Mitigating Epistemic Uncertainty in Ambient Vibration-based Structural Identification of a Concrete-filled Steel Tubular Arch Bridge

Abstract:
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He is quite familiar with the bridge test.

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27 March 2016

To the ASCE Journal of Bridge Engineering Editorial Board:

We would like to submit the enclosed manuscript entitled ‘Mitigating Epistemic Uncertainty in Ambient Vibration-based Structural Identification of a Concrete-filled Steel Tubular Arch Bridge’ for consideration for publication in ASCE Journal of Bridge Engineering.

No conflict of interest exists in the submission of this manuscript, and all authors have approved the manuscript for publication. We deeply appreciate your consideration of our manuscript, and we look forward to receiving comments from the reviewers. If you have any questions, please do not hesitate to contact me at the address below.

Thank you and best regards.

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Mitigating Epistemic Uncertainty in Ambient Vibration-based Structural Identification of a Concrete-filled Steel Tubular Arch Bridge

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1 ABSTRACT

Structural identification (St-Id) is an effective structural evaluation approach for health monitoring and performance-based engineering. However, various uncertainties may significantly influence the reliability of St-Id. This paper presents ambient vibration measurements for the development of a baseline model for a newly constructed arch bridge over Hongshui River in Guangxi, China. A full-scale experiment and numerical studies performed shortly after the construction of the bridge are discussed. Ambient vibration tests were conducted on the bridge deck and arch ribs under natural excitations such as traffic loads and wind loads. In this study, modal parameter identification was performed using random decrement (RD) technique together with the complex mode indicator function (CMIF) algorithm in the frequency domain and stochastic subspace identification (SSI) in the time domain for comparison. Dynamic characteristics such as natural frequencies, mode shapes and damping ratios were determined via operational modal analysis (OMA). Preliminary studies included the construction of a three-dimensional finite element (FE) model to obtain analytical frequencies and mode shapes. Then, the FE model of the arch bridge was tuned to minimize the difference between the
analytically determined and experimentally estimated modal properties by modifying several uncertain modeling parameters, such as material properties and boundary conditions. Three artificial intelligence algorithms including the simple genetic algorithm (SGA), the simulated annealing algorithm (SAA) and the genetic annealing hybrid algorithm (GAHA) have been proposed for calibrating uncertain parameters, as presented in the literature. The simulation results showed that GAHA exhibited the best performance among three methods and that the large-scale arch bridge could be efficiently calibrated using the hybrid strategy, which combines the advantages of the SGA and the SAA. Finally, to check the admissibility of calibration procedure, a sensitivity analysis was performed regarding the Young’s modulus of the steel girders, and the relative error on the static deformation of the bridge deck was compared after calibration. Through a detailed St-IId study with precise modeling, OMA and three artificial intelligence algorithms, the authors confirmed the applicability of the uncertainty mitigation strategies that are discussed.

KEYWORDS
Operational modal analysis; Epistemic uncertainty; Finite element model; Model calibration; Concrete-filled steel tubular arch bridge
INTRODUCTION

The characterization of long-span bridges has been receiving increasing attention in recent years, not only because of the degradation of a large number of structures and the limitations of traditional assessment approaches but also because of the increasing complexity of new bridges (Magalhães et al. 2008). Structural identification (St-Id), as proposed by Liu and Yao (Hart and Yao 1977; Liu and Yao 1978), is a systematic approach to characterizing the structural behavior of an unknown system based on input and output test data and has been adopted for numerous applications, including condition assessment and maintenance management. The St-Id paradigm seeks reliable estimates of the performance and vulnerability of a structural system through the correlation of mathematical models with experimental data. The framework involves six basic steps, including observation and conceptualization, a priori modeling, controlled experimentation, processing and interpretation of data, model calibration and parameter identification, and utilization of the model for simulations. In the third step of controlled experiment, ambient vibration tests are able to provide accurate and reliable descriptions of real structures and have the advantage of being inexpensive and avoiding disturbances to public traffic on bridge decks. A vibration-based assessment can efficiently provide accurate information concerning the actual performance of the bridge under working conditions. Meanwhile, it enables the identification of certain structural properties, particularly the stiffness (flexibility), damping, and mass (Zanardo et al. 2006).

To date, several hundred applications of St-Id to constructed systems have been reported, especially for long-span bridges. Since the late 1980s, Dr. Aktan’s research team at Drexel University has been involved in the testing of a wide range of operating bridges using operational modal analysis (OMA) (Catbas et al. 2007; Grimmelsman 2006; Pan et al. 2009; Zhang et al. 2009) as an experimental tool. These applications have included a variety of long-span bridges, including suspension bridges (the Brooklyn and Throgs Neck Bridges)
and arch bridges (the Henry Hudson and Tacony-Palmyra Bridges). Ren et al. (2004) presented an experimental and analytical modal analysis of a steel-girder arch bridge. Both the peak-picking and stochastic subspace identification (SSI) methods were used for output-only modal identification. Afterward, the finite element (FE) models were validated against the field test results. Jaishi et al. (2005) proposed a practical and user-friendly technique for updating FE models, and various objective functions were utilized in a case study of an arch bridge. Jaishi et al. (2007) presented a model updating method for a bridge under operational conditions using modal flexibility, and the updated FE model of the bridge was demonstrated to offer sufficient improvement in the modal parameters of the modes of interest. A multi-channel dynamic monitoring system was installed on a long-span concrete arch bridge (the Infante D. Henrique Bridge), and a numerical model was tuned to fit the dynamic properties identified using the frequency-domain decomposition method (Magalhães et al. 2008). Schlune et al. (2009) proposed a method to eliminate inaccurate modeling simplifications by means of manual model refinements before the estimation of parameters through non-linear optimization. Thereafter, multi-response objective functions were introduced and the proposed methodology was applied to the new Svinesund Bridge. He et al. (2009) conducted a set of dynamic tests on the Alfred Zampa Memorial Bridge (AZMB) before its official opening to the public and performed a benchmark study on modal identification using three different state-of-the-art St-Id algorithms.

Numerous mathematical model calibration methods have been employed by tuning the FE model to enhance its relevance to field test results. Among the mathematical methods used in structural engineering, direct search methods show good applicability for the identification of uncertain parameters. Generally, optimization methods that consider only the objective function and no additional information, such as gradients, are known as direct search methods. The simple genetic algorithm (SGA) and the simulated annealing algorithm (SAA) are acknowledged to offer certain advantages and are among the most widely used direct search techniques.
Furuta et al. (2014) applied a genetic algorithm (GA) to establish an optimal plan for bridge maintenance. Azamathulla et al. (2010) applied genetic programming in predicting the scour depth at bridge piers, and this approach was found to be more effective for this purpose than regression equations or artificial neural networks. Rifat Sonmez et al. (2014) proposed a hybrid strategy that combined the parallel search ability of a GA with the fine-tuning capability of the SAA to efficiently solve the resource-constrained project scheduling problem.

It is clear that the accurate modeling of constructed systems poses a challenge because of the significant epistemic uncertainties associated with the boundary and continuity conditions, intrinsic force distributions, non-linear and non-stationary behaviors, and material and cross-sectional properties. Moon and Aktan (2006) conducted a detailed review of the impact of uncertainty on the St-Id of constructed systems. Pan et al. (2011) discussed various sources of epistemic uncertainty and described mitigation approaches based on the St-Id of a long-span steel arch bridge. The effects of various modeling uncertainties on the calibrated FE model were evaluated by comparing different identification scenarios. Korhan et al. (2012) designed a physical laboratory model to simulate four key sources of epistemic uncertainty representing the primary test variables. The experimental program used a full factorial design for the investigation of these variables and was conducted independently by two experts. The results demonstrated that proven and accepted data pre-processing techniques and modal parameter identification algorithms can significantly bias OMA results when used in certain combinations under different structural and excitation conditions.

**OBJECTIVE AND SCOPE**

In this paper, the authors will discuss the epistemic uncertainties overcome in a recent application of St-Id concerning a long-span concrete-filled steel tubular arch bridge. The emphasis is placed on mitigating the modeling uncertainty and using heuristic expertise in interpreting the experimental results. An unclear
interaction between issues related to piers and bridge deck leads to difficulties in providing sufficient information regarding the boundary conditions. The bond-slip properties between steel tubes and concrete cores of the arch ribs complicate the simulation of the load-response mechanism of the real structure. Furthermore, the variable cross section of the transverse girders, the compactness of the concrete encased in the steel tubes, the uncertainties in the mechanical properties of the bridge deck and pedestrian deck associated with the construction technology, and the complex geometric cross section of the pedestrian deck introduce epistemic uncertainties during modeling, which significantly impact the reliability of St-Id. An experiment concerning a long-span arch bridge, including field testing, signal processing, FE model construction, model analysis and automatic parameter identification with the aid of an Application Programming Interface (API), is presented as an example to demonstrate the proposed methodology. Field testing, including static testing under truck loads and ambient vibration testing (AVT) under natural excitations, was conducted. The modal characteristics of the bridge were extracted using two different modal parameter identification techniques. A 3D FE model of the bridge based on the existing drawings, which were verified through an on-site inspection, was analyzed to identify its analytical characteristics. The results of the FE modal analysis were compared with those obtained from the experimental modal analysis. An objective function was formulated to use the SGA, the SAA and the genetic annealing hybrid algorithm (GAHA) to calibrate uncertain parameters in the initial FE model. All three methods were implemented in MATLAB and applied using an API to integrate the model built in the FE software Strand7 to automatically achieve multiple parameter identification. Finally, a parameter assessment was performed and the admissibility of the calibrated model was checked to validate the applicability of the entire identification procedure.

BRIDGE DESCRIPTION (STEP 1)

The Laihua Bridge (Fig. 1) is a concrete-filled steel tubular arch bridge built in 2012. It is located in Laibin
City, China, and crosses the Hongshui River; the main span of the bridge is 220 m, with a width of 32 m. The rise-to-span ratio is 1/3.5, and the arch axis coefficient is 1.543. General layout drawings of the entire bridge are presented in Fig. 2. Each cross section of the two main arch ribs consists of four concrete-filled tubes with dimensions of φ 750×20 or φ 750×16 mm. The arch ribs are connected by concrete-filled tubes with dimensions of φ 550×16 or φ 550×12 mm. All of the tubes are filled with C50 concrete. The depth of the main arch ribs varies from 5.50 m at the footing to 3.50 m at the top, with a constant width of 2.0 m. The two main arch ribs of the superstructure are connected by 10 K-type hollow steel tubes. There are 36 main suspenders, consisting of polyether sulfone steel wire ropes, vertically attached to the main arch ribs at 7 m intervals. Below the level of the floor system, 16 concrete-filled tubes (φ 800, filled with C50) are supported between the arch ribs and the bridge deck. The floor system is suspended through this structure. The floor system consists of a 320 mm thick concrete slab supported by 11 longitudinal stringers (typical W16×77, spaced at 2.7 m). The stringers are placed on transverse built-up floor beams and braced by four transverse members (typical MC 10×33.6). The typical sections of the floor beams consist of 1780 mm×16 mm webs and 50 mm cover plates. The length of the floor beams between the main wire rope suspenders is 27 m. The superstructure is supported by expansion bearings, and the arches are supported on massive concrete blocks.

FINITE ELEMENT MODELING (STEP 2)

General description

Building accurate FE models is one of the main challenges in structural dynamic analysis. The development of FE models has attracted considerable interest for many civil engineering applications, such as structural control, structural evaluation, damage detection and health monitoring (Hearn and Testa 1991). The model is of particular importance for the study of bridge structures behavior for which additional difficulties arise because of the structural modeling simplification, the uncertainty of the boundary conditions, and actual
physical property of the materials (Beolchini and Vestroni 1997; Mazurek and DeWolf 1990; Salawu and Williams 1995). Bakht and Jaeger (1990) once noted that best way for a bridge engineer to understand the shortcomings of mathematical models is to investigate the real structural behavior through field testing. Rational FE modeling strikes a balance between accuracy and calculation efficiency. To mitigate the modeling uncertainty, the geometry and member details of FE models were constructed in strict accordance with the design blueprints and field inspection.

**Bridge arch modeling**

An a priori FE model of the bridge was developed in the Midas software using the available design and construction documents and drawings. Thereafter, an element-level 3D FE model was constructed in the Strand7 analysis software, incorporating the main arch span, as shown in Fig. 3. The main structural members consisted of cables, girders, floor beams, a concrete slab and arches. The RC deck was discretized using shell elements with six degrees of freedom (DOFs) at each node. Space frame elements were used to represent the deck stringers, floor beams, verticals, handrails, crash barriers, cushion caps and arch ribs of the substructures, whereas the bracings were modeled using link elements to mimic the actual end connections. The FE model consisted of 19004 nodes; 22972 beam elements for the deck stringers, floor beams, verticals, arch ribs of the substructures, crash barriers and bracings; and 2256 shell elements for the bridge deck and pedestrian deck.

Both the cross girders and arch ribs of the bridge consist of variable sections, which were accurately simulated in the model. Boundary condition simulation is an important issue in dynamic analysis. The main arch is anchored in massive concrete blocks that are founded on rock. The soil-structure interaction was not considered in the model. Fixed bearings are used for the arches, whereas expansion bearings are used for the bridge deck.

The basic material properties of the structural members are summarized in Table 1. The model was
constructed to represent the bridge in its current as-built configuration, with the corresponding structural properties. The cables are 61 PES-7-061 type (GB/T 18365-2001) strands, which are composed of 61 wires. Each strand has a density of 20.5 kg/m and consists of high-strength wires with a diameter of 7 mm. The modeling of the stay cables in Strand7 was accomplished using 3-D tension-only beam elements. Two different modeling strategies were utilized to model the concrete-filled tubes, as shown in Fig. 4. The 1st strategy was to apply the unified modeling theory, in which the steel tube and concrete core are treated as a kind of unified material (Zhong 1994), whereas the 2nd strategy was to apply the general modeling method, in which the cross section is regarded as consisting of separate sections corresponding to the steel tube and concrete core. According to the unified theory, the elastic modulus and shear modulus of the unified material in the FE model were determined based on yield strength of steel, compressive strength of concrete, and the steel ratio as presented by Zhong et al. (2003). To compare these two arch modeling strategies, the deflections of the models were analyzed for Case 15, in which 5 trucks were placed at the 1/4 span. Generally, the FE model constructed using the general modeling strategy showed better consistency with the field test results and thus was selected for use in the subsequent sensitivity analysis and model calibration procedure.

**Sensitivity analysis**

The parameters to which the modeling results were most sensitive were identified to allow the FE model to be iteratively updated to minimize the discrepancy between the measured and simulated results, in a procedure that is typically referred to as sensitivity-analysis-based model calibration. The total modeling error accumulates throughout the entire process of FE modeling and analysis. Modeling simplifications regarding the level of detail, the choice of element types and sizes, the boundary conditions, the loading conditions, the material parameters and the material models can significantly influence the modeling accuracy. It is necessary to identify a set of parameters that can be tuned to efficiently account for the most influential uncertainties.
inherent in the a priori model. To identify these uncertain parameters, a series of parameter sensitivity studies was performed on the a priori model to examine the impact of varying specific parameters on the properties of the FE model. In general, the parameters to which such a model is most sensitive tend to be the material properties and boundary conditions (Aktan et al. 1998). The critical parameters to be analyzed in this study were selected as follows:

1. The Young’s moduli of the concrete arch ribs, bridge deck, pedestrian deck, verticals and crash barriers;
2. The Young’s moduli of the steel arch ribs and stay cables;
3. The vertical stiffness boundary conditions at the ends of the bridge deck.

Generally, a sensitivity analysis concerns an FE model representation of a physical system and represents an attempt to assess the sensitivity of the objective function to variations in uncertain parameters. Two kinds of objective functions, one concerning vertical deflections under static loading conditions and one concerning dynamic modal frequencies, as shown in Eq. (1) and Eq. (2), respectively, were employed in this study.

\[ F(x, i) = \sum (d_{ai} - d_{ei}) \]  

(1)

Here, \( d_{ai} \) represents the deflections predicted by the FE model, \( d_{ei} \) denotes the experimentally measured values, \( x \) refers to the uncertain parameter chosen for the sensitivity analysis, and \( i \) denotes the \( i_{th} \) measurement point in the full static test.

\[ F(x, i) = \sum \frac{f_{ai} - f_{ei}}{f_{ei}} \times 100\% \]  

(2)

Here, \( f_{ai} \) represents the frequencies predicted by the FE model and \( f_{ei} \) represents the frequencies calculated by the following mentioned SSI method. \( x \) refers to the uncertain parameter chosen for the analysis, and \( i \) denotes the \( i_{th} \) considered mode. To evaluate the correlations between mode shapes, modal assurance criteria (MACs), as shown in Eq. (3), were used.

\[ MAC_i = \frac{(\phi_{ai}^* \phi_{ei})^2}{(\phi_{ai}^* \phi_{ai})(\phi_{ei}^* \phi_{ei})} \]  

(3)
In which, $\varphi_{ai}$ represents the mode shape predicted by the FE model and $\phi_{ei}$ represents the mode shape identified by SSI method respectively and $i$ denotes the $i_{th}$ considered mode.

A sensitivity analysis with respect to the initial estimates of the parameters was performed for 8 influential parameters, as shown in Fig. 6. Based on the results, the 5 most sensitive parameters, as listed in Table 6, were selected for calibration. The most sensitive parameters as identified using either the static load data or the modal data were the same, and the two different objective functions showed similar trends with respect to the variation of the boundary conditions.

**DYNAMIC AND STATIC TESTS (STEP 3)**

*Ambient vibration testing (AVT)*

Modal testing on site is an efficient way to obtain reliable and accurate predictions of the dynamic characteristics of a real structure. Generally, two main types of modal testing are performed on bridge structures, namely, forced vibration testing (FVT) and AVT. In the case of long-span bridges, it is difficult to generate sufficient energy to reach the desired signal level in FVT. Moreover, AVT does not disrupt the traffic on the bridge because it uses wind and traffic as natural excitation sources, which correspond well to real operating conditions. Prior to the official opening of the bridge in June of 2013, full-scale AVT was conducted on the Laihua Bridge. Dense instrumentation layouts on the bridge deck and the arch ribs in the vertical and lateral directions were established (Table 2). An LMS Cadax 8 data acquisition system, with 8 channels, was utilized to simultaneously record the ambient vibration signals. KD12000L ultra-low-frequency accelerometers (20V/g) were installed on the bridge deck and arch ribs; of these accelerometers, 6 were moved among various measurement points, whereas the other 2 were used to establish fixed reference points (Fig. 7). Because of the limited number of data acquisition channels, 12 setups in total were utilized to cover all measurement locations. The reference points were selected according to the preliminary information
obtained in a modal analysis of the FE model to avoid placing measurement instruments on modal nodal points. Measurement points were located on each side of the span. A sampling frequency of 512 Hz was chosen, and each dataset was collected for a duration of 15 minutes. Obviously, the average signal levels for all channels in the vertical direction were approximately 3 times higher than those in the transverse direction.

**Full static load testing**

Diagnostic load testing, such as truck load tests, is an independent experimental tool in St-Id and can be regarded as complementary to global modal testing. When properly conducted, static load tests provide excellent verification of the results of AVT and serve as a valuable tool for exploring the localized characteristics of a bridge. Static load tests of the Laihua Bridge were conducted using a level gauge on the bridge deck to measure its deformation and using a general total station to measure the deflections of the arch ribs. Full static load tests were performed for 20 different cases in different configurations in total. As shown in Fig. 8, trucks with known wheel loads were positioned at the 1/4 span of the bridge, and the corresponding displacements were measured.

**DYNAMIC SIGNAL PROCESSING (STEP 4)**

**Dynamic signal analysis**

In OMA, the structure is excited by unknown input forces (such as wind, traffic, earthquakes and waves). In AVT, only output-only data are acquired, and a data quality check should be conducted first to ensure reliable OMA. The quality of the data was evaluated by visually inspecting the raw data in both the time and frequency domains, for which purpose the Fast Fourier Transformation (FFT) technique was used. The signals from each channel were examined to identify any noisy or malfunctioning sensors. Spurious responses in the time history of each channel were removed to preserve the maximum amount of acceptable response data for use.
Ambient vibration signal analysis consists of frequency-domain, time-domain and time/frequency-domain approaches. In many previous applications, there has been a number of missing modes (compared with the results of analytical models) or sporadic modes that appeared or disappeared depending on the various pre- and post-processing techniques used. To eliminate the influence of epistemic uncertainty, the results of the random decrement (RD) technique in combination with the complex mode indicator function (CMIF) were compared with the results of the SSI technique in the following analysis.

Operational modal analysis (OMA)

The basis of the CMIF method is the singular value decomposition (SVD) of a multiple-reference function (FRF) matrix as shown in Eq. 4.

\[
[H(j\omega)] = [U(j\omega)\sum (j\omega)V(j\omega)]^H
\]

Here, \([H(j\omega)]\) represents the FRF matrix, \([U(j\omega)]\) and \([V(j\omega)]\) are the singular matrixes, and \(\sum (j\omega)\) is the diagonal singular value matrix.

SSI method is based on the discrete state-space formulation which represents dynamic behavior of the system (Eq. 5 and 6)

\[
x_{k+1} = Ax_k + w_k
\]
\[
y_k = Cx_k + v_k
\]

Here, \(x_k\) is the discrete state vector; \(y_k\) is the sampled output vector; \(w_k\) is the process noise and \(v_k\) is the measurement noise. The matrix A is the state matrix, which characterizes the dynamics of the system completely by its eigenvalues and the matrix C is the output matrix, which determines how the internal states are transformed to the external world; \(k\) represents the time instant. After a SVD of matrix A, modal parameters of system can be drawn. Previous research on these two identification approaches includes studies performed by Shih et al. (1988); Phillips et al. (1998) and Peeters and DeRoeck (1998).
In this study, the complex mode indicator plot obtained using the CMIF approach and the stabilization diagram obtained using the SSI method clearly indicated consistency in the estimation of the modal frequencies (Fig. 9). For most long-span bridges, the frequency range of interest lies between 0 and 10 Hz, and this region contains most of the relevant modal characteristics. The raw dynamic vertical measurement signals visualized in both the time and frequency domains are illustrated in Fig. 10. The identified natural frequencies and mode shapes of the first 10 vibration modes in the frequency range below 5 Hz are summarized in Tables 3 and 4. The identified damping ratios are very low, which is consistent with the results of previous St-Id studies of long-span arch bridges.

**Correlation between analytically and experimentally identified modes**

The experimentally identified natural frequencies and mode shapes of the bridge below 5 Hz were compared with their analytical counterparts obtained from the FE model. The first 10 vibration modes of the Laihua Bridge were computed using the FE model. To match the identified vibration modes with the analytically identified modes, the MAC values were calculated. For each experimentally identified mode, the computed eigenmode with the highest MAC value was taken as its analytical counterpart. The natural frequencies and mode shapes determined from both the FE model analysis and the OMA are presented in Tables 3 and 4.

In this study, ambient vibration signals were measured in the vertical and transverse directions and the corresponding modal frequencies were identified. Because of the number of sensors and the different signal-to-noise ratios typically associated with different response directions, it is common to post-process datasets to obtain 2-dimensional (2D) mode shapes in each direction separately. However, strong spatial coupling was evident in the vibrations of the bridge deck and arch ribs, indicating that this common 2D post-processing approach might be not sufficient to reveal the real behavior of the structure in multiple directions. Fig. 11 shows the spatial mode coupling at 2.503 Hz, and several of the coupling modes of the
bridge deck and arch ribs are specified in Table 5.

**OPTIMIZATION METHODS AND MODEL CALIBRATION (STEP 5)**

**Simple genetic algorithm (SGA)**

The SGA is a bionic random algorithm that mimics the biological process of natural genetics and natural selection (Goldberg 1989). It is performed using a number of individuals, and adaptive solutions are propagated from one generation to the next until a termination criterion is satisfied. After optimization, these adaptive individuals may not be optimally fit but will be extraordinarily close to the true optimum solution. Generally, the individuals are binary coded and the evolution process is accomplished through selection, crossover and mutation. Computationally simple and powerful, SGAs are practical methods of searching for global optima without previous information in many problems of current interest and have been successfully applied to actual optimization problems such as space truss problems (Krishnamoorthy et al. 2002), multi-objective robot routes (Castilo et al. 2007) and vehicle routing problems (Cheng et al. 2013).

**Simulated annealing algorithm (SAA)**

The SAA is a probabilistic global optimization method motivated by the statistical mechanics of annealing in metallurgy. It is an iterative method in which an initial solution is gradually improved by making local changes with a probability that depends on temperature. The temperature is an important parameter in the optimization process, and premature convergence may occur if it is decreased too quickly. If the search is conducted more slowly, allowing inferior solutions to be accepted, local optima can be avoided. This method was first introduced by Kirkpatrick et al. (1983), and it has been widely applied to large-scale combinatorial problems such as the unit commitment problem (Mantawy et al. 1998), classical job-shop scheduling problems (Aydin and Fogarty 2002), and the detection of leakage in pipe networks (Yeh et al. 2014).

**Genetic annealing hybrid algorithm (GAHA)**
Recently, recognition of the complementary strengths of the SGA and the SAA has led to the development of a hybrid method to achieve a more efficient search when addressing complex combinatorial problems (Blum and Roli 2008). In the proposed GAHA, the optimization operators, the fitness evaluation function, and the selection and SAA integration strategies are designed to improve the convergence of the SGA. The SAA is used to allow adaptive mutations based on a cooling schedule in which the temperature is initially set high to allow for bad mutations leading to worse results and is then slowly decreased. In the early stage, the GAHA performs a parallel search with high efficiency to avoid premature convergence, and in the later stages, a fine-tuned search can be achieved using the benefits of the SAA. The primary difference between the GAHA and the SGA is the adaptive mutation strategy, as illustrated in Fig. 12, and the GAHA has been applied to many complicated engineering problems, such as global function optimization (Chen et al. 2005) and the discrete time cost trade-off problem (Sonmez and Bettemir 2012).

**Application Programming Interface (API)**

The bridge model was updated using the Strand7 software, through an API interfaced with the MATLAB application. The API technique enables user to create and calibrate model parameters in Strand7 while obtaining FE analysis results through coding in MATLAB. Another advantage of the API approach is its ability to link to MATLAB toolboxes, such as the genetic tool box. Therefore, iterative and automatic modification of the model parameters following a predefined pattern can be realized, which significantly facilitates the model calibration procedure.

**Global calibration**

Model calibration through non-linear optimization relies on three main components, namely, the choice of the parameters to be updated, the objective function and the optimization algorithm, which, in this study, were coded in MATLAB and applied with the help of an API. In the application of a general calibration algorithm,
it is assumed that all necessary field data are reliable, and an absolute percentage error on the modal frequencies, as illustrated in Eq. (2), is employed. As presented earlier in the manuscript, the sensitivity analysis revealed 5 parameters with significant relative importance, and these 5 parameters were selected as the parameters to be updated in the optimization procedure. Among these parameters, the thickness of the pedestrian deck, which is regarded as a certain parameter in the real structure, was selected for use in the calibration procedure to validate the applicability of the three optimization methods. The variation bounds for the parameters were chosen based on previous engineering judgments, resulting in variation ranges of 50% for the thickness of the pedestrian deck and 40% for all other parameters. Similarly, for all three optimization algorithms, the generation gap, the number of iterations and the maximum number of generations were defined in accordance with previous research. Three iterative optimization procedures, the SGA, SAA and GAHA, were utilized to search for the global minimum value of the objective function, and the results were compared. One important issue is the correlation of similar modes, which was performed in each generation through MAC matching. The selected parameters were estimated in each generation, and the optimization procedure was terminated when a predefined number of generations was reached. Compared with the SAA, the SGA and GAHA may be both more computationally demanding and more accurate. It is possible for the SAA to converge on an infeasible design because it begins from a random point and then works its way toward the minimum, meaning that a local minimum is more likely to be reached.

In this study, the SGA and GAHA methods were applied based on an initial population consisting of 50 individuals, with 50 generations and a generation gap of 0.9. In practical applications, the SAA requires a relatively small number of parameters, including the cooling ratio (α), the maximum number of iterations (MAXITER), the maximum number of generations (MAXGEN), and the initial and final temperatures $T_0$ and $T_f$, respectively. In this study, these parameters were defined empirically as follows: $T_0=90$, $T_f=-10$, $α=0.9$, $MAXITER=50$, and $MAXGEN=50$. The selected parameters were estimated for each generation, and the optimization procedure was terminated when a predefined number of generations was reached.
MAXGEN=100, MAXITER=8, and $\alpha=0.98$. As each generation develops, the value of the objective function for the current state is denoted by $E_i$ and the value after the application of a perturbation mechanism is denoted by $E_j$. The perturbation will be accepted with a probability $p$ given by

$$
p = \exp \left( \frac{(E_j - E_i)}{b \times \alpha^T} \right)
$$

where $b$ is a constant and $p$ is to be compared with a randomly generated number between 0 and 1. $T$ represents the temperature and is slowly reduced from one generation to the next. If $p > \text{rand}(1)$, then the perturbation is accepted. During each generation, multiple iterations (8, in this study) are performed to ensure that a superior solution can be reached. In the GAHA procedure, both algorithms function complementarily.

The evolution processes of the SGA and the GAHA are illustrated in Fig. 13.

**Identified results**

The previous sensitivity analysis revealed that the modal frequencies and static load deflections are significantly affected by the vertical stiffness at the ends of the bridge deck. In the initial model, this boundary condition was represented by separate spring-damping elements constrained in the vertical direction. Rigid links were used in the transverse and longitudinal directions to simulate the interfaces of adjacent bridge deck sections. After calibration, the bearings in the vertical direction were assumed to be pinned, which agreed well with the field test results. The second step of calibration was to update the uncertain parameters in the initial model to align with the frequencies identified via OMA. The variations in the objective function value are shown in Fig. 13, in which the solid line represents the optimal solution in each generation and the dotted line represents the average objective function value of the entire population in each generation. The ratios of the optimal value of each parameter after calibration relative to its initial design value are presented in Fig. 14; of the updated models obtained in this way, the model calibrated using the GAHA method showed much better agreement with the OMA results compared with those calibrated using the other two methods. The changes in
the selected parameters to be updated are listed in Table 6, and the final analytical frequencies after calibration are given in Table 7. The differences between the analytical and experimental frequencies for the 7 lowest-frequency modes were reduced to 1.53% after calibration, which is a significant improvement compared with the initial model. One important concern in model calibration is to check the physical meanings of the uncertain parameters against typical observations in practice. The updated values of the Young’s modulus of the concrete arch ribs decreased, whereas all other values increased, which is consistent with the possibility that the concrete in the steel tubes may not be completely compact and the fact that the dynamic modulus of concrete is larger than the static modulus.

**MODEL ADMISSIBILITY CHECK**

A model admissibility check was conducted as a validation procedure to evaluate whether the calibrated model was suitable for simulating the real structure. The assessment of the quality of a calibrated model comprises two steps. Firstly, the reliability of the changes made to the parameters of the initial FE model must be checked, and justifications for these changes must be found. Secondly, good agreement between the numerical and experimental data as well as realistic model parameters are necessary but not sufficient conditions for a physically meaningful updated model. After calibration, the changes to all uncertain parameters were found to be less than 20% of their nominal values, which is acceptable considering the epistemic uncertainties introduced by the construction technology and the simplification of the model. Moreover, the thickness of the pedestrian deck, a deterministic parameter in the real structure, retained nearly its nominal value after calibration with all three algorithms; this finding supports the reliability of the calibration procedures using the considered artificial intelligence algorithms. After calibration, the average percentage error between the analytical and experimental frequencies for the 7 lowest-frequency modes decreased to 1.53%, exhibiting an excellent correlation with the OMA results for all except the 4th mode.
Afterward, a sensitivity analysis considering the modulus of the steel girders, which was known to be nearly deterministic, was performed to check the admissibility of the calibrated models. The objective function values of the models calibrated using the 3 methods were calculated for perturbations of the modulus around its nominal value. Unlike the initial FE model, the objective function values of the calibrated models were minimized when the Young’s modulus of the steel girders was set to its nominal value. A comparison of the sensitivity analysis results for the initial model and the three calibrated models is shown in Fig. 15(a).

Moreover, the deflections at critical points on the bridge deck as obtained in the static load tests were also checked. The fully composite behavior of the bridge deck and arch ribs was simulated, and the relative error at every measurement point for Case 15 in the static load tests was determined. Generally, the relative errors between the measured and simulated deflections, which can be regarded as related to aleatory uncertainty and the measurement error on the deflections measured using the level gauge and the general total station, were reduced from 10% to 5% after model calibration (Fig. 15(b)). Hence, the model after calibration is considered to produce acceptable simulations, considering the static load and modal tests.

**CONCLUSIONS**

A complete St-Id study on a long-span concrete-filled steel tubular bridge was presented in this paper, with a focus on mitigating various uncertain parameters in the initial model. By systematically performing full-scale AVT and static load testing, the physical properties in the FE model were updated in detail through calibration using three different optimization methods. This research demonstrated the reduction of epistemic uncertainty in a series of applications in St-Id, and the main conclusions are as follows.

1. Recognizing and mitigating sources of aleatory uncertainty is critical for reliable St-Id. Therefore, after an initial 3D FE model of the Laihua Bridge was established, a careful investigation of the modeling of critical members of the bridge as well as the interaction of the bridge deck and arch ribs was performed to aid in
mitigating uncertainties from the modeling perspective. A sensitivity analysis considering the results of both static and modal tests is a powerful means of identifying highly uncertain parameters. Given the recognition that epistemic uncertainty governs the behavior of long-span bridges in St-Id, the application of an analytical process consisting of precise 3D FE modeling and field tests is helpful for the reliable St-Id of complex real structures.

(2) In this study, OMA indicated strong spatial coupling in the vibrations of the bridge deck and arch ribs of the long-span arch bridge, suggesting that traditional 2D mode shape identification might be not sufficient to reveal the real behavior of the structure in multiple directions. Although the 2D approach to designing and post-processing ambient vibration data may be common and convenient for bridges with coupled modes, it introduces uncertainties and renders the field test results incomplete.

(3) Various OMA techniques were employed to reduce the measurement errors induced by signal processing. Visual inspection of the raw data, time window selection, data averaging and modal analysis in both the frequency and time domains significantly mitigated the aleatory uncertainties, which can exert considerable influence on modal parameter identification. The results of two independently applied methods (RD+CMIF and SSI) showed excellent agreement, confirming the applicability of the AVT and OMA procedure as a whole.

(4) After model calibration using 3 different artificial intelligence algorithms (the SGA, SAA, and GAHA), the optimal values of the parameters were identified while retaining the parameters’ physical meanings. Among the updated models, the model calibrated using the hybrid method, which combines the advantages of the SGA and SAA, showed the best agreement with the AVT results and the best performance in a modal admissibility check. After calibration, the average error between the analytical and experimental frequencies was reduced to 1.53% and the average error on the static load deflections was approximately 5%.
ACKNOWLEDGMENTS

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<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s modulus (GPa)</th>
<th>Density (kg/m³)</th>
<th>Shear modulus (GPa)</th>
<th>Structural members</th>
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<tr>
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<td>15.8</td>
<td>Crash barriers, concrete arch ribs</td>
</tr>
<tr>
<td></td>
<td>44.0</td>
<td>2400</td>
<td>15.8</td>
<td>Pedestrian deck, verticals</td>
</tr>
<tr>
<td></td>
<td>68.6</td>
<td>3300</td>
<td>15.8</td>
<td>Bridge deck</td>
</tr>
<tr>
<td>Steel</td>
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<td>7850</td>
<td>79.0</td>
<td>Cables</td>
</tr>
<tr>
<td></td>
<td>206.0</td>
<td>7850</td>
<td>79.0</td>
<td>Stringers, girders, K-bracing, steel arch ribs, steel tubes, handrails</td>
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</table>
Table 2 The station positions used in each instrumentation setup

<table>
<thead>
<tr>
<th>Setup</th>
<th>Moveable stations</th>
<th>Base stations</th>
<th>Direction</th>
<th>Location</th>
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<td>4, 16</td>
<td>Vertical/lateral</td>
<td>Arch deck</td>
</tr>
<tr>
<td>Cases 3 and 4</td>
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<td>4, 16</td>
<td>Vertical/lateral</td>
<td>Arch deck</td>
</tr>
<tr>
<td>Cases 5 and 6</td>
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<td>4, 16</td>
<td>Vertical/lateral</td>
<td>Arch deck</td>
</tr>
<tr>
<td>Cases 7 and 8</td>
<td>7, 8, 9, 15, 17, 18</td>
<td>4, 16</td>
<td>Vertical/lateral</td>
<td>Arch deck</td>
</tr>
<tr>
<td>Cases 9 and 10</td>
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<td>4</td>
<td>Vertical/lateral</td>
<td>Arch ribs</td>
</tr>
<tr>
<td>Cases 11 and 12</td>
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<td>4</td>
<td>Vertical/lateral</td>
<td>Arch ribs</td>
</tr>
<tr>
<td>Mode number</td>
<td>Experimental frequencies (Hz)</td>
<td>Analytical frequencies (Hz)</td>
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<td></td>
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<td>-------------</td>
<td>-------------------------------</td>
<td>-----------------------------</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SSI</td>
<td>RD+CMIF</td>
<td>Strand7</td>
<td>Error (%)</td>
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Table 4 Comparison of calculated and experimentally identified mode shapes

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<th>No.</th>
<th>Strand7 mode shape</th>
<th>SSI mode shape</th>
<th>CMIF mode shape</th>
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<td><img src="image23" alt="SSI mode shape" /></td>
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<tr>
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<tr>
<td>Order number</td>
<td>Vertical vibration of the bridge deck (Hz)</td>
<td>Transverse vibration of the bridge deck (Hz)</td>
<td>Vertical vibration of the arch ribs (Hz)</td>
</tr>
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<td>--------------</td>
<td>------------------------------------------</td>
<td>---------------------------------------------</td>
<td>----------------------------------------</td>
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<td>2.808 / / / / /</td>
<td>2.876 / / / / /</td>
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Table 6. Parameters of the FE model before and after calibration using the GAHA algorithm

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<th>Parameter updated</th>
<th>Initial Value</th>
<th>Updated Value</th>
<th>Change (%)</th>
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</thead>
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<tr>
<td>Modulus of concrete arch ribs (MPa)</td>
<td>$37.4 \times 10^3$</td>
<td>$35.0 \times 10^3$</td>
<td>-6.37</td>
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<tr>
<td>Modulus of steel arch ribs (MPa)</td>
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<td>$239.9 \times 10^3$</td>
<td>5.88</td>
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<tr>
<td>Modulus of bridge deck (MPa)</td>
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<td>$81.5 \times 10^3$</td>
<td>18.83</td>
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<tr>
<td>Modulus of pedestrian deck (MPa)</td>
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<td>$55.2 \times 10^3$</td>
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<tr>
<td>Thickness of pedestrian deck (m)</td>
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<td>0.2</td>
<td>0.00</td>
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**Table 7** Analytically identified natural frequencies before and after updating the model

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Initial analytical frequencies (Hz)</th>
<th>SGA (Hz)</th>
<th>Relative error (%)</th>
<th>SAA (Hz)</th>
<th>Relative error (%)</th>
<th>GAHA (Hz)</th>
<th>Relative error (%)</th>
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<tbody>
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<td>0.685</td>
<td>0.690</td>
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<td>1.401</td>
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<td>1.408</td>
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<td>1.434</td>
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<td>1.678</td>
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<td>2.494</td>
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<td>2.497</td>
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<td>0.24</td>
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<td>7</td>
<td>2.743</td>
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<td>4.571</td>
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<td>4.574</td>
<td>0.84</td>
</tr>
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Fig. 15. (a) Comparison of the sensitivities of the initial model and the models calibrated using the SGA, the SAA, and the GAHA with respect to the Young’s modulus of the steel girders; (b) relative errors between the measured and simulated deflections for the four models
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Fig. 2 Structural layout plan of the Laihua Bridge: (a) elevation view, (b) plan view, and (c) detailed cross-sectional drawings
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\(E_c, E_s, E_b, E_p\) and \(D_p\) denote the moduli of the concrete arch ribs, the steel arch ribs, the bridge deck, and the pedestrian deck and the thickness of the pedestrian deck, respectively.
Fig. 15 (a) Comparison of the sensitivities of the initial model and the models calibrated using the SGA, the SAA, and the GAHA with respect to the Young’s modulus of the steel girders; (b) relative errors between the measured and simulated deflections for the four models.
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