Dynamic Study of the Barqueta Cable-Stayed Bridge

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ABSTRACT

A theoretical and experimental research work of the Barqueta cable-stayed bridge is described in this paper. The Barqueta Bridge, across Guadalquivir river, links the city of Seville with the Scientific Park Cartuja 93. At jam hours cars may cover one half of the bridge lanes for more than one hour. Full-scale tests were carried out to measure the bridge dynamic response. The experimental program includes the dynamic study for both cases: the bridge with one half of it lanes full of cars, and empty.
Modal parameters estimations were made based on the acquired data. Ten vibration modes have been identified in the frequency range of 0-6 Hz by different techniques, being two of these modes very close to each other. The traffic-structure interaction is also studied. The experimental results were compared with those obtained from a three-dimensional finite element model developed in this work. Both sets of results show very good agreement.
Finally, a damage identification technique has been applied to determine the integrity of the structure.

1. INTRODUCTION

Experimental tests constitute the most reliable method to obtain the dynamic properties (natural frequencies, mode shapes and damping ratios) of real structures and to validate, from these results, numerical models. They permit also to asses the state of damage of the structure by comparison with the dynamic properties obtained in previous analyses.

Dynamic identification methods from ambient vibration have been used in complicated structures like dams [1], offshore platforms [2], sports stadium [3], bridges [4], etcetera. A theoretical and experimental research work of the Barqueta bridge is described in this work. It is a steel arch bridge with cable-stayed deck that links the old town of Seville with the Scientific Park Cartuja 93. At certain hours, a traffic jam occurs on the bridge during one hour, approximately, cars covering one half of the bridge lanes (Figure 1).

Figure 1. Barqueta Bridge (left hand-side) and traffic jam on Barqueta Bridge (right hand-side)

The experimental program developed in this work includes the dynamic characterization of the structure under normal conditions and also when it is covered by traffic. The modal properties of the bridge are identified from ambient vibration and the results obtained for both states of the bridge compared. By this analysis, the effect that vehicles have in the dynamic behaviour of the structure is studied. Modal properties of a sport stadium determined at times when different activities are celebrated have been reported in [3]. The authors of that paper concluded that the dynamic properties of the structure depend to some extent on the type of the activity celebrated in the stadium. The possibility of these changes taking place in other types of structures is studied in this work.

The obtained experimental results are compared with those from a three-dimensional finite element model. Finally, a damage identification methodology to verify the structural integrity of the bridge in the future is applied.

2. DESCRIPTION OF THE STRUCTURE

The aesthetic functions and symbolic values of the structures are more and more important, overcoat when these structures are built in urban zones [5, 6]. The Barqueta bridge was built in Seville for the 1992 International Exhibition [7]. It is an innovative structure, designed by J. J. Arenas and M. Pantaleón, with a flying central arch rising from the vertex of two lateral triangular frames. Fifteen years after its construction, Barqueta is an indispensable piece of the urban landscape of Seville.

Barqueta is a bowstring steel bridge. The 168 m. span over the Guadalquivir River rests in four vertical supports spaced 30 m. in the transverse direction. The cross-sections of the arch and inclined legs include deep grooves that produce enough local inertia to avoid any internal longitudinal stiffener (Figure 2).

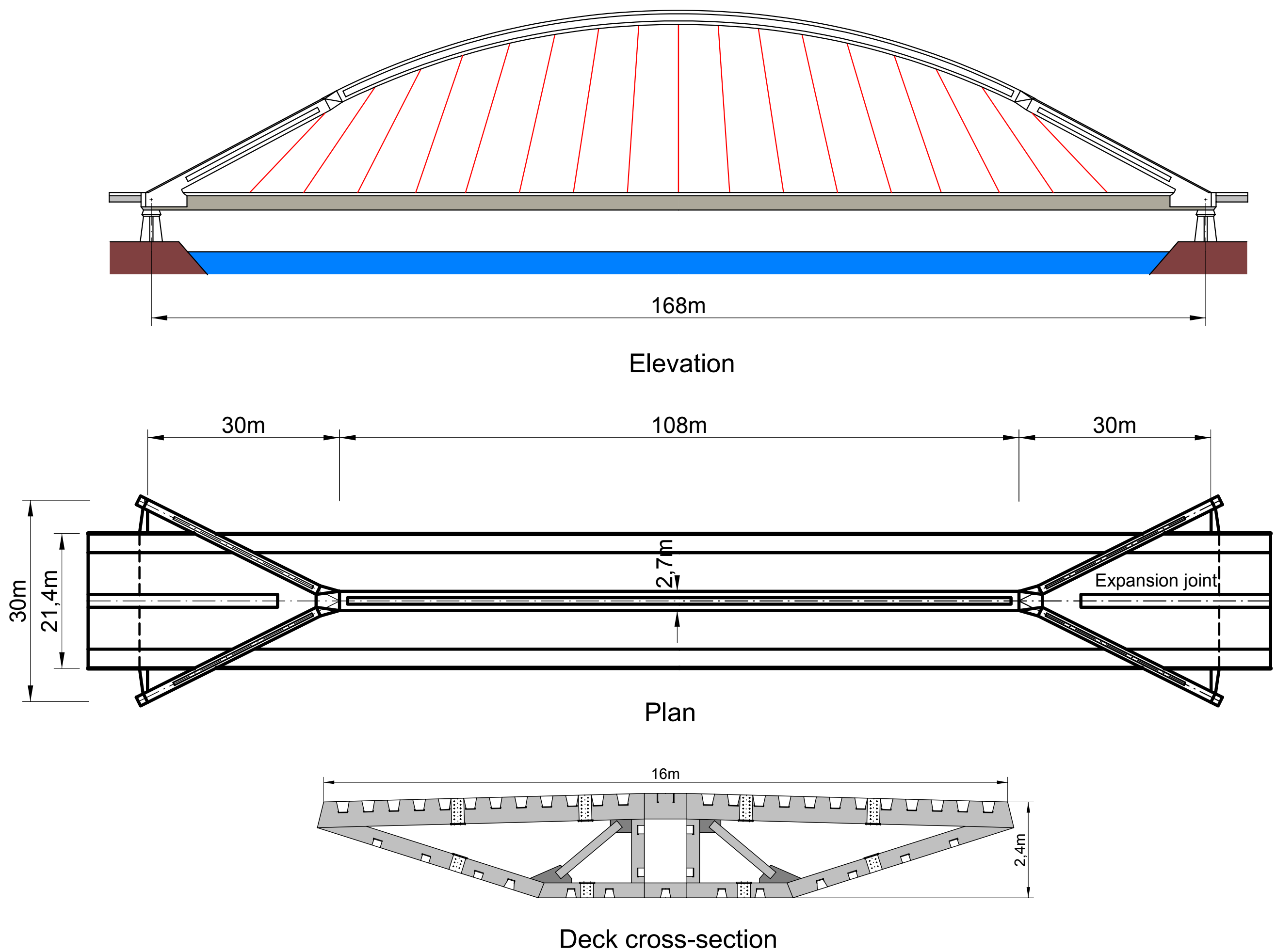

Figure 2. Elevation, plan and deck cross-section of the Barqueta Bridge

The deck cross-section is shown in Figure 2. It is 16 m. wide and 2.4 m. deep. The total wide of the bridge is 21 m. with two cantilever pedestrian decks. The deck cross-section includes two vertical webs at 1m. distance. The hangers are anchored between then. The hangers have variable inclination. The deck is supported on the extremes by transversal beams, with variable depth, which rest on the vertical supports.

3. FINITE ELEMENT MODEL

A three dimensional finite element model (FEM) has been developed for the numerical analysis of the structure using as-built drawings of the bridge and in-situ measurements. Modal analysis was carried out in ANSYS [8].

The arch, supports, and the internal stiffener were represented as two-node beam elements (BEAM44) with 6 degrees of freedom per node. This element permits the end nodes to be offset from the centroidal axis of the beam. The hangers were modelled as truss elements (LINK10) with 3 degrees of freedom per node. The deck slab was modelled using eight-node shell elements with 6 degrees of freedom per node (SHELL93). The connection between the big extreme beams and the vertical supports was established by using a spring element (COMBINE14).

A detailed model of all the elements of the bridge was intended. As a consequence, the number of degrees of freedom is high. The full model consists of 11934 beam elements, 17 truss elements, 18044 shell elements and 8 spring elements, resulting into 30003 elements and 53699 nodes. Figure 3 shows the full 3-D view of the finite element model of the Barqueta Bridge and a detail of the deck cross-section.

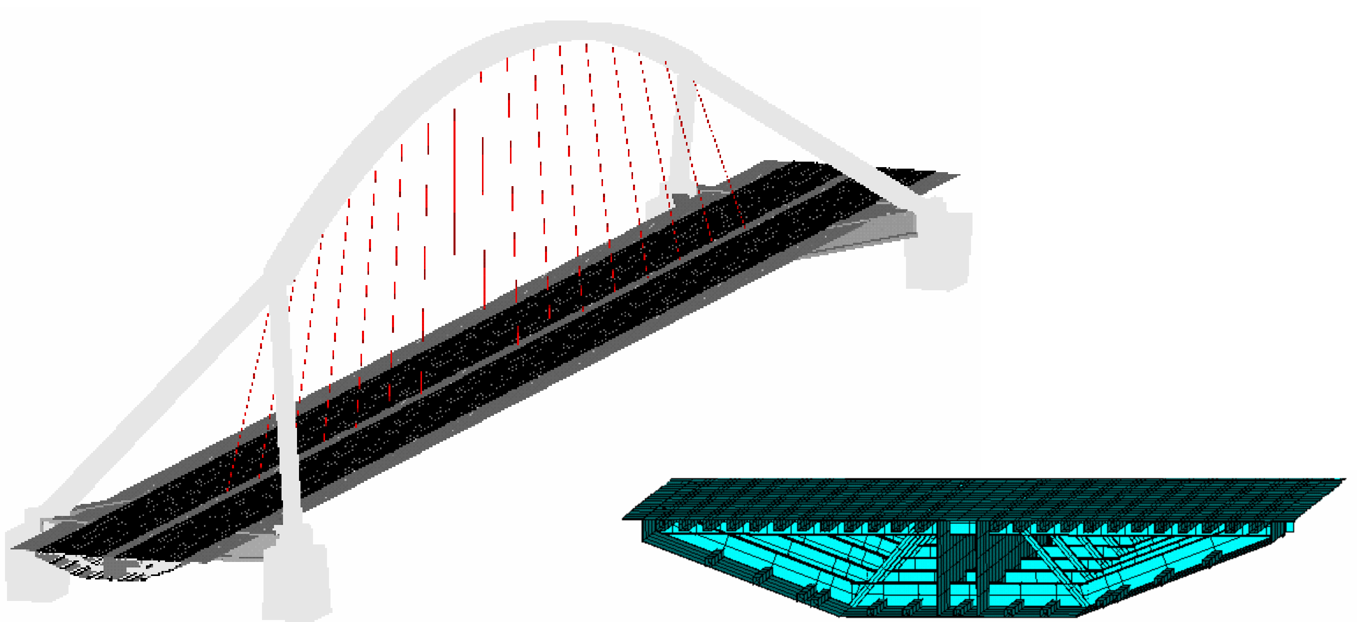

Figure 3. The 3-D FE model of Barqueta Bridge (left hand-side) and detail of the deck cross-section model (right hand-side)

4. FULL-SCALE TESTING

Dynamic properties can be obtained by measurement of ambient vibrations. This technique is simpler than classical modal analysis for civil engineering structures because it is not necessary to control shakers at high levels. In addition, the structure can be used during testing.

The experimental program, carried out during July 15 2005, includes dynamic characterization of the structure in normal conditions and when the bridge is covered by traffic. The response of the structure was measured at 16 selected points using Endevco (Model 86) piezoelectric accelerometers (Figure 4). These accelerometers have a nominal sensitivity of 1000 mV/g and a low frequency limit of approximately 0.1 Hz. The results from the FE dynamic analysis were used to determine the location of the sensors.

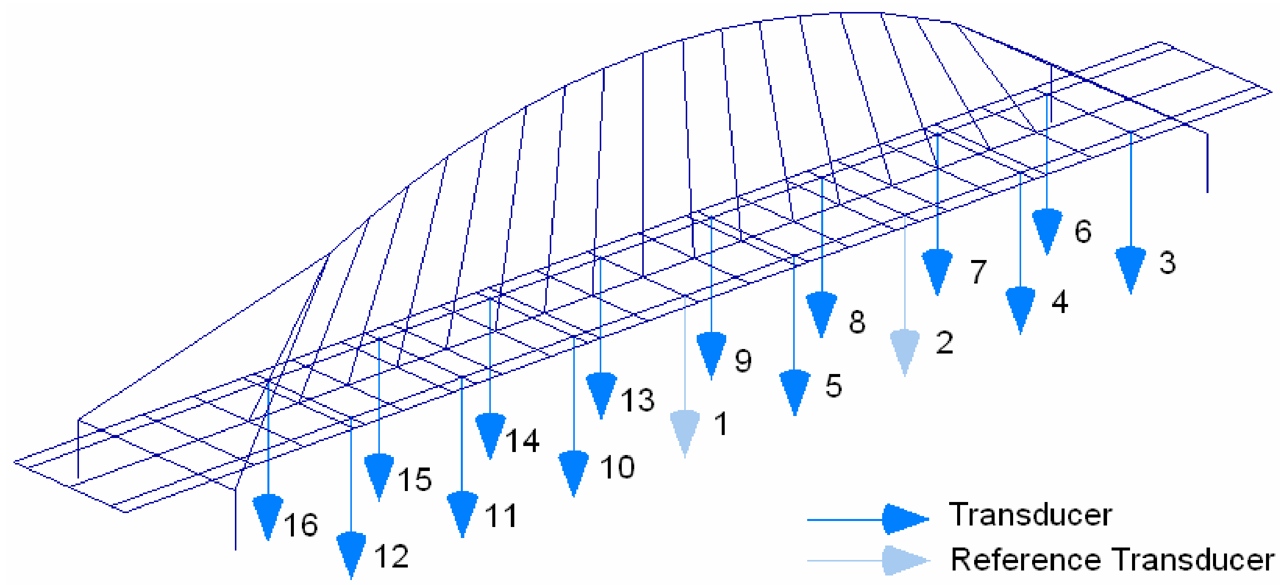

Figure 4. Measurement locations.

Since a maximum of nine accelerometers was available for the testing and two of these sensors were held stationary for reference measurements, two set-ups were required to cover the 16 measurement points. In output-only modal analysis where the input force remains unknown and may vary between the set-ups, the different measurements setups can only be linked if there are some sensors in common. The reference accelerometers were settled very carefully to measure all global modes of the bridge.

The hangers were not instrumented because, from numerical analysis made and preceding experimental studies [9, 10], a significant interaction between the stay-cables and the rest of the structure was not expected. This interaction seems to be controlled by the closeness between the lowest natural frequencies of the structure, and the natural frequencies of the cables. In the present case, cables are relatively short, estimating their natural frequencies in about 4 Hz.; existing, at least, 5 global modes of the structure below this values. Therefore it is not expected that cables have an important participation in the global modes of the bridge.

Data of the response of the structure were acquired when the structure was in normal conditions, in the locations points 1 to 9, and when cars cover one half of the lanes of the bridge, in points 1,2 and 10 to 16. To obtain the mode shapes of the structure it has been used the responses in all points, because it is not expected that the traffic-structure interaction cause a change in the mode shapes, while to obtain natural frequencies and damping ratios, it has been used each one of the set-ups independently.

Ambient vibration response was acquired in 1000 seconds per channel per set-up. The data were sampled to 64 Hz. Data were downsampled to 20 Hz. or were decimated (order 3), according to the identification technique, to carry out data analysis in the interest frequency range (0 to 10 Hz.). Data records were Hannning-windowed with 66.67% overlapping for spectral averaging.

The acceleration histories at the reference locations in both traffic conditions are shown in Figure 5. It can be observed how traffic vibrations when the bridge is in normal conditions are higher than when there is traffic jam. This is probably because vehicles in the opposite sense to the jam, are faster when bridge is in normal conditions than when the jam exists.

Acquired data are available for interested reader sending an e-mail to the authors.

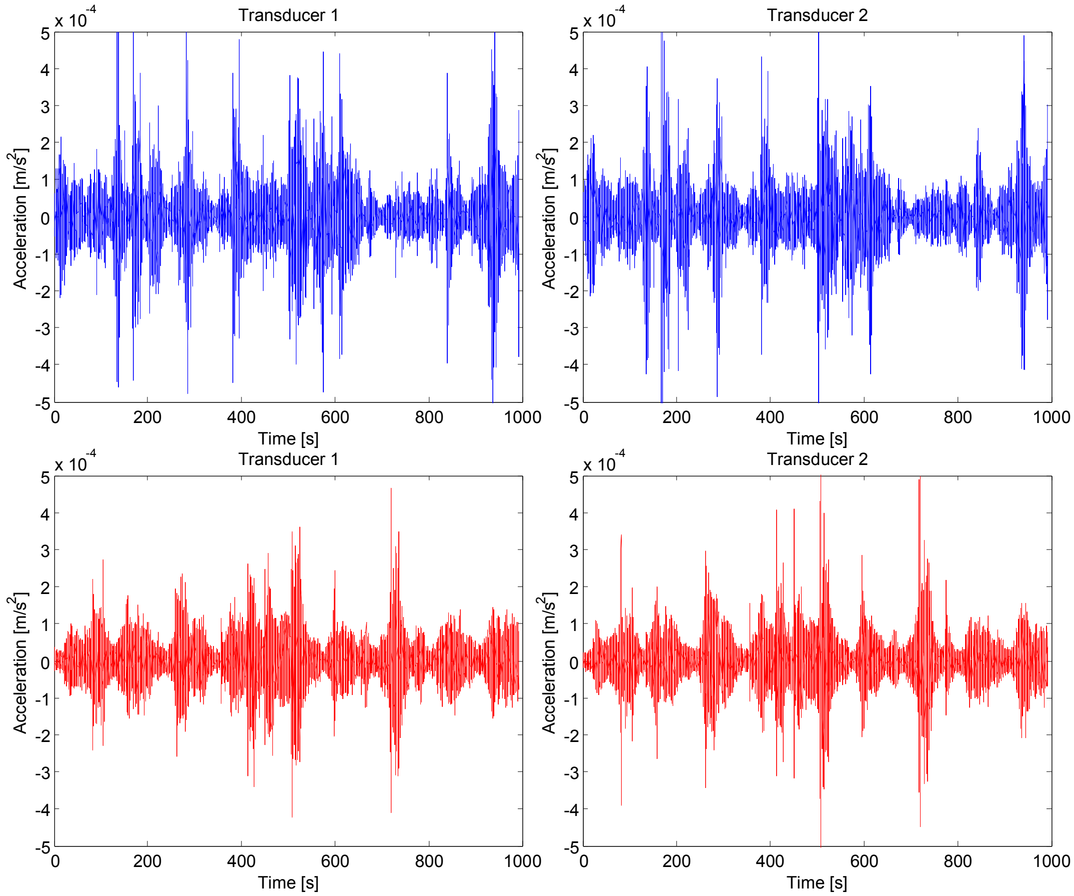

Figure 5. Acceleration histories at the reference locations in normal conditions (upper) and when the traffic jam is generated (lower)

5. DATA ANALYSIS

Different procedures to extract the modal parameters from ambient vibration data have been employed. In output-only modal analysis, applied force is unknown and, therefore, neither the frequency response function nor the impulse response function can be obtained to determine modal parameters using classical modal analysis [11]. In output-only analysis, also called operational modal analysis, it calculates the frequency response function and the impulse response function using as reference one of the fixed transducers.

In this work, four complementary identification methods have been employed: three in the frequency domain and one of in the time domain. The analysis has been developed using MATLAB [12] and ARTEMIS [13] software.

The first identification method employ is Peak-Picking (PP), which has been used with success in many other applications [5,10,14,15]. This method is based on the fact that when the frequency response function reaches a peak at certain frequencies, it can be associated to the force or to the resonance frequencies of the structure [16]. To distinguish between peaks associated to the excitation and those that represent the resonance frequencies of the structure, the mode shapes can be used [16]: the response values at all points of a structure, weakly damped, in one of its resonance frequencies, are in phase or in against phase, depending on the mode shape. Peaks of spectra density function associated to the excitation and not to the dynamic behaviour of the structure, present a phase of the cross-spectra function between two measurements points that will be, normally, different from 0º and 180º. In addition, the coherence function between two signals has a value close to one for the resonance frequencies of the structure, due to the high relation response-noise in resonance frequencies. This fact helps to decide which of the frequencies can be considered natural frequencies of the structure. The method is based on

the assumption that the dynamic response of the structure at resonance peaks, is determined for each mode. This is valid when modes are well separated. Therefore, it is difficult to identify very close modes using this method.
The auto-spectra, cross-spectra and coherence functions, are shown in Figure 6. The natural frequencies were identified from resonance peaks in auto-spectra and cross-spectra functions. Coherence function peaks are the same that the peaks of the previous functions, and the phase value of the cross-spectra function in those peaks are 0º or 180º, providing still more evidence that these peaks correspond to natural frequencies of the structure.

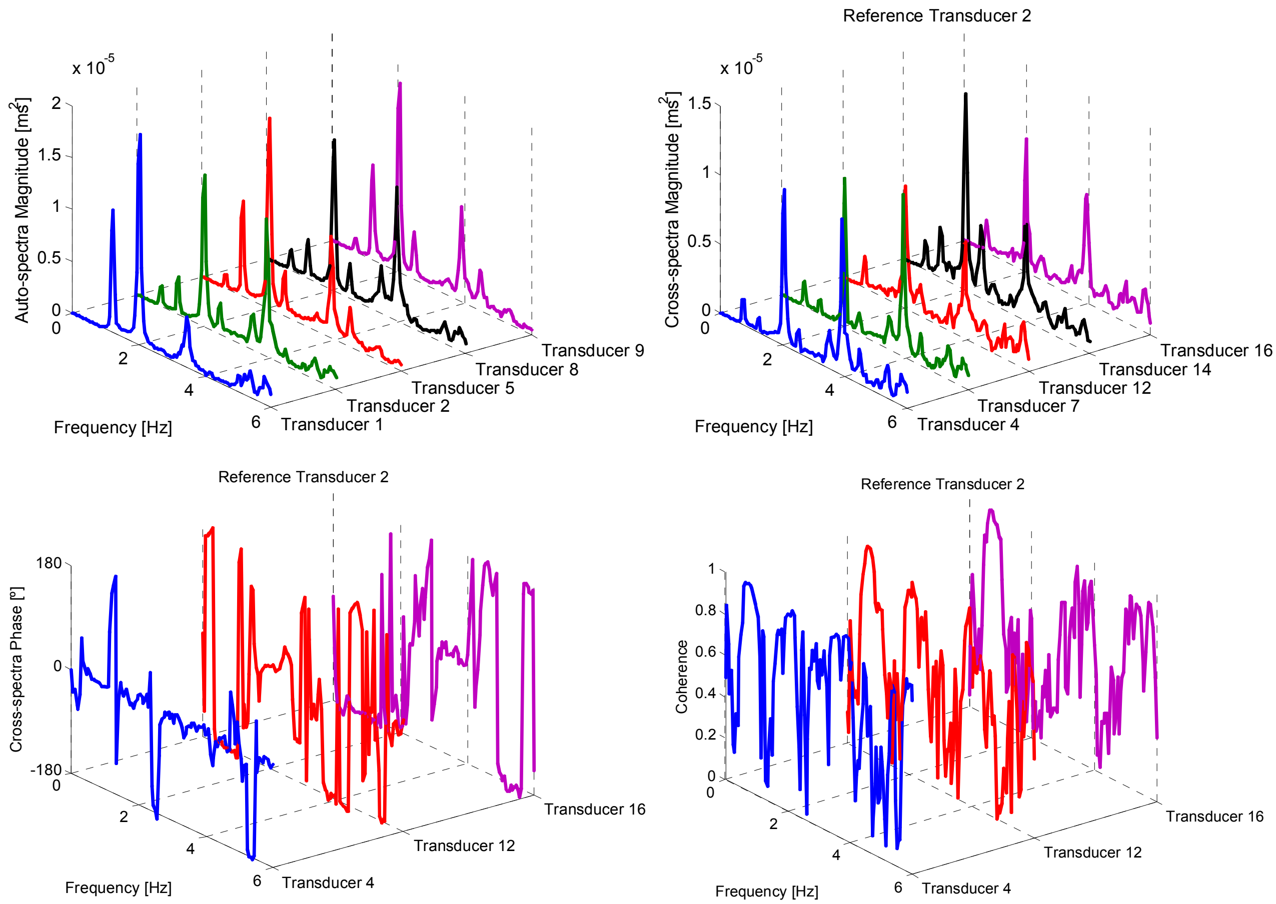

Figure 6. Auto-spectra function, cross-spectra function (magnitude and phase) and coherence function.

The second identification technique is called Averaged Normalized Power Spectral Densities (ANPSDs) [17]. In this case, auto-spectra functions are normalized and averaged to obtain an average spectra density function that, normally, shows all resonance frequencies of the system. The natural frequencies of the structure are obtained from the simple observation of the peak in the ANPSDs diagram. This method was used successfully in the dynamic characterization of an arch bridge in reference [18]. The ANPSDs diagrams for both situations of the bridge are shown in Figure 7.

The third identification technique used, also in the frequency domain, is called Enhanced Frequency Domain Decomposition (EFDD) [19] which, from a simple form, introduces significant improvements to Peak Picking Technique. This method is based on a modal decomposition realization of the spectral density matrix, being one of the advantages of the method the possibility to identify very close modes.

The modes of the structure are determined from singular value and singular vector decomposition of the spectral density matrix, while to obtain the natural frequencies and the damping ratios, the spectral density function is fitted well near the peaks and then, the time domain response of a one degree of freedom system is obtained by the Inverse Fast Fourier Transform.

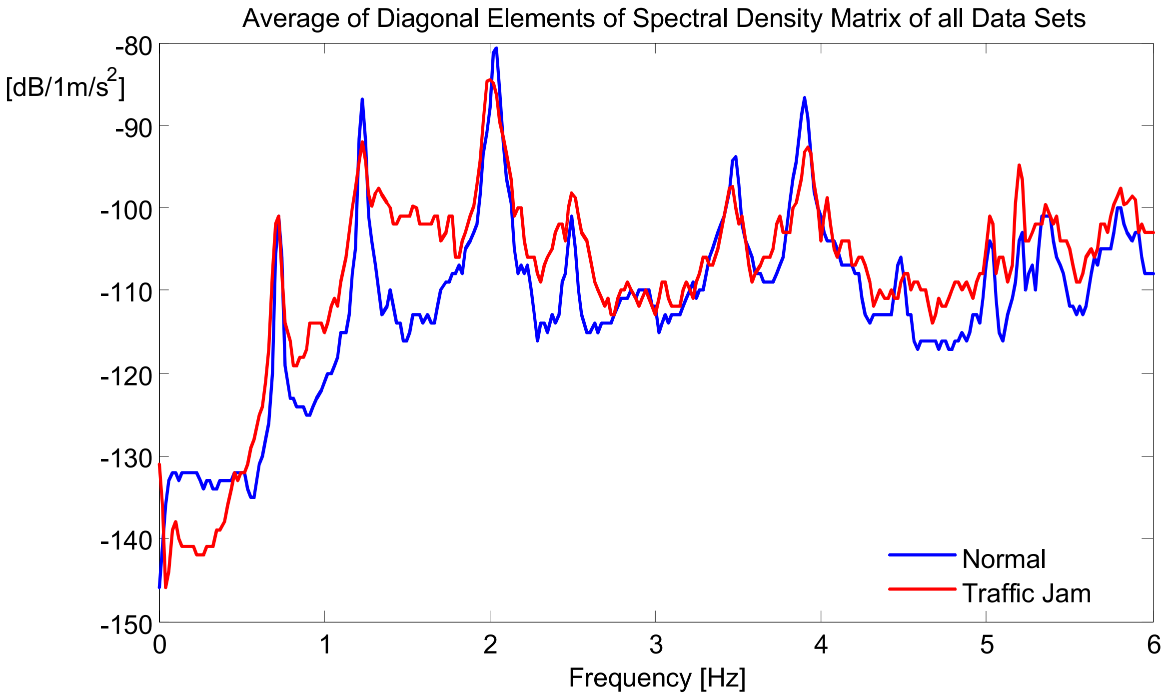

Figure 7. Average Normalized Power Spectral Densities

The singular values decomposition of the spectral density matrix is shown in Figure 8. Those peaks that represent vibration modes have been selected.

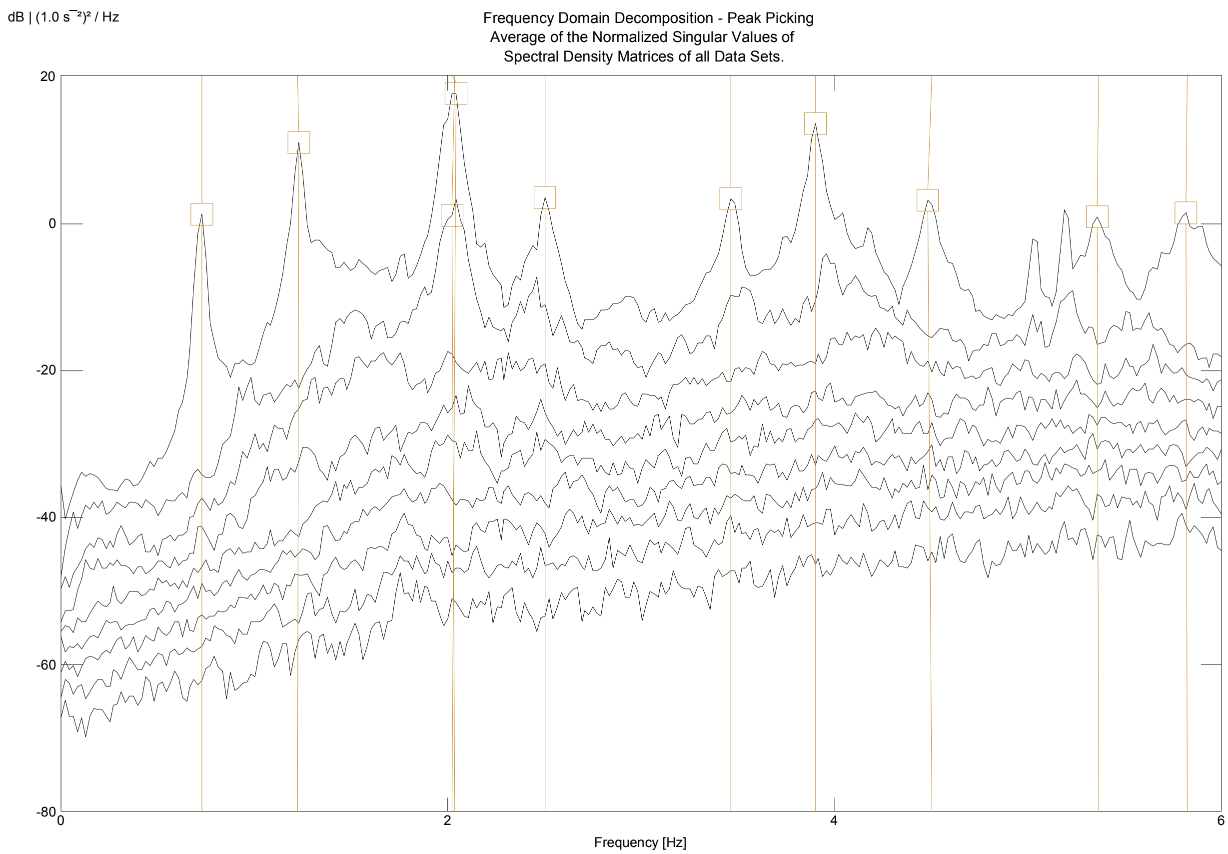

Figure 8. Singular Value Decomposition of the Spectral Densities Matrices

It can be observed that two modes very close to 2 Hz. exist, one of them not been determined by the previous methods. This type of modes can be easily identified with EFDD, by observation not only of the highest singular value but also of the next one.

The last technique is a more elaborated mathematical procedure that works directly with time domain acquired data. It is called Stochastic Subspace Identification (SSI). The interested reader can find details of the mathematical approach in references [20,21]. SSI is a powerful tool (perhaps the most advanced identification method that exists up to day) that has been used for dynamic characterization of many structures [20,22,23].

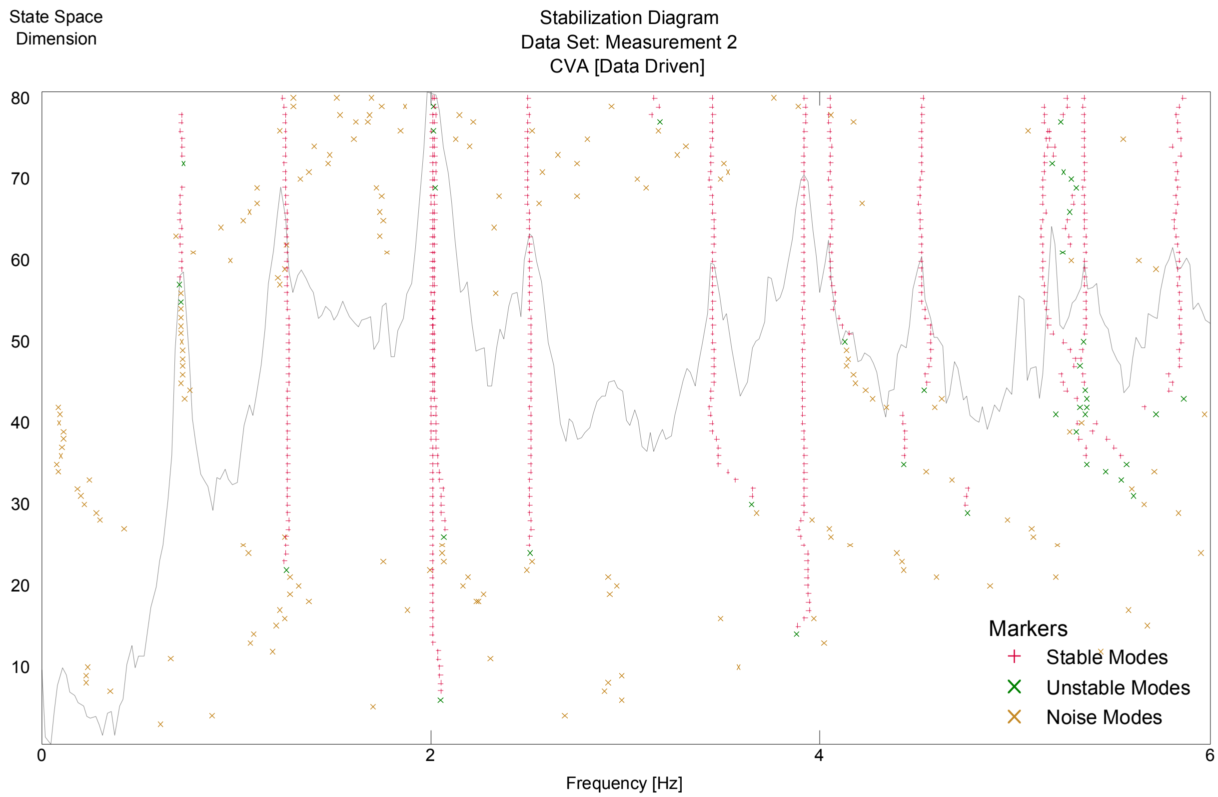

Figure 9. Stabilization Diagram.

Figure 9 is shows a stabilization diagram obtained by applying SSI. System order and stable poles can be found in these diagrams which provide modes of the structure.

6. DYNAMIC BEHAVIOUR OF THE BRIDGE

The dynamic behaviour of the Barqueta Bridge is governed by vertical bending and torsional modes, in the frequency range of 0-6 Hz. Ten modes have been identified in this frequency range. Table 1 shows the obtained results when the bridge is in normal condition and Table 2 when the bridge is jammed. Damping ratios obtained for both bridge conditions are shown in Table 3.

The natural frequencies of the structure change very little due to the different loading conditions of the bridge. However, damping ratios may increase up to 200 per cent when the bridge is jammed with vehicles, as compared to the empty situation. The estimated damping ratios for both cases are compared in Figure 10. A very similar conclusion was reached in [3], where it was concluded that the damping ratios in a sport stadium increases to a significant extent when it is crowded.

f_{PP}(Hz)	f_{ANPSDs}(Hz)	f_{EFDD}(Hz)	f_{SSI}(Hz)	f_{FEM}(Hz)
0.75	0.75	0.7259	0.6904	0.96787
1.25	1.25	1.23	1.228	1.1904
2.05	2.05	2.031	2.043	2.2614
-	-	2.029	2.03	-
2.5	2.5	2.491	2.498	2.2707
3.45	3.45	3.459	3.443	3.4284
3.9	3.9	3.893	3.901	4.2416
4.5	4.5	4.485	4.479	4.4037
5.35	5.35	5.346	5.316	5.7196
5.8	5.8	5.827	5.847	5.9997

Table 1. Natural frequencies. Normal conditions.

f_{PP}(Hz)	f_{ANPSDs}(Hz)	f_{EFDD}(Hz)	f_{SSI}(Hz)	f_{FEM}(Hz)
0.75	0.75	0.7186	0.7146	0.96787
1.25	1.25	1.207	1.247	1.1904
2.05	2.05	2.021	2.021	2.2614
-	-	1.994	2.003	-
2.5	2.5	2.485	2.491	2.2707
3.45	3.45	3.459	3.448	3.4284
3.9	3.9	3.908	3.906	4.2416
4.5	4.5	4.5	4.45	4.4037
5.35	5.35	5.357	5.358	5.7196
5.8	5.8	5.831	5.705	5.9997

Table 2. Natural frequencies. Traffic jam on the bridge.

ζ_{EFDD}(%)	ζ_{SSI}(%)
1.905	2.355
1.317	1.03
0.7157	1.047
1.243	0.8285
0.821	1.067
1.064	0.5442
0.5205	0.5924
0.5074	1.311
1.251	1.073
1.231	2.468

ζ_{EFDD}(%)	ζ_{SSI}(%)
2.068	2.204
2.056	3.391
2.305	2.109
1.843	1.524
1.591	1.483
1.435	1.61
1.055	1.416
1.054	2.955
1.176	1.165
1.402	4.236

Table 3. Damping ratios. Normal conditions (left hand-side) and when half of the bridge is full of cars(right hand-side)

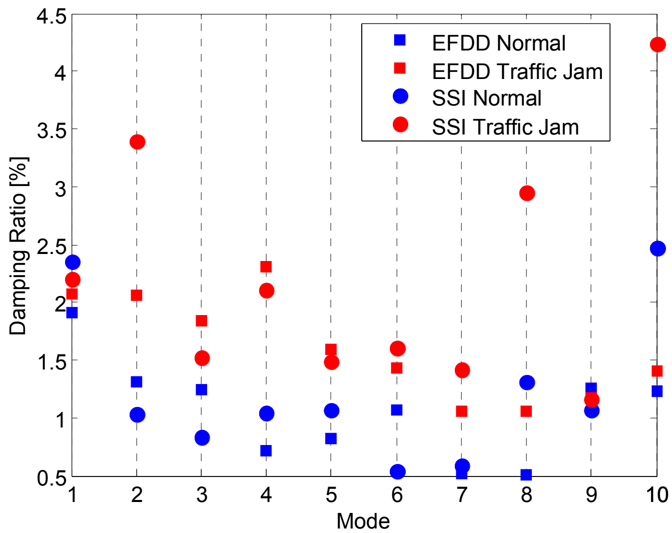

Figure 10. Damping ratios for both conditions.

Mode shapes of the structure identified from experimental modal analysis and from numerical analysis, are shown in Figures 11 and 12.

The experimental results have been compared with the obtained results from the finite element model, to validate the proposed approach. The mode shapes have been compared using the Modal Assurance Criterion (MAC) [24]. The MAC values vary from 0 to 1; a value of one implies perfect correlation of the two modes vectors (one vector is proportional to the other), while a value close to zero indicates no correlated modes (orthogonal modes). The MAC value is defined as:

$$MAC(\phi_{A,k}, \phi_{B,j}) = \frac{(\phi_{A,k}^T \phi_{B,j})^2}{(\phi_{A,k}^T \phi_{A,k})(\phi_{B,j}^T \phi_{B,j})}$$

where $\phi_{C,k}$ is the k-mode of data set C, and ^{T} means transpose matrix.

The obtained MAC matrix between SSI and EFDD, SSI and FEM, and EFDD and FEM, are shown in Figure 13. The mode vector correlation using MAC seems quite good, finding the highest difference in the seventh bending mode shape. The third bending mode is not identified from numerical modal analysis.

One of the objectives of this work is to obtain some reference data to detect the damage of the Barqueta Bridge, by comparing the dynamic parameters that will be obtained in the future with those obtained in this investigation. As previous data of the bridge do no exist, and in order to develop this methodology, experimental results have been compared with those obtained from the FEM model. The damage state has been evaluated using Damage Index Method [25]. In this method, the damage state is controlled by the index β, based on the decrease in modal strain energy between two structural degrees of freedoms, as defined by the curvature of the measured mode shapes. For point j of the structure, the strain energy change between the reference state and the current state, for the i mode shape, is related with curvature changes at the point j. The damage index at point j, for mode shape i, is defined as:

$$\beta_{ij} = \frac{\int_a^b [\psi^{*\prime\prime}(x)]^2 dx + \int_0^L [\psi^{*\prime\prime}(x)]^2 dx}{\int_a^b [\psi^{\prime\prime}(x)]^2 dx + \int_0^L [\psi^{\prime\prime}(x)]^2 dx} \cdot \frac{\int_0^L [\psi^{\prime\prime}(x)]^2 dx}{\int_0^L [\psi^{*\prime\prime}(x)]^2 dx}$$

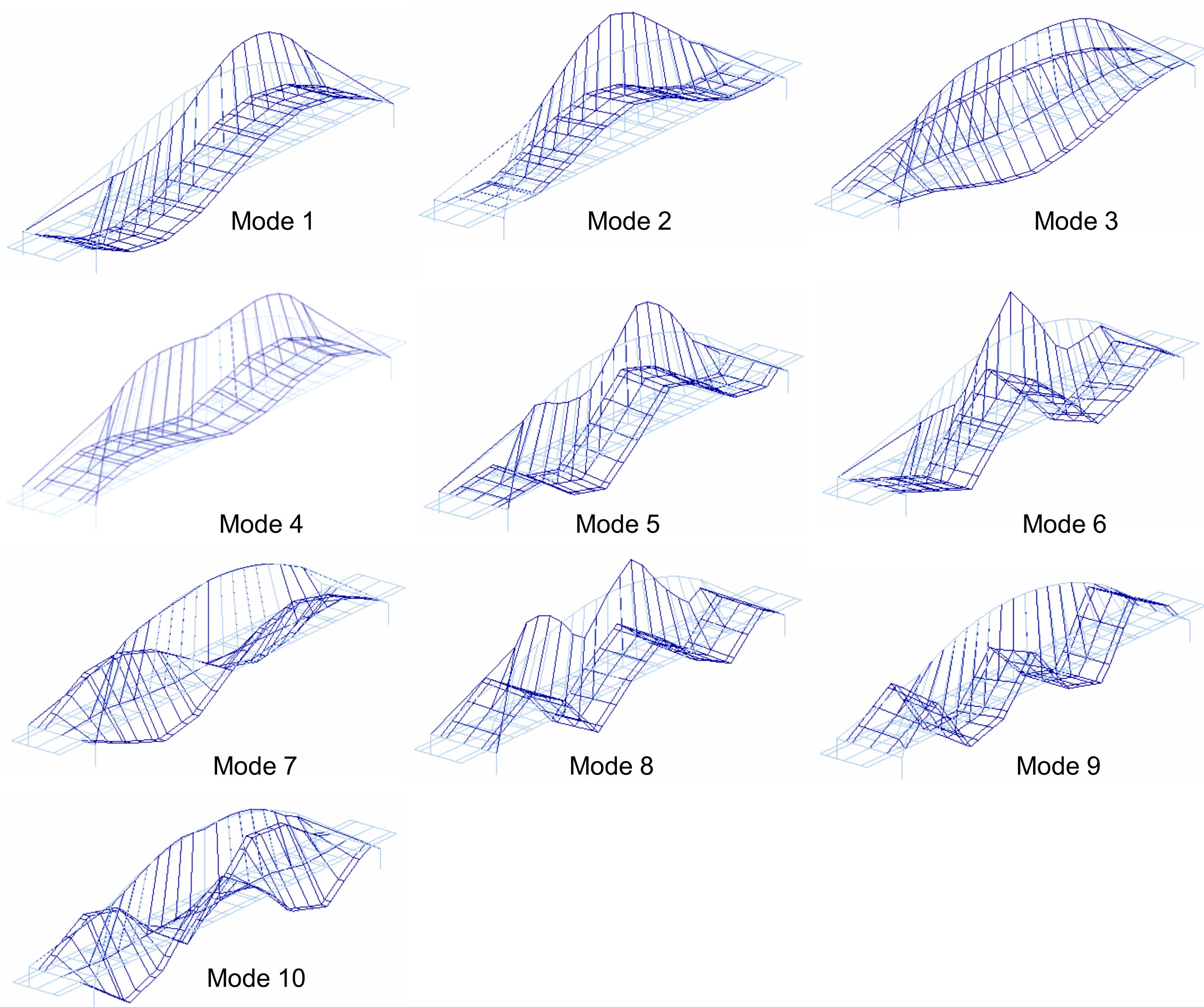

Figure 11. Mode shapes from experimental modal analysis.

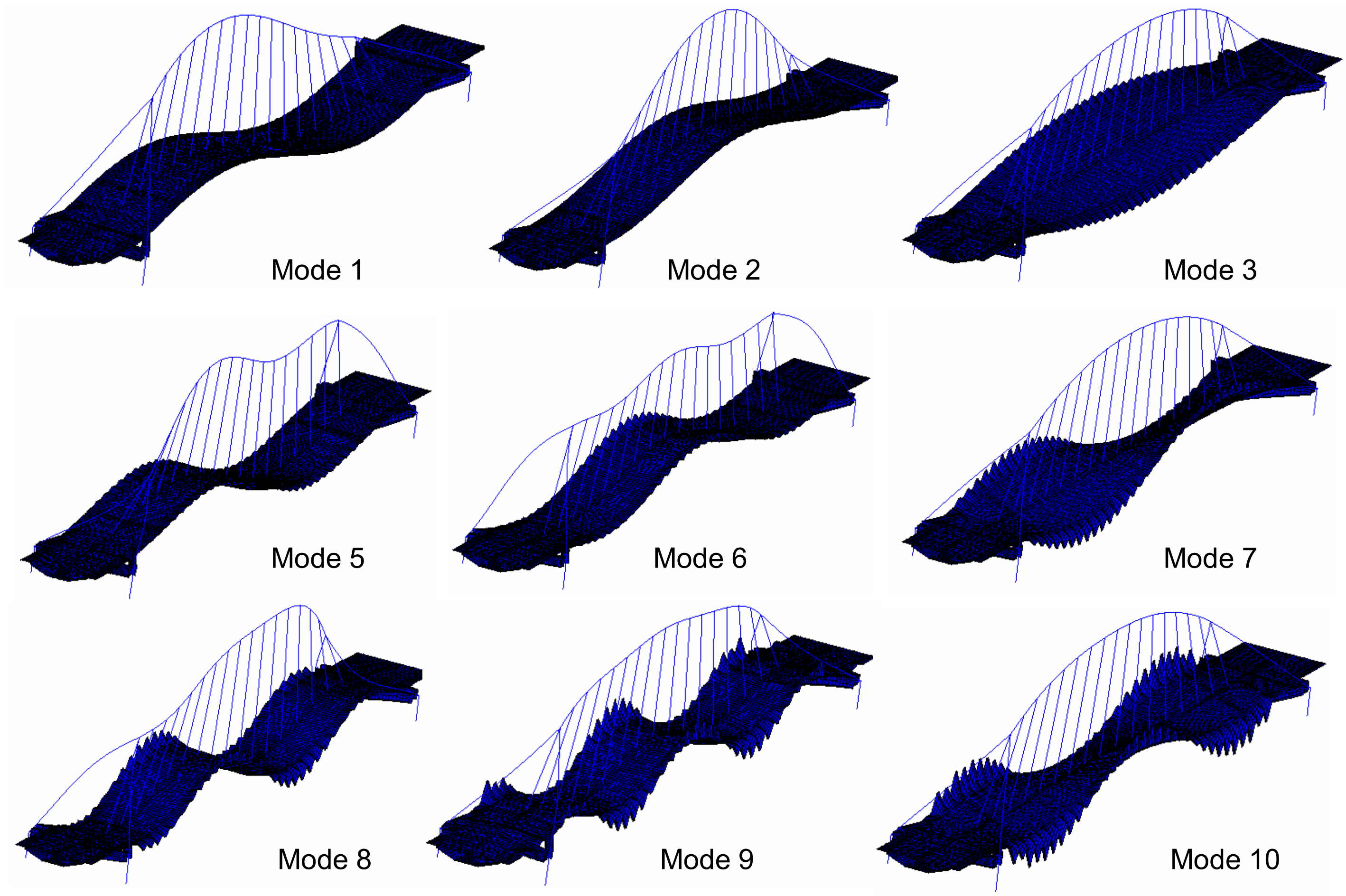

Figure 12. Mode shapes from numerical modal analysis.

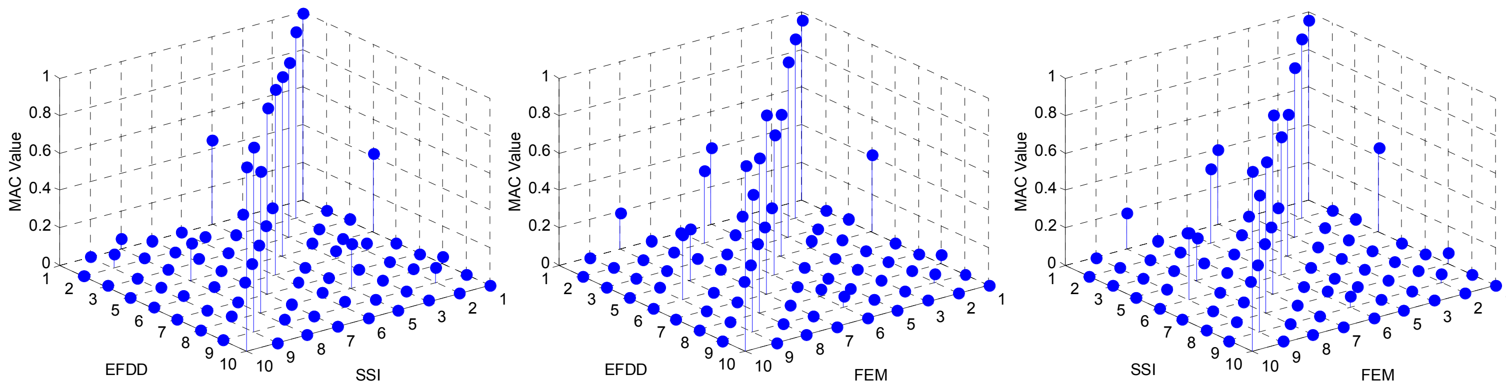

Figure 13. MAC Matrix between SSI and EFDD, SSI and FEM, and EFDD and FEM.

where $\psi''(x)$ and $\psi^{*''}(x)$ are the second derivates of the i mode shape, in the reference state and the current state, respectively; L is the length of the beam element (the structure is discretized by beam elements); and, a and b , are the ends of the element in which the damage is evaluated. When two or more mode shapes are used to identify the state of damage, the obtained indexes for each mode shape are added. At points in which there are not transducers, an interpolation is carried out using spline polynomials to determine the different modal shapes.

The damage index obtained from experimental modal analysis and numerical modal analysis are shown in Figure 14. It can be observed that the β value is practically one at all points. This means a perfect correlation of the two analyses. There is not damage in the structure. It was expected, from previous observation by MAC, that both groups of results were quite well correlated.

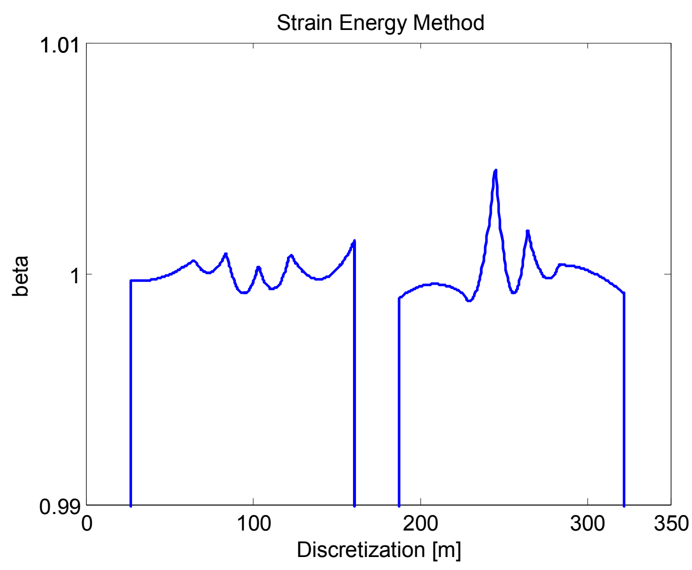

Figure 14. Strain Energy Method.

7. CONCLUSIONS

An experimental and numerical procedure, developed to obtain the dynamic behaviour of an arch bridge with cable-stayed deck has been presented. The modal parameters of the bridge have been identified from ambient vibration using different identification techniques. EFDD and SSI techniques have shown their efficiency to detect close modes.

The dynamic behaviour of the Barqueta Bridge is governed by vertical bending modes and torsional modes in the frequency range of 0-6 Hz. Ten vibration modes have been identified in this frequency range. Consistency in the obtained results from the different records and the coherence information, indicate that the structure had a linear behaviour during tests.

The obtained experimental modes have been compared with the numerical modes using MAC, finding a quite good agreement. This investigation will continue by using the experimental results to update the finite element model.

The vehicle-structure interaction has been studied. At certain hours, have of the bridge is totally covered by vehicles. The vehicles, in addition to add mass to the structure, produce an increase in the estimated damping ratios. It is possible that the vehicles act as an energy dissipation mechanism.

The results of this investigation could be applied to verify the integrity of the structure along its life. As previous data of the bridge do no exist, the results obtained from experimental modal analysis have been correlated to those obtained from finite element analysis using the damage index method. It can be concluded from the analysis that there is not damage in the structure, since the agreement between both results is quite good. This methodology will be employ to study damage state of the structure in the future.

8. ACKNOWLEDGMENTS

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9. REFERENCES

1. B. G. Kemp, C. E. Ventura, D. L. Anderson & A. J. Felber. Ambient Vibration Measurement of Ruskin Dam for Seismic Assessment, 7th Canadian Conference on Earthquake Engineering, Montreal, Canada, pp. 641-648, (1995).
2. J.L. Jensen. System identification of offshore platforms, Ph.D. Thesis, Aalborg University, Denmark, (1990).
3. P. Reynolds, A. Pavic & Z. Ibrahim. Changes of Modal properties of a stadia structure occupied by a crowd, Proceedings of International Modal Analysis, January 26-29, 2004, Deaborg, Michigan.
4. Abdel-Ghaffer A.M. & Scalan R.H. Ambient vibration of Golden Gate Bridge. I: Suspended structure, Journal of Engineering Mechanics, v.111, pp. 463-482, (1985).
5. J. R. Casas. Full-scale dynamic testing of the Alamillo cable-stayed bridge in Seville (Spain), Earthquake Engineering and Structural Dynamics, v. 24, pp. 35-51, (1995).
6. L. F. Troyano, J. M. Armisen, M. A. A. Suárez. The inclined towers of the Ebro and Lerez Bridge, Structural Engineering International, v. 8, pp. 248-260, (1998).
7. J. J. Arenas & M. J. Pantaleón. Puente de Barqueta, Sevilla, España, Structural Engineering International, v. 2, pp. 251-252, (1992).
   http://www.arenasing.com/ingles/menu_1/proyectos/p_urbanos/barqueta/informacion.html
8. http://www.ansys.com.
9. Y. Deger, R. Cantieni, C. A. M. deSmet & A. J. Felber. Finite element model optimization of the new Rhine bridge based on ambient vibration testing, Proceedings of EURODYN´96, pp. 817-822, (1996).
10. C. Gentile & F. Martinez y Cabrera. Dynamic performance of twin curved cable-stayed bridges, Earthquake Engineering and Structural Dynamics, v. 33, pp. 15-34, (2003).
11. D. J. Ewins.Modal Testing, Research Studies Press Ltd. (2000).
12. http://www.matlab.com.
13. http://www.svibs.com.

14. J.M.W. Brownjohn. Ambient vibration studies for system identification of tall buildings, Earthquake Engineering and Structural Dynamics v.32, pp. 71-95, (2003).
15. M.Q. Feng, J.M. Kim & H. Xue. Identification of a Dynamic System Using Ambient Vibration Measurements, Journal of Applied Mechanics v.65, pp. 1010-1021, (1998).
16. J. S. Bendat & A. G. Piersol. Engineering Applications of Correlation and Spectral Analysis. Wiley & Sons, (1980).
17. A. J. Felber. Development of a hybrid bridge evaluation system, Ph. D. Thesis, Department of Civil Engineering, University of British Columbia, Vancouver, Canada, (1993).
18. Z.H. Zong, B. Jaishi, J.P. Ge & W.X. Ren. Dynamic analysis of a half-through concrete filled steel tubular arch bridge, Engineering Structures, v. 27, pp. 3-15, (2005).
19. R. Brincker , L. Zhang & P. Andersen. Modal Identification of Output-Only Systems using Frequency Domain Decomposition, Smart Materials and Structures, v. 10, pp. 441-445, (2001).
20. B. Peeters. System Identification and Damage Detection in Civil Engineering, Ph.D. Thesis, Katholieke Universiteit Leuven, Belgium, http://www.kuleuven.be/bwm/papers/peet00a.pdf,(2000).
21. P. Van Overschee & B. De Moor. Subspace Identification for Linear Systems: Theory, Implementation, Applications. Kluwer Academic Publ.,(1996).
22. P. Andersen. Identification of civil engineering structures using vector ARMA models, Ph.D. Thesis, Aalborg University, Denmark. http://www.aub.auc.dk/phd/department06/text/p andersen01.pdf., (1997).
23. G. De Roeck, B. Peeters, W. Ren. Benchmark study on system identification trough ambient vibration measurements, Proceedings of the 18th International Modal Analysis Conference, San Antonio, Texas, USA., pp. 1106-1112,(2000).
24. R. J. Allemang, D. L. Brown. Correlation coe±cient for modal vector analysis, Proceedings of the 1st International Modal Analysis Conference, USA.,pp. 110-116,(1983).
25. Stubbs N., Kim J.T., Farrar C.J. Field Verification of a Nondestructive Damage Localization and Severity Estimation Algorithm, Proceedings of the 13th International Modal Analysis Conference, Nashville, Tennessee, USA.,(1995).