Ambient vibration based evaluation of a curved post-tensioned concrete box-girder bridge

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X-H. Chen

The University of Auckland, Auckland, New Zealand.

P. Omenzetter

The University of Aberdeen, Aberdeen, UK.

S. Beskhyroun

The University of Auckland, Auckland, New Zealand.

ABSTRACT: This paper describes ambient vibration based evaluation of a curved, post-tensioned, concrete, box-girder bridge, the Newmarket Viaduct. The procedure includes ambient vibration testing, system identification, finite element modelling and finite element model updating. Since the dynamic excitations were not measured in the ambient testing, two operational modal analysis methods, namely enhanced frequency domain decomposition and stochastic subspace identification, were applied to identify the experimental dynamic modal characteristics. A three dimensional finite element model of the bridge was created to determine the dynamic characteristics analytically. Analytical and experimental dynamic modal characteristic were compared with each other and the finite element model of the bridge was updated by changing the material properties and boundary conditions to reduce the differences between the experimental and analytical results. It is demonstrated that the proposed procedure can successfully identify the most significant modes of the bridge and the in-situ material properties and boundary conditions.

1 INTRODUCTION

Finite element (FE) analysis of important engineering structures such as highway bridges is now commonly performed in the design or reassessment process. With the advances in numerical modelling, the FE models are first built in the design phase based on technical design data, as built drawings, on-site geometry surveys and professional engineering knowledge to predict structural behaviour, both static and dynamic. However, material properties, boundary conditions and section properties accepted in the analysis are subject to inherent stochastic variability, can change as a result of construction errors and will deteriorate with time due to aging and damage. Therefore, the performance and structural behaviour of bridges in service have to be determined with the help of field testing or experimental measurements. The experimental results can later be used to check the construction quality and validate or update numerical models so that these models can better reflect the as-built, in-situ structural stiffness, boundary conditions and structural connectivity (Farrar & James III 1997).

There are several typical ways for conducting field experiments, ranging from the traditional static and quasi-static loading tests with weight-controlled heavy trucks, the forced vibration tests through the use of the eccentric mass exciter or the electrohydraulic shaker, free vibration tests using a pendulum device, test vehicle, or by imposing the initial displacements, to the ambient vibration tests using traffic or natural forces such as wind, waves or earthquake (Catbas et al. 2007). Ambient vibration techniques for the measurement of global parameters (i.e. natural frequencies, mode shapes and damping ratios) that characterize the dynamic behaviour of bridges have unique advantages over other experimental approaches, including static and dynamic free or forced vibration load tests. These are low cost, easiness of execution (Ren et al. 2004), lack of disturbance to traffic during the test (Cunha et al. 2001), ability of simultaneous evaluation of motion in different directions, long duration of excitation and frequency content suitable for long-span and flexible bridges (Hsieh et al. 2006).

The data recorded from ambient vibration tests and the resulting modal parameter estimates are used for establishing correlations with numerical predictions or, in some cases, developing and updating of FE models (Brownjohn et al. 2003; Jaishi & Ren 2005; Gentile 2006; Daniell & Macdonald 2007; Lin et al. 2009; Ribeiro et al. 2012) to (i) supply information to experimental databases from which analytical methods adopted in the design of new similar structures can be improved or evaluated, (ii) assist in the evaluation of structural integrity after the occurrence of an extreme event, (iii) accurately quantify structural parameters that dictate the level of safety and reliability, and (iv) check if structural performance is within expectations (Salawu & Williams 1995).

The objective of this study is to develop an ambient vibration based procedure for evaluation of a newly built, curved, post-tensioned, concrete, box-girder bridge, the Newmarket Viaduct. Ambient vibrations tests were performed and experimental dynamic characteristics were extracted using enhanced frequency domain decomposition (EFDD) and stochastic subspace identification (SSI) methods. A three dimensional FE model of the bridge was created using the SAP2000 FE software. The FE model of the bridge was updated by refining the boundary conditions and changing the material properties to eliminate the differences between analytical and experimental dynamic characteristics.

2 BRIDGE DESCRIPTION

The original Newmarket Viaduct, constructed in 1966, was one of the major and most important bridges within the New Zealand (NZ) road network and also the first post-tensioned, balanced cantilever bridge in NZ. It was a part of the country's busiest road link and the average daily traffic at the beginning of the 21st century was approximately 160,000 vehicles. After over 40 years in operation, deficiencies in the design and construction (no consideration on differential temperature effects, poor crash barriers and low resistance), on-going structural deterioration of the viaduct and the continuing pressures on the motorway network capacity have all contributed to the decision for the old viaduct to be replaced with a new structure.

The replacement works started in June 2009. The new viaduct is a post-tensioned concrete bridge, comprising two parallel, twin bridges. The Southbound Bridge was constructed first and opened to traffic at the end of 2010; this was followed by the construction of the Northbound Bridge completed in January 2012. Now, both bridges are opened to traffic. The total length of the bridge is 690 m, with twelve different spans ranging in length from 38.67 m to 62.65 m and average length of approximately 60 m. The bridge has two horizontal curves with radius of 760 m and 690 m, and also a slight vertical curve. There are 24 columns and the height of the intermediate piers varies from 4.14 m to 21.78 m (Figure 1). At the abutments and some interior supports (PH to PM), bridge deck is supported on bi-directional elastomeric seismic devices (Figure 2). For the other supports (PB to PG), the bridge bent bearing were fixed in all directions.

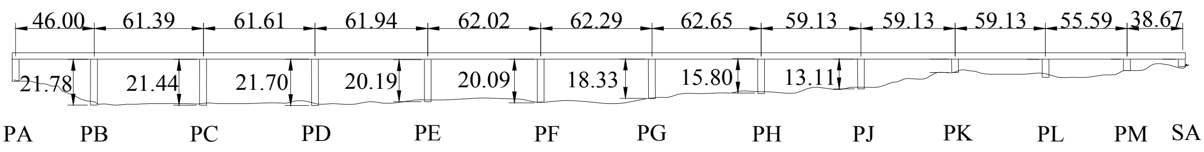

Figure 1. Newmarket Viaduct elevation (all dimension in m).

Figure 2. Seismic isolation bearing at Newmarket Viaduct.

3 AMBIENT VIBRATION TESTING AND MODAL IDENTIFICATION

3.1 Ambient vibration testing

The ambient vibration testing reported on herein was conducted on the Southbound Bridge on November 29 and 30, 2011 under operational conditions and did not interfere with the normal flow of traffic over the bridge as the testing personnel worked exclusively inside the box girder. During the test the wind speed was always very low and the ambient temperature change measured by a sensor attached to a girder web was only 1º, and the maximum change of structural temperature indicated by the embedded monitoring sensors was only 2º (Chen & Omenzetter 2013). The accelerometers used for the test were two models of wireless USB accelerometers produced by the Gulf Coast Design Concepts (www.gcdataconcepts.com): X6-1A and X6-2. The measurements points chosen for placing the accelerometers were on both sides of the bridge girder. The accelerometers were 'lightly' glued to the internal surface of the bridge deck using silicone adhesives (Figure 3). A total of 188 locations (14 for the span 1 and 12, 16 for all other spans) inside the bridge girders were chosen (Figure 4). Five test setups were used to cover the planned testing locations of the Southbound Bridge. As a good selection of the reference locations is very important to ensure the accurate identification of all the important mode shapes, the tests were carried out with several reference locations in different spans during all the test setups that were performed, while the remaining accelerometers were moved to different points. Five reference nodes (15, 82, 109, 141, and 144) were measured by using seven accelerometers. Thus, in each test setup, seven fixed reference accelerometers and a maximum of 45 roving accelerometers were used. The sampling frequency was 160 Hz, value that is imposed by the filters of the acquisition equipment and that is much higher than necessary for this multi-span bridge, where the most relevant natural frequencies are below 8 Hz. The raw measurement data from node 60 in the vertical and transverse directions are presented in Figure 5 as an example to characterize the levels of vibration observed during the ambient vibration tests.

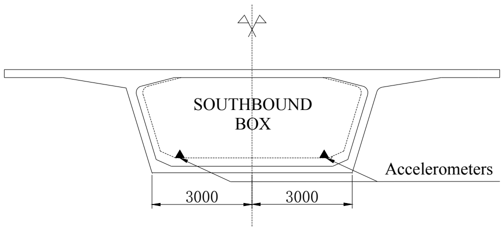

Figure 3. Accelerometers locations inside the box girder (left) and an accelerometer in operation (right).

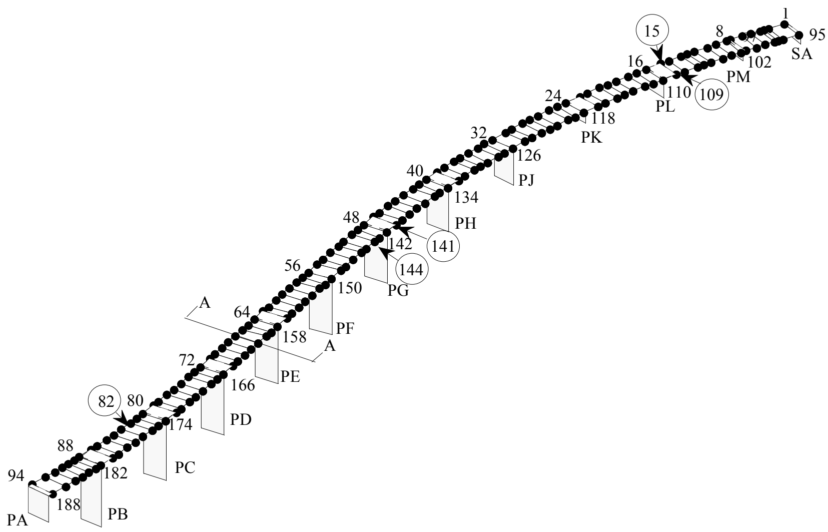

Figure 4. Accelerometers locations in Southbound Bridge deck (encircled numbers indicate reference

nodes).

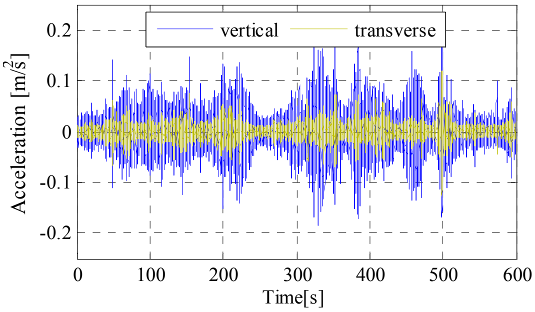

Figure 5. Example of acceleration time series collected at node 60.

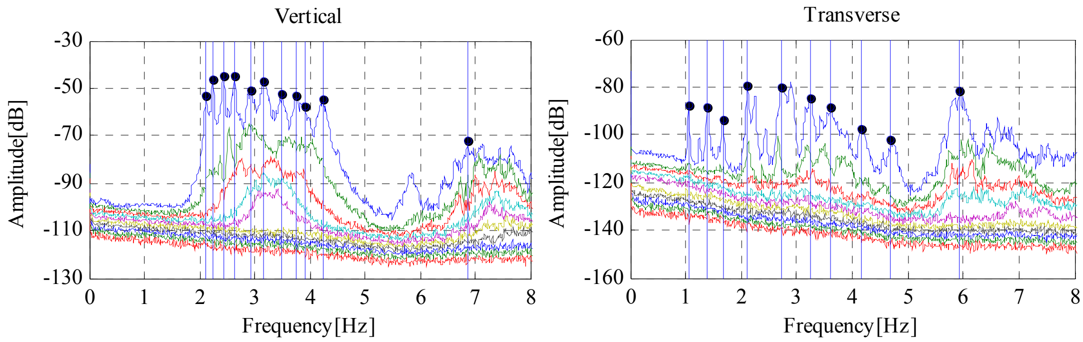

Figure 6. Singular values of spectral density functions of accelerations.

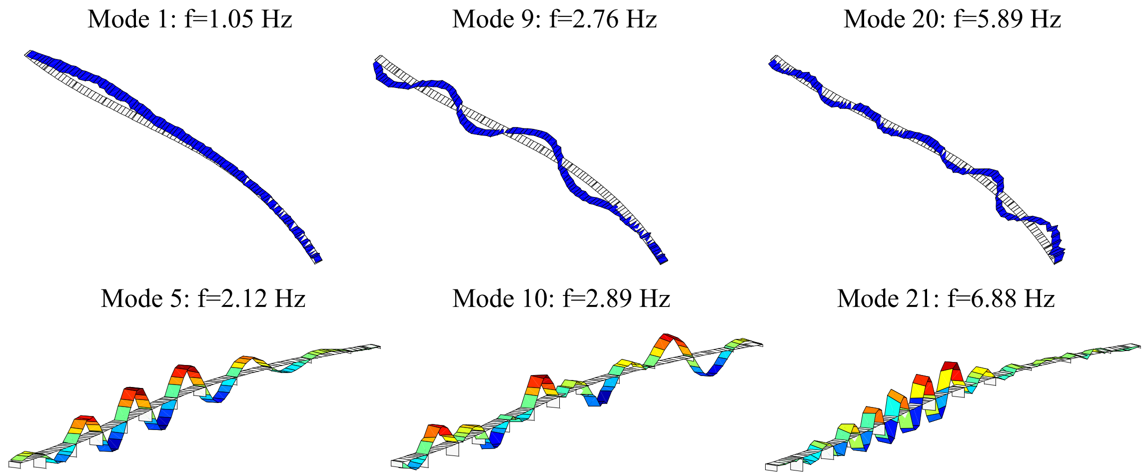

Figure 7. Selected transverse (top) and vertical (bottom) bending mode shapes identified by EFDD.

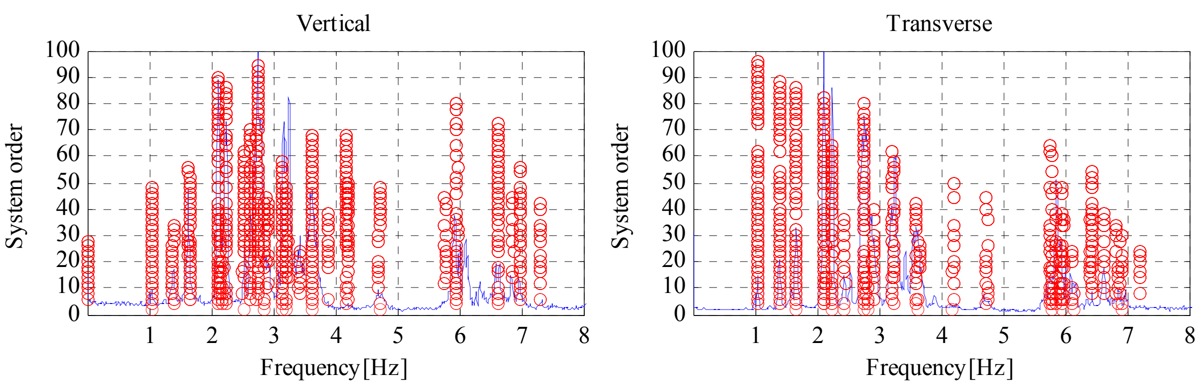

Figure 8. Stabilization diagrams of accelerations.

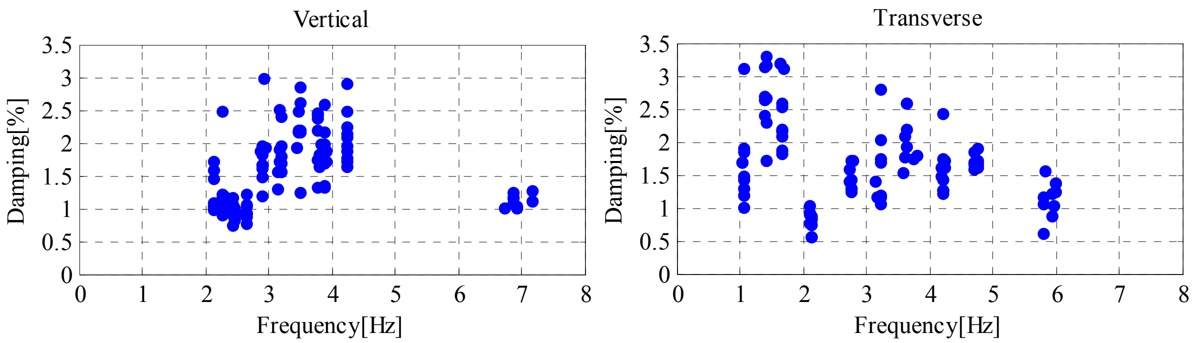

Figure 9. Dispersion of estimated modal damping coefficients.

Table 1. Comparison of experimental and FE results

Mode	Mode Type	EFDD	SSI		FE Frequencies [Hz]	
		F [Hz]	F [Hz]	ζ [%]	Before Update	After Update
1	Transverse	1.05	1.05	1.7	0.93	1.02
2	Transverse	1.39	1.40	2.6	1.12	1.32
3	Transverse	1.66	1.66	2.5	1.36	1.64
4	Transverse	2.11	2.11	0.8	1.68	2.10
5	Vertical	2.12	2.12	1.2	2.01	2.16
6	Vertical	2.25	2.26	1.2	2.14	2.22
7	Vertical	2.44	2.43	1.0	2.35	2.43
8	Vertical	2.64	2.64	1.0	2.60	2.66
9	Transverse	2.76	2.75	1.4	2.15	2.66
10	Vertical	2.89	2.90	1.8	2.83	2.86
11	Vertical	3.18	3.17	1.8	3.10	3.17
12	Transverse	3.21	3.20	1.6	2.66	3.31
13	Vertical	3.48	3.48	2.2	3.40	3.49
14	Transverse	3.62	3.65	2.0	3.40	3.70
15	Vertical	3.76	3.77	2.0	3.68	3.79
16	Vertical	3.88	3.89	1.9	3.89	3.88
17	Transverse	4.19	4.21	1.6	4.21	3.99
18	Vertical	4.21	4.23	2.1	4.17	4.24
19	Transverse	4.70	4.67	1.7	4.72	4.86
20	Transverse	5.93	5.90	1.2	5.89	5.68
21	Vertical	6.88	6.93	1.1	6.68	6.86

3.2 Modal identification processes and results

The data processing and modal parameter identification were carried out using an in-house system identification toolbox written in MATLAB (Beskhyroun 2011). The system identification techniques

EFDD and SSI are both adopted in this toolbox. For EFDD method (Jacobsen et al. 2007), the estimates of the natural frequencies were obtained by the analysis of singular values (Figure 6) calculated from the spectral matrices associated with the performed testing setups. This allowed the identification of 10 transverse and 11 vertical natural frequencies. Column 3 of Table 1 presents, for each mode, the mean of the 10 frequencies identified from different data segments. The standard deviations associated with the identified natural frequencies were very small (between 0 and 0.05 Hz). Figure 7 shows some selected mode shapes identified by using EFDD. Among these modes, the fundamental transverse mode has larger component in the spans 3 and 4 close to the higher pier PD, and the first vertical mode demonstrates the largest vertical motion in span 7. There was no torsional mode identified in the considered frequency band. For the SSI method (Van Overschee & De Moor 1996; Katayama 2005), stabilization diagrams, like the two presented in Figure 8, were used to distinguish the spurious modes from the physical ones. Figure 9 shows the ten estimates of frequencies and modal damping coefficients for the 10 transverse and 11 vertical identified modes. The variations of the natural frequencies are bigger than for the EFDD method and vary between 0.004 Hz (in the 1st vertical mode) and 0.166 Hz (in the 9th transverse mode), whereas the modal damping coefficients have a significant dispersion, with standard deviation between 0.1% (11th vertical mode) and 0.6 % (1st transverse mode). The dispersion shows how challenging it is to achieve reliable estimates of damping. By averaging the natural frequencies and the modal damping ratios identified, columns 4 and 5 of Table 1 present the mean of the identified modal parameters. The transverse and the vertical bending mode shapes are very similar to the ones identified with EFDD and therefore are not presented.

4 STRUCTURAL MODELING

4.1 Preliminary FE modelling

An FE model of the Newmarket Viaduct was constructed using the SAP2000 software. The complexity of the viaduct and the spatial character of its geometry suggested the construction of the preliminary 3D FE model of the bridge. Special attention was paid to the reproduction of the geometry, particularly the correct definition of the transverse and longitudinal profile of the deck, the description of bearing devices, and the position of the piers. Deck and all the piers were represented using solid elements. Average size of FEs for the deck structure and piers is of the order of 0.80 m and 1.00 m in the transverse and longitudinal direction of the bridge, respectively. A refined FE mesh was adopted in the upper region of the pier to model the girder-pier connections. The bearings were modelled using link elements. The nominal value of the stiffness provided by the manufacturer was assigned to each link element. The stiffness of pavement and crush barriers was lumped into concrete stiffness. Fixed boundary conditions were specified at the base of the piers, ignoring soil-structure interaction effects on the dynamic of the bridge system. It should be noted that only the Southbound Bridge is included in the model as the two bridges were not connected when performing the ambient tests.

Young's modulus of concrete was determined by testing 100×200mm cylinder specimens that were cast during the construction of the deck slab and piers. Based on these tests and also the analogous tests conducted by the contractor, the average value of Young's modulus of concrete was 36 GPa, both for deck and piers. The compressive strength of concrete was 60 MPa and the mass density was 2550 kg/m^3. A distributed mass of 5000 kg/m was also considered in the FE modal analysis to take into account the additional dead loads (asphalt, pipeline, barriers, etc.).

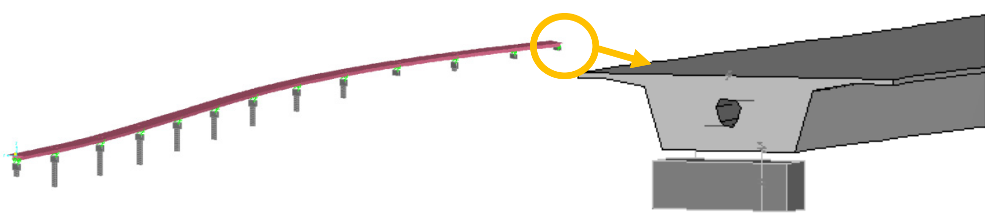

Figure 10. 3D FE model: whole Southbound Bridge (left) and deck cross section at abutment SA (right).

4.2 Model refining and updating

When the numerally and experimentally identified dynamic characteristics of the Newmarket Viaduct are compared with each other, it is seen that there is a good agreement between mode shapes but a larger difference between nature frequencies. Natural frequencies associated with the preliminary FE model are lower than the corresponding experimental values. For example, the numerical values are around 0.93-1.68 Hz for the first four transverse modes, while the experimental values are ranging between 1.05 Hz and 2.10 Hz. In addition, experimental frequencies of the deck bending, especially in transverse directions are underestimated by about 10-21 %. These differences suggest that the preliminary FE model of the viaduct should be significantly improved.

The fact that experimental frequency values of the lower horizontal vibration modes are underestimated suggests that underestimates of the stiffness properties may be present in the preliminary FE model of the viaduct. An extended series of numerical investigations showed that the cause of these large differences is primarily due to an underestimation of the shearing stiffness of the isolators supporting the deck. A numerical study have been done for the estimation of the shearing stiffness of the isolators based on the theoretical approach developed by Bedon and Morassi (2014) and Ntotsios et al. (2009). Numerical simulations based on error minimization of the natural frequencies for the first eight vibration modes, suggest that the effective shear stiffness of the isolators under low amplitude ambient excitation is about 3-4 times higher than the nominal value. Although the satisfactory results can be calculated by the refined FE model, a FE updating of the bridge was performed also by changing the material properties to further eliminate discrepancies. FE model updating results showed an increase of Young's modulus of approximately 20% in the deck and piers. Comparison of the analytical and experimental dynamic characteristics of the Newmarket Viaduct before and after FE model updating is given in columns 6 and 7 of Table 1, and it can be seen that maximum differences in the natural frequencies are reduced on average from 21% to 5%.

5 CONCLUDING REMARKS

In this paper, dynamic characteristics of the Newmarket Viaduct were determined using FE analyses and ambient vibration tests. Five different test setups were performed under normal traffic load. Several vibration modes were identified from the ambient vibration tests using the output-only measurements in the frequency range of up to 8 Hz. It is thus demonstrated that the ambient vibration response measurements are sufficient to identify the most significant modes of such a large concrete bridge with confidence. The results from the two methods applied for extracting the dynamic characteristic, EFDD and SSI, show that a very good agreement was found between the natural frequencies and mode shapes obtained by both methods. A 3D dimensional FE model of the bridge was formulated using the SAP2000 software. When comparing the numeral and experimental results, it was seen that there was a rather large difference between the natural frequencies predicted by the preliminary FE model and obtained through two experimental methods, and the numeral frequencies were underestimated. To eliminate those differences, the FE model of the bridge was refined and updated by tuning the boundary conditions and material properties. After the refining and updating, the maximum difference between the natural frequencies was on average reduced to 5%. Good agreement was finally found between the natural frequencies and mode shapes obtained from the updated FE model of the bridge and experimental measurements. The experience has shown the ambient vibration based evaluation procedure can successfully obtain essential information (the most significant modes of the bridge) to provide guidance on the validation of a mathematical model of the structure and improve the knowledge of a bridge, with the purpose of evaluating its structural performance during life span.

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