Evaluation of dynamic properties of Infante D. Henrique Bridge

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ABSTRACT: The Douro valley at Porto is crossed by several outstanding bridges linking the cities of Porto and Gaia, like the centenary metallic Maria Pia and Luiz I Bridges or the concrete Arrábida and S. João Bridges. Recently, a new crossing was constructed and open to road traffic in the vicinity of these well-known bridges: the Infante D. Henrique Bridge. The evaluation of the most relevant dynamic properties of the bridge, after completion, was developed by the Laboratory of Vibrations and Monitoring of FEUP, using both experimental and numerical tools. This study allowed to achieve an experimentally validated finite element model of the bridge, which may constitute a baseline model for future damage detection studies. In this context, the present paper describes the main features of the developed work and presents the obtained results, stressing the excellent correlation achieved between measured and calculated dynamic properties.

1 INTRODUCTION

The Douro valley at Porto is crossed by several outstanding bridges linking the cities of Porto and Gaia, like the centenary metallic Maria Pia and Luiz I Bridges, designed by Eiffel and Teophile Seyrig, respectively, or the concrete Arrábida and S. João Bridges, designed by Edgar Cardoso.

On March 2003, a new crossing open to road traffic in the vicinity of these well-known bridges: the Infante D. Henrique Bridge (Fig. 1). This bridge was conceived and designed under the leadership of António Adão da Fonseca, of AFA - Consultores de Engenharia, SA and of José Antonio Fernández Ordóñez and Francisco Millanes Mato, of IDEAM, SA. It is a bridge with an extremely shallow and thin arch “flying” 280 m over the Douro river with a rise of 25 m, at the height of almost 75 m.

The extreme shallowness and flexibility of the arch implied great complexity of construction and required a very complete control of geometry, deformations and forces. This was performed by three separate instrumentation systems, one for concrete elements, another for the temporary stay cables, and another for the granite massives. In this framework the Laboratory of Vibration and Monitoring of FEUP (VIBEST, www.fe.up.pt) was contracted to regularly measure the tension installed in the provisional stay cables using the vibration chords theory (Fonseca et al. 2002).

Complementary, after completion of the bridge, the same laboratory performed an ambient vibration test of the bridge to evaluate its most relevant dynamic properties. The results provided by the test were used to update a finite element model of the bridge, which may constitute a baseline model for future damage detection studies.

The numerical modelling involved the discretization of the structure in a 3D beam finite element mesh, with appropriate boundary conditions, while the experimental identification of modal parameters (natural frequencies, vertical, lateral and torsional mode shapes, and modal damping ratios) was done on the basis of efficient output-only system identification techniques.

In this context, the current paper describes the main features of the developed ambient vibration test and numerical modelling and presents the obtained results, stressing the excellent correlation achieved between measured and calculated dynamic properties.

Figure 1. Aerial view of the Maria Pia, Infante D. Henrique and Luiz I bridges.

2 DESCRIPTION OF THE BRIDGE

The Infante D. Henrique Bridge is composed of two mutually interacting fundamental elements: a very rigid prestressed reinforced concrete box beam, 4.50 m height, supported by a very flexible reinforced concrete arch, 1.50 m thick, as shown in the elevation and cross-sections represented in Figure 2. The span between abutments of the arch is 280 m long and the rise until the crown of the arch is 25 m, thus with a shallowness ratio greater than 11/1. In the 70 m central span, the arch joins the deck to form a box section 6 m height. The arch has constant thickness and its width increases linearly from 10 m in the central span up to 20 m at the abutments (Fonseca & Mato 2005). The high stiffness of the deck in relation to the slenderness of the shallow arch turns the arch bridge into a beam bridge defined between arch abutments and with intermediate elastic supports 35 m apart.

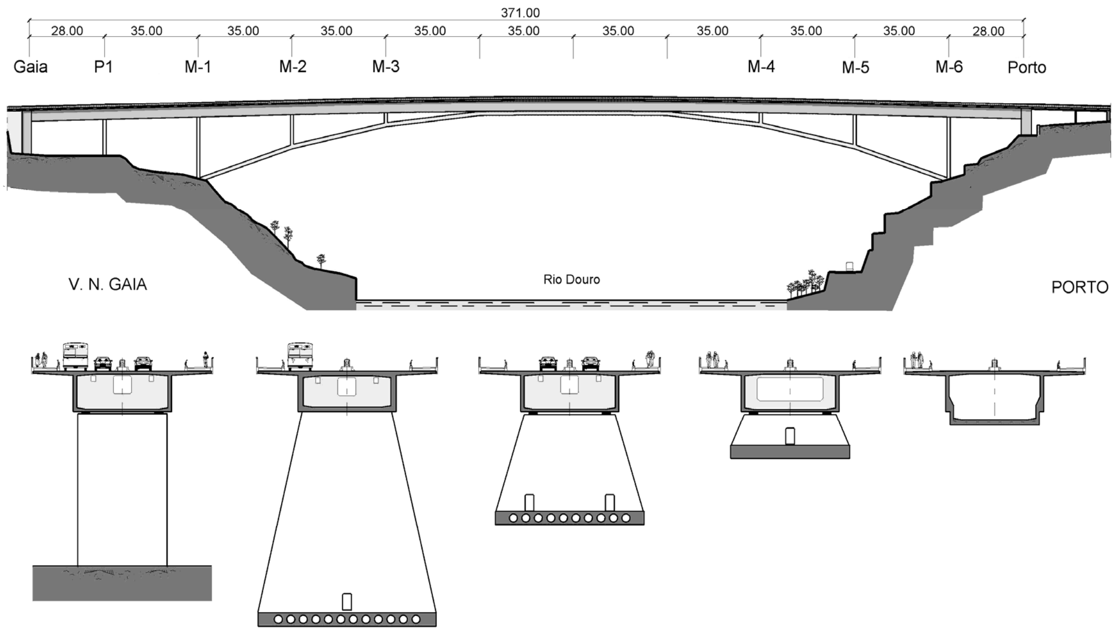


Figure 2. Elevation and cross-sections of the bridge.

3 AMBIENT VIBRATION TEST

The ambient vibration test was developed without disturbing the normal use of the bridge, the measured accelerations being mainly induced by traffic and wind. To measure the very low amplitude accelerations 4 tri-axial 18-bit strong motion recorders were used (Fig. 3). These devices are constituted by very sensitive internal force balance accelerometers (linear behaviour from DC to 100Hz), analogue to digital converters with 18 bit (to guarantee high resolution), batteries that enable autonomy for one day of tests, memories materialized by removable Compact Flash cards, that permit a fast download of the acquired data and external GPS sensors to deliver a very accurate time, so that the several recorders can work independently and synchronously, avoiding the use of cables and minimizing the labour associated with the preparation of the dynamic test.

During the ambient vibration test, two recorders served as references, permanently located at section 8 (Fig. 3), at both sides of the deck (upstream and downstream), while the other two scanned the bridge deck in 15 consecutive setups, measuring the acceleration along the 3 orthogonal directions, at both sides of the sections represented in Figure 3.

The position of the reference section was selected using the configuration of the mode shapes provided by a previously developed numerical model, in order to avoid that the reference section is placed close to a node of a relevant mode shape.

For each setup, time series of 16 minutes were collected. The sampling frequency was 100 Hz, value that is imposed by the filters of the acquisition equipment and that is much higher than the necessary for this bridge, as the most relevant natural frequencies of the bridge are below 10 Hz. Therefore, a decimation of order 5 was applied before the application of the identification tools, reducing the sampling frequency from 100Hz to 20Hz.

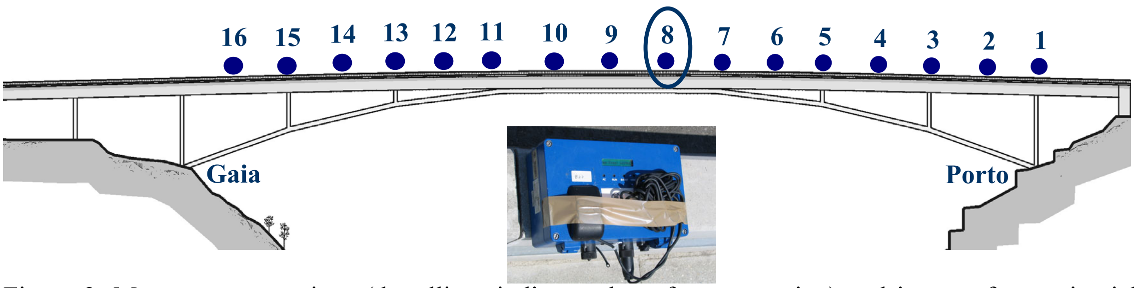


Figure 3. Measurement sections (the ellipse indicates the reference section) and image of one tri-axial strong motion recorder.

4 IDENTIFICATION OF THE MODAL PARAMETERS

The identification of the modal parameters from the data collected in the ambient vibration test was performed using two output-only identification methods with completely different theoretical backgrounds: the Enhanced Frequency Domain Decompotion (developed in the frequency domain) and the Data driven Stochastic Subspace Identification (developed in the time domain). The simultaneous application of these two different methods permitted to achieve a higher confidence level on the identified parameters.

4.1 *Enhanced Frequency Domain Decomposition*

The first step of this method is the construction of a spectrum matrix of the ambient response for each test setup, with a number of lines equal to the number measurement points and with as much columns as the number of points elected as references, each column containing the cross spectra relating the structural response at all the measured points with the corresponding response at a reference point.

In the present application, four time series were considered for each instrumented section: upstream and downstream vertical accelerations, mean of the two measured lateral accelerations and mean of the two measured longitudinal accelerations. So, for each setup, spectra matrices with 8 rows and 4 columns were organized. The elements of these matrices were estimated using the Welch procedure, dividing the available time series in segments of 102.4 seconds (2048 points) and considering an overlap between segments of 66%. The selected parameters allowed the realization of 26 averages and produced spectra with a frequency resolution of 0.00977 Hz.

It can be shown (Brincker et al. 2000) that, under some assumptions (white noise excitation, low damping and orthogonal mode shapes for close modes), the singular values of the spectrum matrix, in the vicinity of each resonant frequency, are auto-spectral density functions of single degree of freedom systems with the same frequency and damping as the structure vibration modes.

At Figure 4, a graphic with the average of the normalized singular values covering all setups is presented. Inspection of this figure shows that 13 resonant frequencies were identified (marked with the dashed vertical lines) in the frequency range of analysis (0 - 5 Hz).

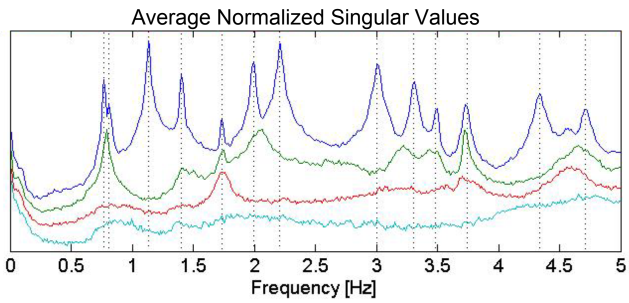


Figure 4. Average Normalized Singular Values.

The auto-spectra density functions, formed by the singular values of the spectrum matrices of the 15 setups, were then used to calculate auto-correlations functions, associated with the different modes of the structure, applying an inverse FFT. From these functions, it is straightforward to identify the modal damping coefficients and obtain enhanced estimates of the natural frequencies. These frequencies are calculated looking at the time intervals between each zero crossing. The modal damping coefficients are estimated adjusting an exponential decay to the relative maxima of the auto-correlation functions.

Table 1 (at the end of Section 4.2) presents the mean of the 15 identified frequencies and modal damping coefficients, as well as the standard deviation (Std.) of the estimates. It is important to note that the observed low values of damping ratios are associated with low amplitudes. However, it is expectable an increase of these values for higher amplitudes. The relatively high standard deviations of the damping estimates show that these parameters suffer variations between setups and also that there is some uncertainty in the estimation.

The mode shapes are estimated from the singular vectors of the spectrum matrices evaluated at the identified resonance frequencies and associated with the singular values that contain the peaks. In each setup, 8 ordinates are calculated. These are then grouped together using the ordinates of the reference sections.

Figures 5 and 6 show some of the identified modes of vibration of the bridge deck plotted with Artemis (SVS 1999-2004).

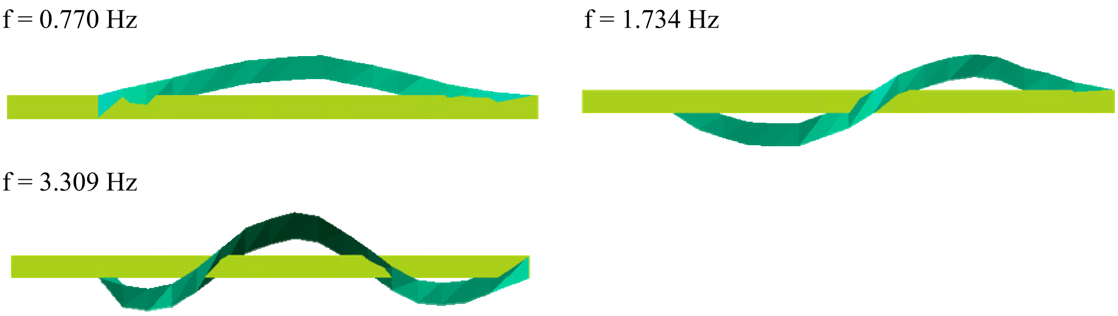


Figure 5. Top view of the first, second and third lateral bending modes.

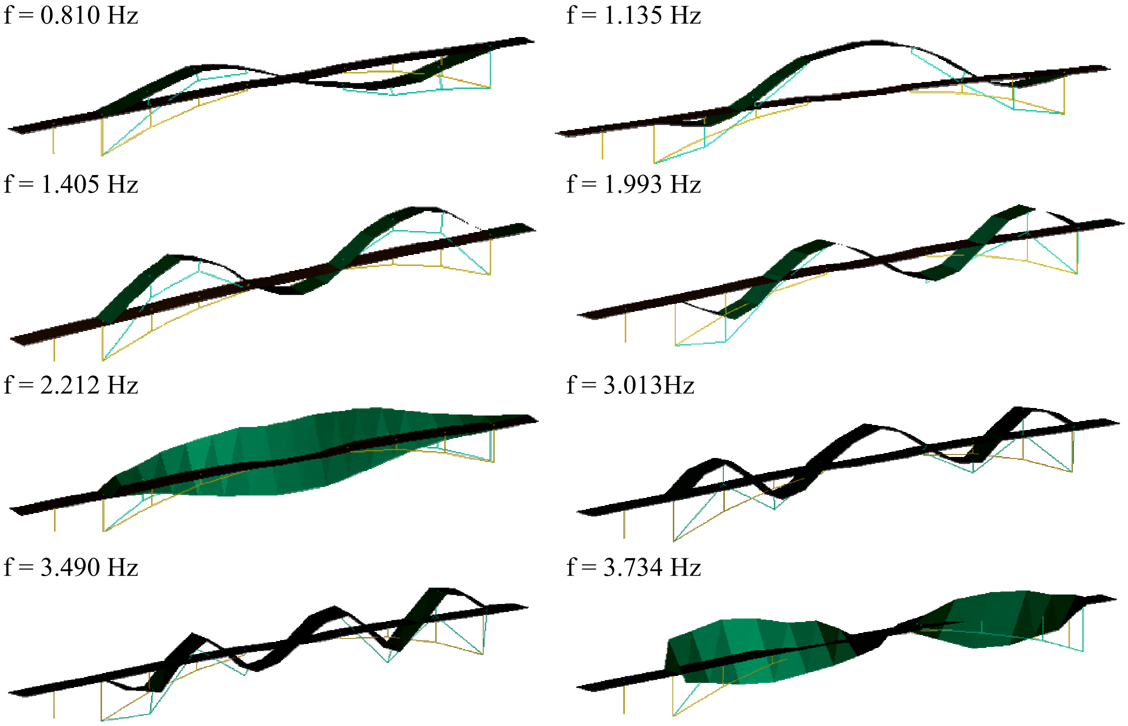


Figure 6. Perspective of some identified vertical bending and torsion modes.

4.2 *Data driven Stochastic Subspace Identification*

The Data Driven Stochastic Subspace Identification method (SSI-DATA) performs the identification of the modal parameters using a stochastic state space model that, in its discrete form and assuming the excitation as a white noise, is represented by the following equations:

$$x_{k+1} = A \cdot x_k + w_k$$
$$y_k = C \cdot x_k + v_k \tag{1}$$

The identification of matrices A and C is performed directly from the time series using the concept of projection of subspaces. The algorithm of the method is based on the main theorem of stochastic subspace identification (Overschee & Moor 1996) and involves a QR factorization, a singular value decomposition and the resolution of a least-squares equation.

After the identification of the state space model, the modal parameters are extracted from matrices A and C (Peeters 2000).

It is worth noting that the identification of the state space model requires the definition of the order of the model. However, for real Civil Engineering structures it is not possible to predict the order of the model that better fits the experimental data and more realistically characterizes the dynamic behaviour of the structure. The most appropriate way to overcome this difficulty is to estimate the modal parameters using models with an order within an interval previously defined in a conservative way. The identified modal parameters are then represented in a stabilization diagram (Figure 7). Such diagram permits to distinguish the parameters that are stable for models of increasing orders, and these are in fact the ones that have structural meaning. The others are just associated with numerical modes, which are important to model the noise always present in measured data.

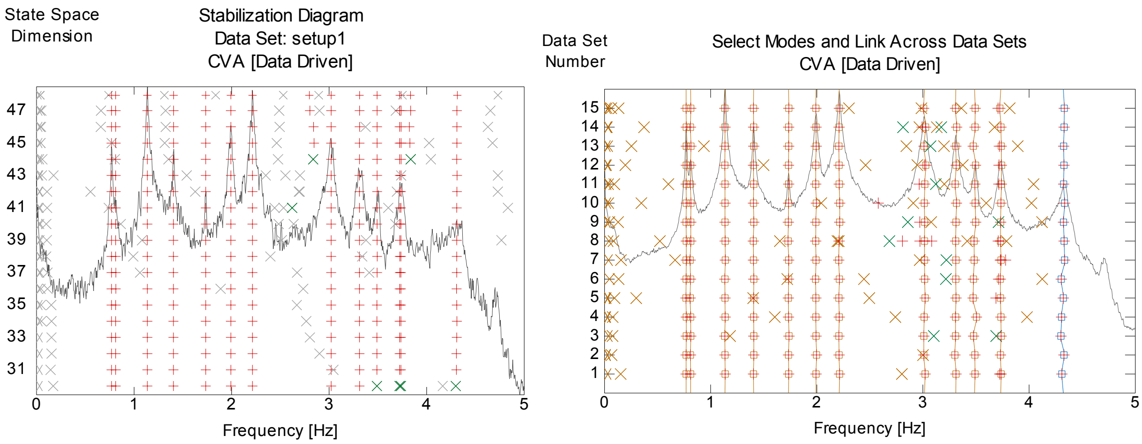
In the present application, in order to reduce the order of the models used to fit the experimental data and consequently the computation effort, the acquired time segments were low pass filtered with a cut frequency of 5 Hz.

The filtered data of the 15 setups was then processed independently using the same 8 collected time segments that were used in the EFDD method, and so 15 stabilization diagrams were constructed, like the one presented in Figure 7a (stable modes represented by the symbol +). The presented diagram shows that the dynamic behaviour of the structure is well represented by state-space models of order between 30 and 50.

Using the 15 stabilization diagrams, the order of a suitable model was selected and the natural frequencies and the modal damping coefficients of that model were calculated. Figure 7b shows the 15 estimates of frequencies. Table 1 presents the mean and the standard deviation (Std) of the identified frequencies and damping ratios using the EFDD and SSI-DATA.

The comparison of the modal parameters estimated by the two applied methodologies shows that the identified natural frequencies are almost coincident. Concerning the modal damping coefficients, a significant difference only occurs in the first lateral mode (L1). This difference stems probably from the inability of the EFDD method to provide reliable damping estimates for modes with low frequencies and damping coefficients using relatively short time segments.

The last column of Table 1 presents the MAC coefficient between the modes shapes estimated by both techniques. All the values are close to 100%, meaning that the correlation is excellent.



a) b)

Figure 7. SSI-DATA method results: a) stabilization diagram of setup 8 with the 1st singular values of the spectrum matrix plotted in the background; b) Representative points of modes selected in the 15 setups.

Table 1. Identified natural frequencies and modal damping ratios: EFDD vs SSI-DATA.

	EFDD				SSI-DATA				
Mode*	Frequency [Hz]	Std. F. [Hz]	Damp. R. [%]	Std. D. R. [%]	Frequency [Hz]	Std. F. [Hz]	Damp. R. [%]	Std. D. R. [%]	MAC [%]
L1	0.770	0.0011	0.96	0.11	0.770	0.0010	0.52	0.14	99.81
V1	0.810	0.0029	1.73	0.53	0.811	0.0023	1.73	0.71	98.92
V2	1.135	0.0014	0.71	0.11	1.135	0.0013	0.47	0.09	99.98
V3	1.405	0.0013	0.69	0.13	1.405	0.0016	0.54	0.16	99.47
L2	1.734	0.0012	0.68	0.10	1.734	0.0021	0.58	0.15	97.43
V4	1.993	0.0029	0.62	0.10	1.992	0.0025	0.47	0.08	99.96
T1	2.212	0.0025	0.57	0.11	2.211	0.0028	0.46	0.10	99.98
V5	3.013	0.0051	0.60	0.18	3.015	0.0060	0.66	0.65	99.65
L3	3.309	0.0050	0.63	0.16	3.310	0.0041	0.47	0.12	99.89
V6	3.490	0.0052	0.47	0.09	3.487	0.0093	0.57	0.30	96.11
T2	3.734	0.0049	0.52	0.12	3.734	0.0044	0.47	0.09	97.75
V7	4.339	0.0051	0.90	0.25	4.322	0.0119	1.04	0.39	98.08
T3	4.714	0.0049	0.61	0.12	-	-	-	-	-

* Mode type, defined considering the most relevant components
(L – lateral bending; V – vertical bending; T – torsion).

5 NUMERICAL MODELLING

The structural behaviour of the bridge was modelled in the ANSYS software using 3D beam finite elements. The cross section properties (area, two area moments of inertia, torsion moment of inertia and two shear deflection constants) were defined according to the geometry of the deck, arch and columns. Concerning the material properties, it was adopted an elasticity modulus of 37 GPa (provided by the material tests performed during the bridge construction) for the deck and arch, and an elasticity modulus of 34 GPa (value defined by the Eurocodes for a C35/45 concrete) for the columns.

The connections between the deck and the highest columns (M1 and M6, Figure 2) are monolithic, whereas the connections with the other columns and the abutments are constituted by two unidirectional sliding pot bearings. These bearings are designed to allow relative movements along the longitudinal direction of the bridge and relative rotation in all directions. However, for low levels of excitation, as is the case during the ambient vibration tests, the behaviour of these connections can be different. For low levels of displacements, it can happen that the friction forces are sufficient to preclude relative displacements or rotations.

To analyze the influence of the behaviour of these connections on the modal parameters, three alternative models were developed:

- Model 1 (M1): longitudinal displacements and rotations free in all pot bearings;
- Model 2 (M2): longitudinal displacements and rotations fixed in all pot bearings;
- Model 3 (M3): longitudinal displacements and rotations fixed in the pot bearings of the columns, but free in the pot bearings of the abutments.

The natural frequencies of the most relevant modes provided by the three models are presented in Table 2. Models 1 and 2 are ideal models that define the lower and upper bounds of the numerical natural frequencies calculated using the material properties previously defined. It is interesting to observe the significant variation of the natural frequencies of the vertical modes, and especially of the first one, with changes in the characteristics of the connections.

The experimentally identified modes show that the deck has longitudinal movements. Therefore, the hypothesis of fixing the longitudinal movements in the abutments is not realistic. On the other hand, M3 shows that even fixing the relative movements and rotation in all columns, the numerical frequency of the first mode is considerably lower than the experimental one.

In order to better understand the longitudinal behaviour of the bridge, two additional measurements were performed: measurement of the longitudinal acceleration at the deck near the dilation joint of Gaia and measurement of the longitudinal acceleration of the Gaia abutment. The spectra of the collected time series are represented in Figure 8. These graphics show the presence of some of the natural frequencies of the bridge in the response of the abutments and in particular, the presence of the natural frequency of the first vertical mode. This means that a fraction of the stiffness of the abutments is mobilized in the movements associated with the first vertical bending mode, due to the existence of friction forces. Accordingly, the appropriate modelling of the behaviour of the bridge requires the inclusion in Model 3 of horizontal springs to simulate the additional stiffness provided by the abutments.

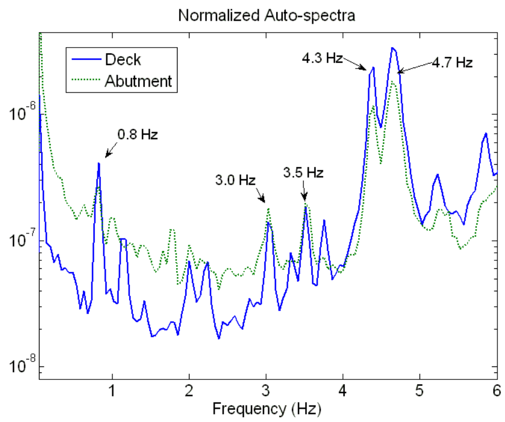


Figure 8. Normalized auto-spectra of longitudinal accelerations measured simultaneously at the deck and at the Gaia abutment.

This last conclusion led to the development of a final model of the bridge (Model 4). This model is similar to Model 3, but includes a horizontal spring at each abutment with a stiffness constant that was adjusted in order to obtain a good matching between the numerical and experimental frequencies. Table 2 shows that the correlation between the modal parameters of the final numerical model and the experimental ones (estimated by the SSI-DATA method) is very good, the relative errors of the natural frequencies being lower than 5% and the MAC values always greater than 95%.

Table 2. Modal parameters of the developed numerical models

	Exp. (SSI)	Model 1		Model 2		Model 3		Model 4		
Mode	Freq. [Hz]	Freq. [Hz]	Error (%)	Freq. [Hz]	Error (%)	Freq. [Hz]	Error (%)	Freq. [Hz]	Error (%)	MAC
V1	0.811	0.541	-33.29	1.054	29.96	0.701	-13.56	0.810	-0.12	99.50
V2	1.135	1.081	-4.76	1.164	2.56	1.144	0.79	1.149	1.23	99.41
V3	1.405	1.244	-11.46	1.473	4.84	1.465	4.27	1.466	4.34	99.26
V4	1.992	1.766	-11.35	2.088	4.82	2.086	4.72	2.086	4.72	99.42
L1	0.770	0.794	3.12	0.799	3.77	0.794	3.12	0.794	3.12	99.68
L2	1.734	1.767	1.90	1.788	3.11	1.768	1.96	1.768	1.96	98.94
L3	3.310	3.353	1.30	3.394	2.54	3.357	1.42	3.357	1.42	96.54
T1	2.211	2.169	-1.90	2.211	0.00	2.185	-1.18	2.185	-1.18	95.69
T2	3.734	3.632	-2.73	3.724	-0.27	3.641	-2.49	3.641	-2.49	95.20

6 CONCLUSIONS AND FUTURE RESEARCH

This paper shows that the development of an ambient vibration test together with the application of powerful output-only modal identification techniques allows the very accurate estimate of the most relevant dynamic structural properties of bridge structures. These are essential to achieve an updated numerical model that can enable a realistic characterization of the dynamic behavior of the bridge. The updated model can be used, in a future stage, to detect possible changes in the modal parameters stemming from potential structural damage.

This study suggests however some further development in order to better understand the real behavior of the bearings. In particular, it would be interesting to measure accelerations simultaneously at one point of the deck and at the top of all columns with sliding bearings, in order to check the existence, or not, of relative movements.

This work constitutes a first step towards the development and installation of a continuous dynamic monitoring system in Infante D. Henrique Bridge, to investigate the variation of modal parameters with temperature and traffic intensity.

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