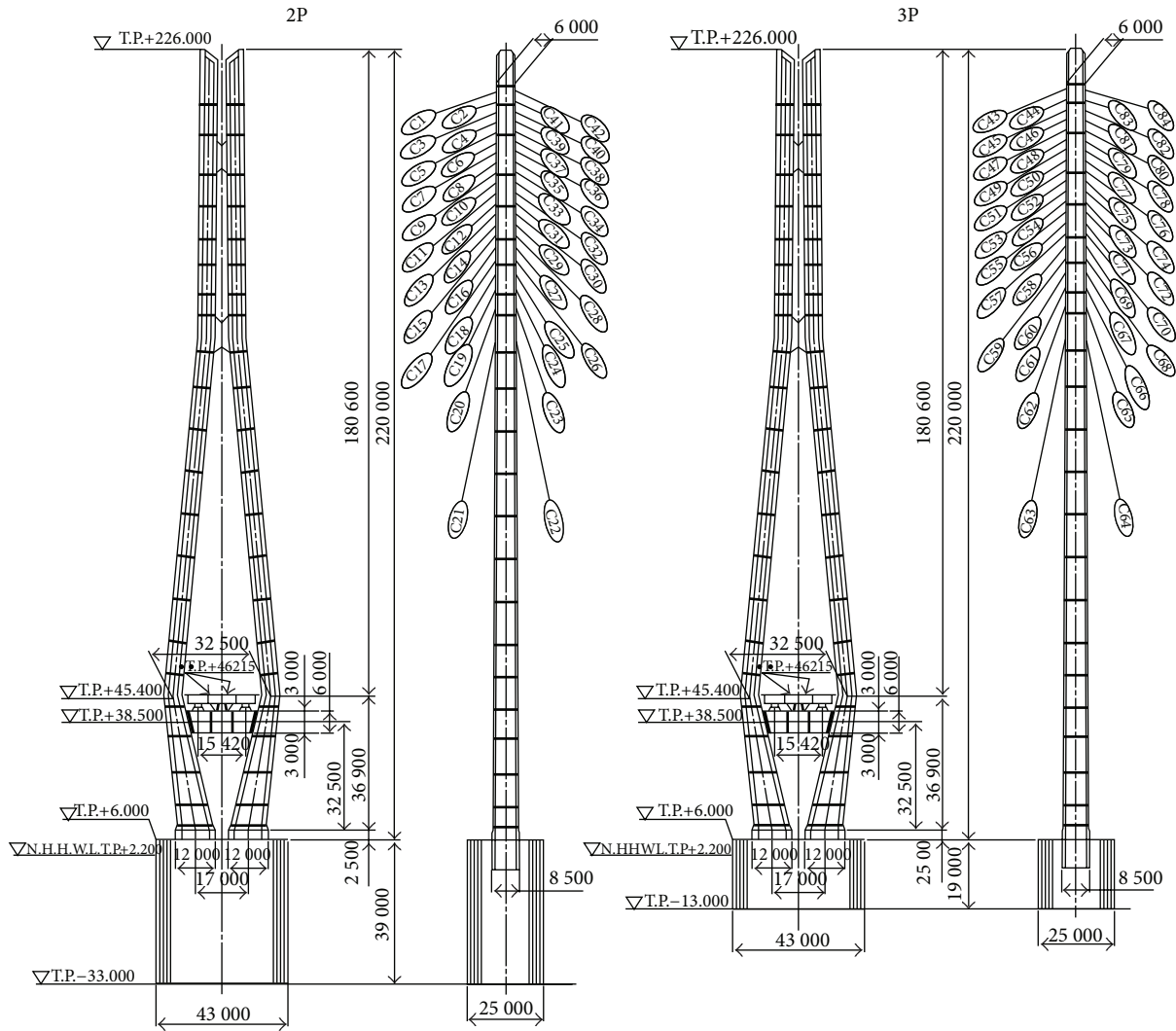
Figure 5: Main tower general arrangement [21].

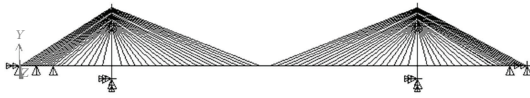Figure 6: Boundary conditions of the Tatara cable-stayed bridge.

(ii) the mass density and Young's modulus of the towers (ρ_t, E_t) and cables (ρ_c, E_c);

(iii) the constant of transverse springs in the deck-to-tower connections (K_z).

The sensitivity coefficients are defined as $(\partial R_j / \partial P_i)(P_i / R_j)$, and they represent the change in natural frequencies for an approximately 60% change in parameters (30% in each direction). The sensitivity of the model to change in the deck, tower, and cable properties is investigated in the prestressed modal analysis to compare the natural frequencies. The tensions in the cables are adjusted manually to correspond with the adjustments in deck mass.

The elastic moduli of cables are modified automatically in ANSYS to consider the sag effect.

Young's modulus and the weight of steel girders affect the bridge responses significantly, while the same parameters of PC girders have almost no effect on natural frequencies (Tables 3 and 4). It can be justified that the PC girders are much smaller than the steel girders of the main span; the PC girders perform only as counterweights and do not have any significant effect on the dynamic characteristics of the bridge. Thus, the properties of steel girders are considered for the sensitivity analysis of the deck (E_{deck}, ρ_{deck}, A_{deck}, I_{xx}, I_{zz}, I_{yy}). The inspection of the sensitivity coefficients of the first vertical and torsional-lateral modes for the parameters of steel girders (as shown in Figure 7) clearly reveals the following.

(i) The E_{deck} and ρ_{deck} change the natural frequencies inversely, while ρ_{deck} changes the first vertical modes more significantly. The change of E_{deck} primarily affects the first torsional-lateral modes of the deck.

Table 3: Comparison of natural frequencies (Hz) with changes in Young's modulus of steel girders (E_{d1}) and PC girders (E_{d2}).

E_{deck}	TL1	V1	V2	TL2	V3	TL3
$0.7E_{d1}, E_{d2}$	0.103	0.209	0.26	0.276	0.355	0.535
$0.8E_{d1}, E_{d2}$	0.109	0.212	0.264	0.293	0.359	0.547
$0.9E_{d1}, E_{d2}$	0.115	0.214	0.267	0.308	0.363	0.555
$1.1E_{d1}, E_{d2}$	0.126	0.217	0.273	0.327	0.370	0.571
$1.2E_{d1}, E_{d2}$	0.130	0.218	0.275	0.331	0.373	0.578
$1.3E_{d1}, E_{d2}$	0.135	0.220	0.277	0.333	0.376	0.585
$\mathbf{E_{d1}, E_{d2}}$	**0.120**	**0.216**	**0.270**	**0.32**	**0.367**	**0.564**
$0.7E_{d2}, E_{d1}$	0.120	0.215	0.269	0.320	0.367	0.564
$0.8E_{d2}, E_{d1}$	0.120	0.215	0.269	0.320	0.367	0.564
$0.9E_{d2}, E_{d1}$	0.120	0.215	0.270	0.320	0.367	0.564
$1.1E_{d2}, E_{d1}$	0.120	0.216	0.270	0.320	0.367	0.564
$1.2E_{d2}, E_{d1}$	0.120	0.216	0.270	0.320	0.368	0.564
$1.3E_{d2}, E_{d1}$	0.120	0.216	0.270	0.320	0.368	0.564

Table 4: Comparison of natural frequencies (Hz) with changes in mass density of steel girders (ρ_{d1}) and PC girders (ρ_{d2}).

ρ_{deck}	TL1	V1	V2	TL2	V3	TL3
$0.7\rho_{d1}, \rho_{d2}$	0.136	0.243	0.305	0.333	0.374	0.577
$0.8\rho_{d1}, \rho_{d2}$	0.130	0.233	0.292	0.331	0.369	0.571
$0.9\rho_{d1}, \rho_{d2}$	0.125	0.224	0.28	0.327	0.365	0.569
$1.1\rho_{d1}, \rho_{d2}$	0.116	0.208	0.261	0.311	0.354	0.546
$1.2\rho_{d1}, \rho_{d2}$	0.112	0.202	0.252	0.303	0.343	0.529
$1.3\rho_{d1}, \rho_{d2}$	0.109	0.196	0.245	0.294	0.336	0.514
$\boldsymbol{\rho_{d1}, \rho_{d2}}$	**0.120**	**0.216**	**0.270**	**0.320**	**0.367**	**0.564**
$0.7\rho_{d2}, \rho_{d1}$	0.121	0.216	0.270	0.320	0.367	0.564
$0.8\rho_{d2}, \rho_{d1}$	0.120	0.216	0.270	0.320	0.367	0.564
$0.9\rho_{d2}, \rho_{d1}$	0.120	0.216	0.270	0.320	0.367	0.564
$1.1\rho_{d2}, \rho_{d1}$	0.120	0.216	0.270	0.320	0.367	0.564
$1.2\rho_{d2}, \rho_{d1}$	0.120	0.216	0.270	0.320	0.367	0.564
$1.3\rho_{d2}, \rho_{d1}$	0.120	0.216	0.270	0.320	0.367	0.564

Table 5: Comparison of natural frequencies (Hz) with changes in the transverse stiffness (K_z) of bearings.

K_z	TL1	V1	V2	TL2	V3	TL3	TOW1	TOW2
$K_z = K_x$	0.067	0.216	0.270	0.14	0.367	0.238	0.342	0.434
$K_z = 2K_x$	0.075	0.216	0.270	0.154	0.367	0.248	0.342	0.434
$K_z = 10K_x$	0.100	0.216	0.270	0.224	0.367	0.352	0.342	0.434
$K_z = 15K_x$	0.105	0.216	0.270	0.248	0.367	0.435	0.342	0.430
$K = \infty$	0.120	0.216	0.270	0.320	0.367	0.564	0.342	0.427

TOW indicates tower modes.

(ii) The A_{deck} slightly affects both the vertical and torsional-lateral modes of the bridge.

(iii) The changes of I_{xx}, I_{zz} have no significant effect on mode frequencies, while the change of I_{yy} has an increasing effect on the first two torsional-lateral modes.

The sensitivity coefficients of the first deck and tower modes to the parameters of steel towers and cables are shown

Table 6: Effective parameters on the first mode responses of the bridge.

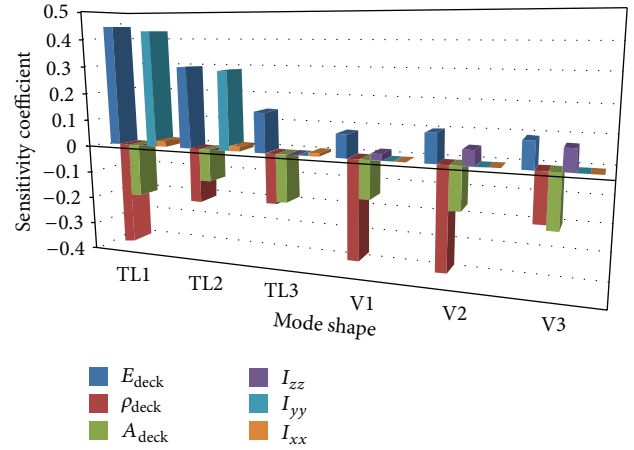| Mode	Effective parameters
TL1	$E_{\text{deck}}, I_{yy}, \rho_{\text{deck}}, A_{\text{deck}}, K_z$
TL2	$E_{\text{deck}}, \rho_{\text{deck}}, K_z, I_{yy}, E_{\text{tower}}, \rho_{\text{tower}}$
TL3	$E_{\text{deck}}, \rho_{\text{deck}}, A_{\text{deck}}, K_z$
V1	$\rho_{\text{deck}}, E_{\text{cable}}, A_{\text{deck}}, E_{\text{deck}}$
V2	$\rho_{\text{deck}}, E_{\text{cable}}, A_{\text{deck}}, E_{\text{deck}}$
V3	$E_{\text{cable}}, \rho_{\text{deck}}, A_{\text{deck}}, E_{\text{deck}}$
TOW1	$\rho_{\text{tower}}, E_{\text{tower}}, I_{yy}$
TOW2	$\rho_{\text{tower}}, E_{\text{tower}}, I_{yy}$

Figure 7: Sensitivity of modes to the parameters of steel deck.

in Figure 8. The tower properties (E_{tower} and ρ_{tower}) change the vertical and transverse deck modes slightly; however, they have a significant effect on the following tower modes. The cross-sectional areas of the towers (A_{tower}) are not considered for updating because they are parameters that cannot be updated simply. Increasing Young's modulus of the cables (E_{cable}) will considerably increase the natural frequencies of the first modes, while the mass density of cables (ρ_{cable}) does not have a significant effect on the natural frequencies.

The dynamic characteristics of the recent cable-stayed bridges prove that the deck-to-tower bearings act in all directions and using uniaxial spring elements cannot represent the dynamic behavior of the cable-stayed bridge accurately. Therefore, the transverse stiffness of the springs (K_z) is also considered to represent the actual behavior of bearings in the deck-to-tower connections. Longitudinal and transverse springs (K_x, K_z) representing the stiffness of bearings in deck-to-tower connections mainly affect the deck torsional-lateral modes. Table 5 indicates that increasing the transverse spring stiffness (K_z) results in an enhancement of the natural frequencies. The spring parameters can be freely updated, and no variation bounds are assumed because they are quite uncertain in the first application.

The results of the sensitivity analysis clearly reveal that the dynamic characteristic of a cable-stayed bridge model can be improved by changing the structural and material

TABLE 7: Comparison between calculated FE and measured frequencies.

| Modes | Measured frequencies (Hz) | FE calculated frequencies (Hz) | | | | | | Final frequency error (%) |
| | | Initial FE frequency (HZ) | Estimated frequencies | | | | | |
			First	Second	Third	Forth		
TL1	0.097	0.120	0.120	0.110	0.100	**0.100**		3
TL2	0.248	0.320	0.269	0.260	0.255	**0.250**		3
TL3	0.470	0.564	0.390	0.410	0.450	**0.463**		1
V1	0.225	0.216	0.220	0.218	0.218	**0.218**		3
V2	0.263	0.270	0.269	0.269	0.267	**0.267**		1
V3	0.365	0.367	0.367	0.370	0.370	**0.370**		1

TABLE 8: Updated parameters of the bridge.

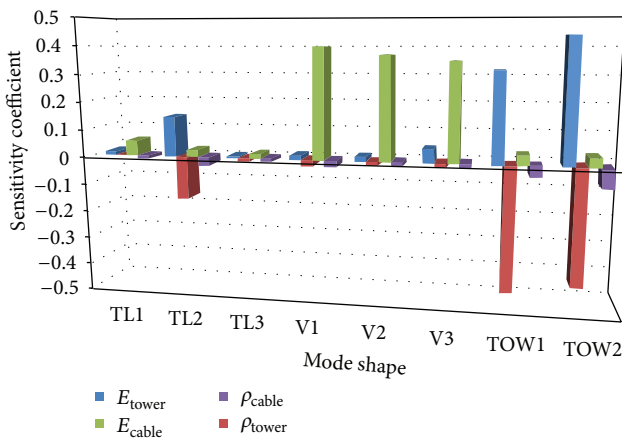| Parameters	Initial FE parameters	Updated FE parameters	Change (%)
E_{deck} (MPa)	2.10×10^5	2.15×10^5	2.38
ρ_{deck} (kg/m^3)	7850	8242.5	5
A_{deck} (m^2)	1.367	1.367	0
I_{yy} (m^4)	140	133	−5
E_{cable} (MPa)	2.00×10^5	2.05×10^5	2.5
K_z (N/m)	—	3.92×10^7	—

FIGURE 8: Sensitivity of modes to the parameters of towers and cables (TOW indicates tower modes).

parameters. In the case of the Tatara cable-stayed bridge, the most effective parameters on the first frequency modes are shown in Table 6. Based on the results of sensitivity studies, material and structural parameters can be updated to achieve the closest natural frequencies to the experimental tests of the bridge.

4.2. Manual FE Model Tuning of the Bridge. Based on the sensitivity analysis results, the FE model of the bridge is updated using the algorithm that is presented in Figure 1. The manual model tuning is considered rather than automatic methods because of the following.

(i) In manual model tuning, the selection of parameters for updating is more meaningful than other proposed

methods. In the case of the modeled cable-stayed bridge, the homogeneous properties of steel main deck and towers permit fewer changes in the mass density and cross-sectional parameters of the deck and towers in the updating procedure in comparison with concrete ones.

(ii) In the manual tuning method the restricted lower and higher values of parameters can be different during the procedure with respect to engineering judgment. The different variations in the higher and lower bounds of parameters increase the accuracy of the procedure. The deck-to-tower connection parameters, which act as bearings, can be updated according to trials and errors with no variation bounds, while other sectional properties should vary in a restricted bound to prevent meaningless results. A maximum 10% variation is considered for the elastic moduli, mass densities, and cross-sectional areas of all components. For the moments of inertia of the deck, a maximum 20% variation is considered due to the relative complexity compared with the other components.

(iii) The number of updating candidates of parameters is lower than that of other methods due to elimination of some parameters based on engineering judgment and natural characteristics of the bridge.

Despite all the advantages, the disadvantage of the manual tuning method is that the variation in selected stiffness and mass parameters cannot be very refined without an impracticable number of model variations [8]. However, the manual method is considered in this study for model updating of a long-span cable-stayed bridge considering all the mentioned advantages. Table 7 shows the different attempts at model updating of the bridge. The tuning procedure will be stopped when the differences between the measured and the calculated frequencies become less than 3% for the selected first vertical and lateral-torsional modes of the bridge. The values of updated parameters are shown in Table 8.

Using the presented manual tuning method, material and structural parameters are updated to achieve the closest natural frequencies to the experimental tests of the bridge. The updated mode shapes of the bridge are shown in Figure 9. In the case of the updated bridge model, it seems that the combinations of updated parameters represent the dynamic

1st lateral bending mode (0.100 Hz)

(a)

3rd vertical bending mode (0.370 Hz)

(b)

1st vertical bending mode (0.218 Hz)

(c)

2nd vertical bending mode (0.267 Hz)

(d)

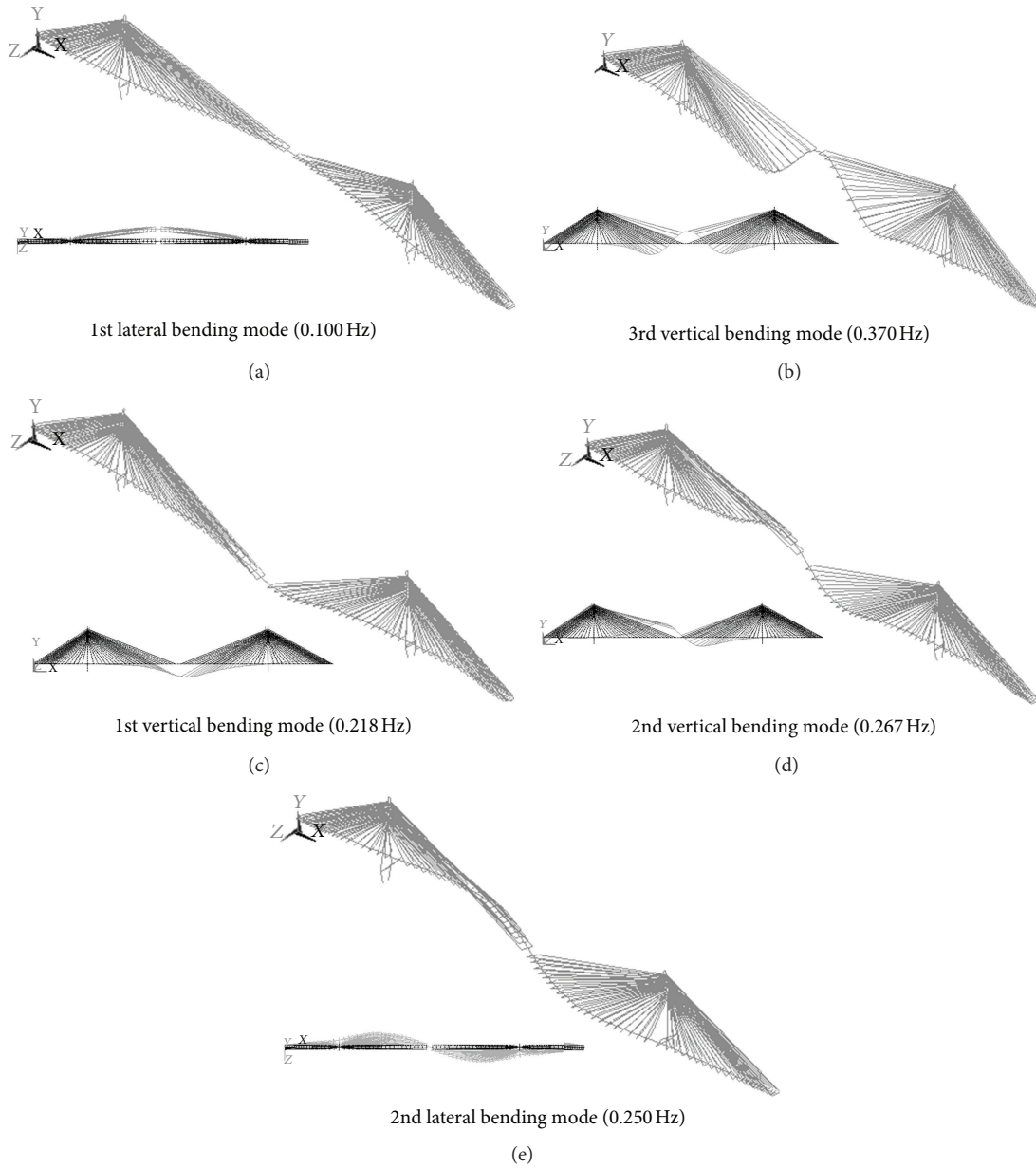2nd lateral bending mode (0.250 Hz)

(e)

Figure 9: First mode shapes from the updated FE model.

behavior of the bridge accurately, and further changes would not be effective in modeling bridge responses. The dynamic and modal characteristics of the FE model show good agreement with the field test results, and the updated model can be used as a baseline model for future uses such as health monitoring of the bridge.

5. Conclusions

A manual updating algorithm is presented in this paper for model tuning of highly redundant cable-stayed bridges based on sensitivity analysis. The following aspects are concluded from the proposed model updating procedure.

(i) Sensitivity analysis is a proper way to identify the effects of structural and material parameters on the dynamic behavior of cable-stayed bridges. The results of the sensitivity analysis on the FE model of the simulated cable-stayed bridge show that the vertical modes of the deck are mostly affected by ρ_{deck}, A_{deck}, and E_{cable}, while the lateral-torsional modes are affected by E_{deck}, $I_{yy(\text{deck})}$ and the stiffness of deck-to-tower connections. The results of the mentioned sensitivity analysis can be helpful for understanding the dynamic behavior of long-span cable-stayed bridges and future health monitoring of these superstructures.

(ii) The verification of simulation results in this study shows that in FE models of cable-stayed bridges,

the vertical deck modes are calculated more accurately than the lateral-torsional ones, which could be the result of errors in parameters across the width of the deck, assumptions made in initial modeling simplifications of nonstructural members, or defined boundary conditions and connections of the bridge. The parameters that influence lateral-torsional deck modes are the first parameters to be updated in FE models of cable-stayed bridges.

(iii) The proposed manual tuning algorithm can successfully update the FE model of long-span cable-stayed bridges. In comparison with other proposed automatic model updating methods the manual method makes more reasonable adjustments in parameters. Furthermore, selecting fewer updating parameters and assigning different lower and higher variations of parameters increase the accuracy of the tuning procedure.

Conflict of Interests

The authors do not have any conflict of interests with regard to the content of the paper.

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