Static and dynamic testing of a skewed overpass bridge

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ABSTRACT: This paper presents the static and dynamic testing of the skewed overpass PS3. The static load test was performed with truck loads up to a total of 1268 kN. The vertical displacements were measured. The dynamic tests were carried out in three set-ups. 16 modes of the natural vibration of the structure were identified, using the technique of output-only modal identification, and the dynamic characteristics for each mode were estimated. For interpretation of the test results a finite element model for the bridge was built. The experimental results were compared with the analytical values computed by the FE model.

1 INTRODUCTION

The overpass bridge PS3 is located at highway A2 between Lisbon and Algarve, Portugal (Figure 1). In June of 2002, before the opening to the traffic, the static and dynamic tests were carried out. The tests were conducted in order to perform an evaluation of the static behavior of the bridge and to identify its dynamic characteristics, namely, the vibration frequencies, mode shapes and damping ratios. The experimental results of both tests are compared with a finite element model of the bridge. This paper presents the results of the static and dynamic tests in association with the numerical model.

The PS3 is a skewed prestressed concrete overpass bridge, 102 m long, with two central spans of 34 m and two lateral spans of 17 m. The bridge is supported monolithically by tree concrete piers. The axes of the supports are skewed at an angle of 40.19º to the bridge's longitudinal axe (TRIEDE 2000). The deck has 9.40 m width (Figure 2). It is a slab with a central rib, which has a constant height of 0.90 m on the lateral spans and a varied height on the central spans. The maximum height is 1.75 m over the central pier. The piers are rectangular with 0.40 m width. The length is 1.60 m for the lateral piers and 2.00 m for the central one. The central pier has a maxim height of 9.5 m. Figure 3 and Figure 4 present the elevation and plan views of the bridge.

2 ANALYTICAL MODEL

A finite element model for the bridge PS3 was developed to evaluate its response to the static and dynamic tests. The FE model was built via SAP2000 (CSI 2000).

The FE model consists of 399 shell elements and 147 frame elements. The shell elements are used for the lateral cantilevers and the piers and the frame elements are used for the central rib. The connection between the lateral cantilevers and the central beam or the piers was modelled with body constraints. Six link elements were used for modelling the bearings at the abutments. Figures 5 and 6 present the FE model of PS3.

Before the load tests the preliminary FE model was used to estimate the deformation of the structure on static loads and the shapes of the natural vibration modes.

After the tests the FE model was calibrated with the static load tests results at first, and then adjusted to the dynamic characteristics identified from the dynamic tests.

Figure 1. The panorama of the PS3

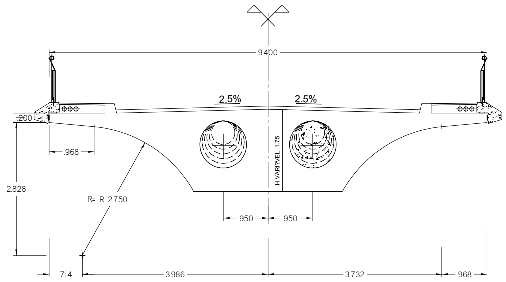

Figure 2. The deck transverse section

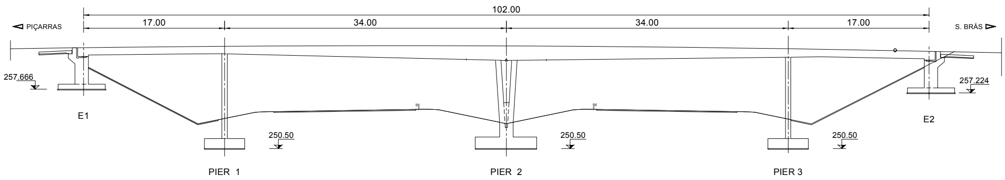

Figure 3. Elevation view of the overpass bridge PS3

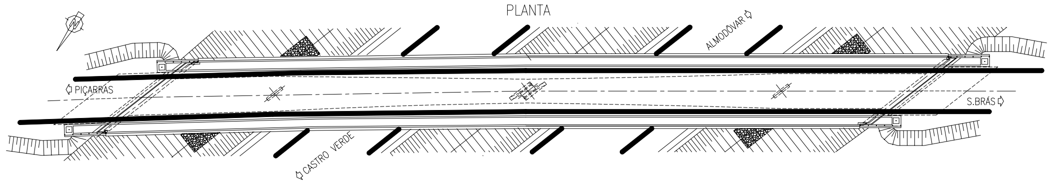

Figure 4. Plan view of the overpass bridge PS3

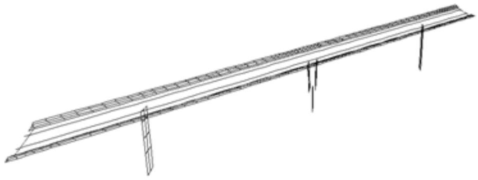

Figure 5. The FE model

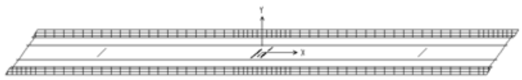

Figure 6. The FE model (plan)

Figure 7. The truck loads on the position 5

Figure 8. The truck loads on the central pier

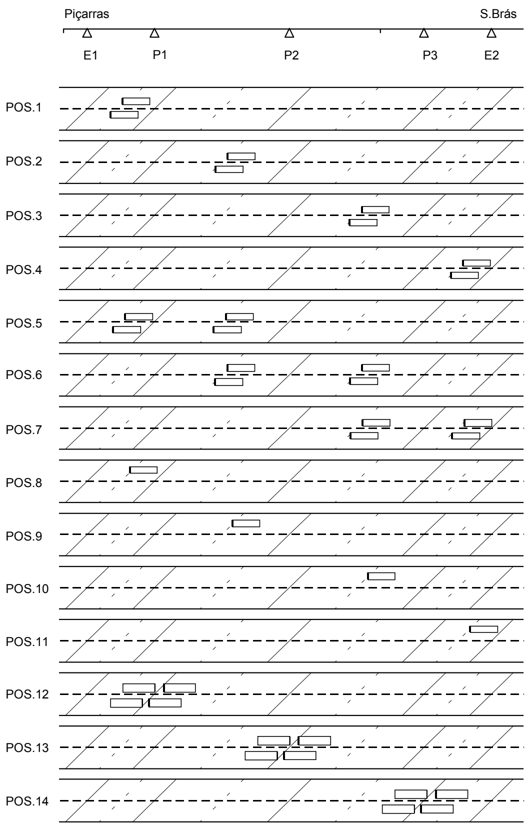

Figure 9. The distribution of truck loads

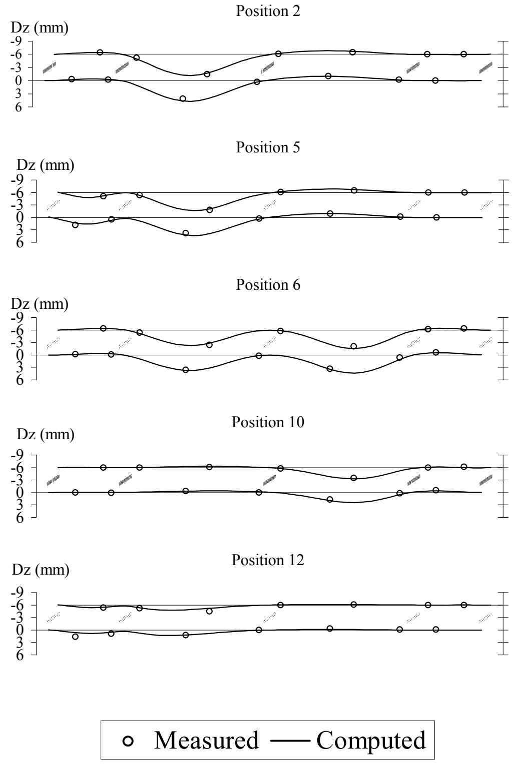

Figure 10. The vertical displacements

3 STATIC TEST

The static test was performed with 4 truck loads up to a total of 1268 kN. These loads were placed in accordance to the load plan that maximizes the most important effects in the structure (Figure 7 and Figure 8), however without inducing unwanted situations of early cracking in the structure. The 14 load positions presented in Figure 9 were performed. During the test vertical displacements at mid spans and at the supports were measured.

The FE model was used for interpretation of the experimental results of the static test. The modulus of elasticity of the concrete was considered as 35 GPa.

Figure 10 shows the vertical displacements of the deck on some load positions and the computed structural deformation. It is evident that a good agreement between experimental values and analytical results has been obtained.

4 DYNAMIC TESTS

4.1 *Testing procedure*

Dynamic tests were performed to obtain experimentally the dynamic characteristics of the structure, namely, the vibration frequencies, mode shapes and damping ratios. During the tests, accelerations induced in the structure by the truck movement were measured using Kinemetrics Uniaxial Episensor (ES-U) accelerometers (Figure 11) and other equipments for signal conditioning and data acquisition (Figure 12) (Rodrigues 2002).

The dynamic tests of PS3 were carried out in three set-ups. During the tests 14 accelerometers were used. Three of them were used as reference sensors, always in the same points, 2 for vertical acceleration (points 13 and 14) and the other for transverse acceleration (point 13). In total, vertical acceleration was measured in 32 points, transverse in 3 points and longitudinal in 1 point. The localization of the points is illustrated in Figure 13. The points instrumented in each test set-up and the corresponding acceleration directions are described in Table 1.

The sampling rate for data acquisition was selected as 200 Hz. The records at each set-up had 163 840 lines corresponding to a time length of about 14 minutes.

4.2 *Modal identification*

The software ARTeMIS – Output-only modal identification was employed for the modal identification of the bridge (SVS 2002). This program allows to accurately estimate natural frequencies of vibration and associated mode shapes and modal damping of a structure from measured response only. It is fast and simple to use.

Before the signal processing for modal identification, the test signals were pre-processed with the following operations (Rodrigues 2002): trend removal; low-pass filtering at 20 Hz with a 4 poles Butterworth filter; decimation of the signals from 200 Hz to 50 Hz. The advantage of decimating the signals was to reduce the size of the records, speeding up all the following computed processes without loosing information in the frequency range of interest.

Figure 11. The accelerometers

Figure 12. The dynamic test

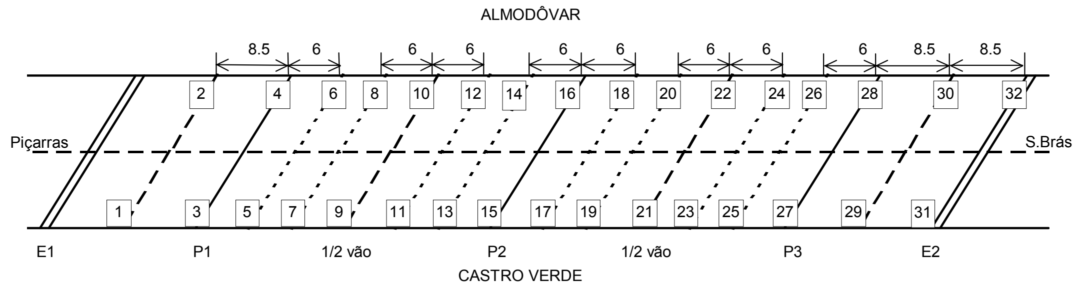

Figure 13. Instrumented points in the dynamic tests

Table 1. Dynamic tests

Set-up	BOX1			BOX2			BOX3			BOX4			BOX5	
	Cn1	Cn2	Cn3	Cn1	Cn2	Cn3	Cn1	Cn2	Cn3	Cn1	Cn2	Cn3	Cn2	Cn3
1	1V	7V	13V (ref.)	2V	8V	14V (ref.)	15V	19V	27V	16V	20V	28V	9T	13T (ref.)
2	3V	9V		4V	10V		17V	23V	29V	18V	24V	30V	21T	
3	5V	11V		6V	12V		21V	25V	31V	22V	26V	32V	21L	

Based on the test data, the spectral densities and correlation functions were estimated. The power spectral density (PSD) matrix was computed from independent samples with 2048 data points each one, with 66.67% overlap. For the sampling frequency of 50 Hz, the frequency resolution of the spectra is therefore 0.024 Hz.

The technique of Frequency Domain Decomposition (FDD) implemented in ARTeMIS was applied to estimate natural frequencies and mode shapes. In this technique the power spectral density (PSD) matrix is decomposed at each frequency line via singular value decomposition (SVD). The singular values (SV) plots, as functions of frequency, estimated from SVD can be used to determine modal frequencies and mode shapes. The peaks of singular values plots indicate the existence of structural modes. The singular vector corresponding to the local maximum singular value is the respective unscalled mode shape (Tamura 2002).

Figure 14 shows the spectra of the first 4 singular values of the PSD matrix. In this figure the natural frequencies identified by FDD are also presented.

The technique Enhanced Frequency Domain Decomposition (EFDD) was mainly developed to estimate the damping of vibration modes (Brincker 2000). Actually the singular values in the vicinity of each natural frequency are equivalent to the PSD function of the corresponding modes. Based on a MAC criterion, each PSD function is identified around the peak of the singular values by comparing the respective mode shape estimate with the singular vectors for the frequency lines around the peak (Figure 15). The piece of the SDOF spectral density function obtained around the peak is taken back to time domain (with the IFFT algorithm), and the frequency and the modal damping ratio are simply estimated from the zero crossing times and the logarithmic decrement of the corresponding SDOF auto correlation function (Figure 16).

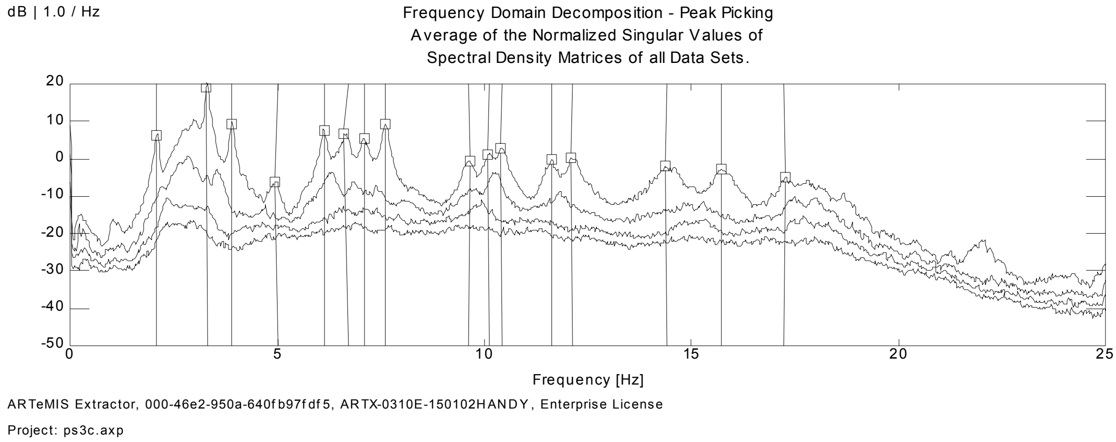

Figure 14. FDD: Spectral of singular values of the PSD matrix and identified natural frequencies

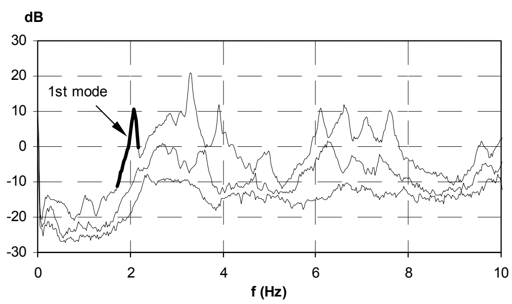

Figure 15. EFDD: PSD identified for 1st mode (MAC=0.94)

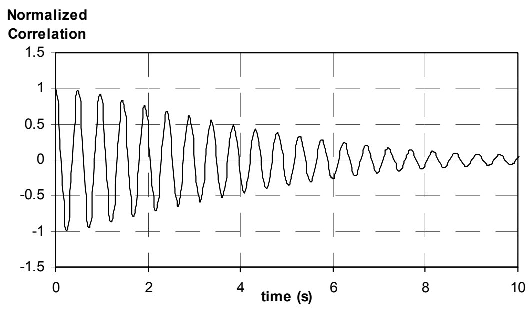

Figure 16. EFDD: Auto correlation function for 1st mode

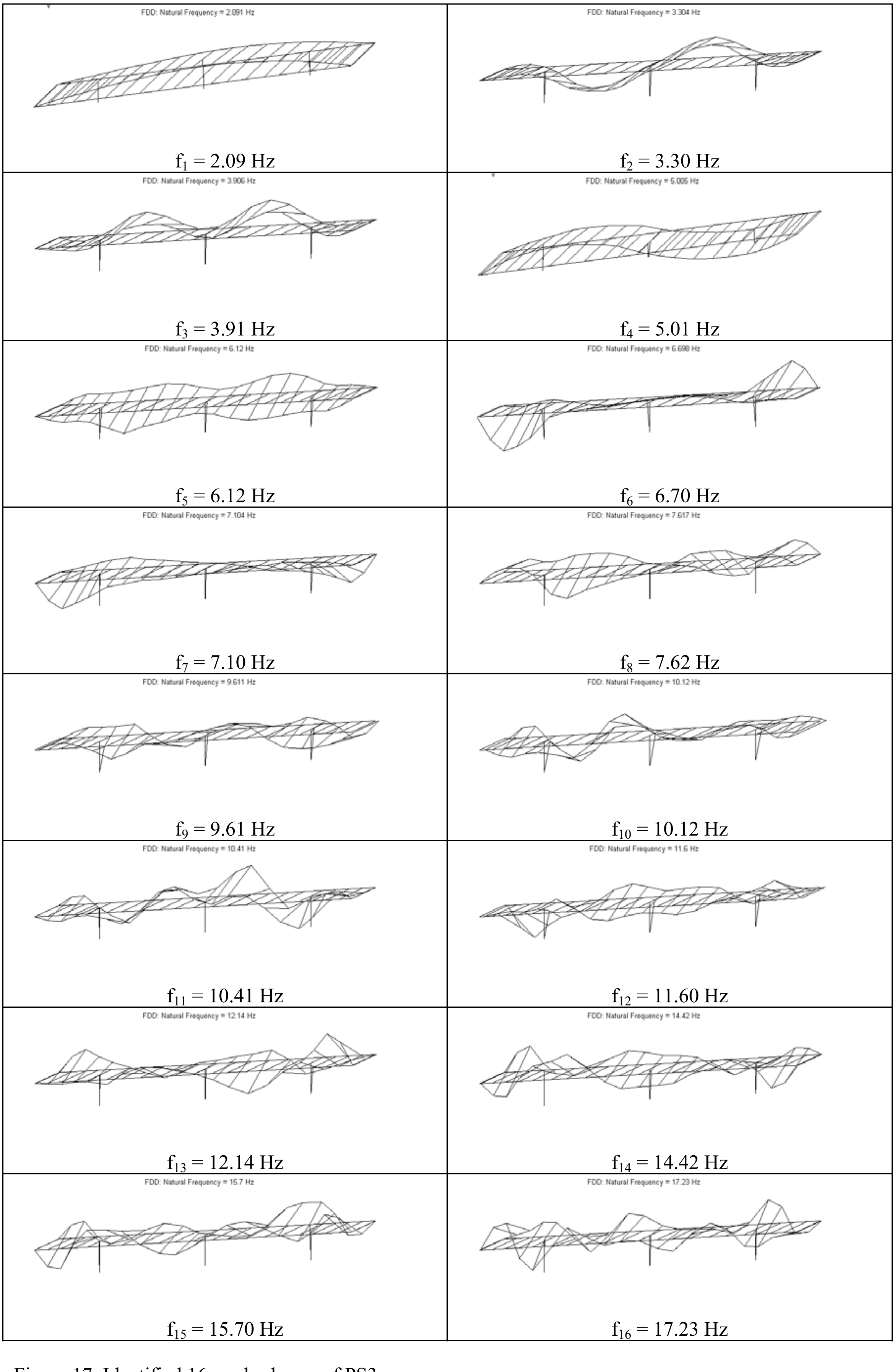

Figure 17. Identified 16 mode shapes of PS3

A total of 16 vibration modes of PS3 were identified from the analysis of the dynamic tests data, using the techniques FDD and EFDD. The 16 mode shapes are depicted in Figure 17. The dynamic characteristics for each identified mode are presented in Table 2.

4.3 *Analysis with the finite element model*

The FE model verified by the static load tests results was also used to interpret the experimental results of the dynamic tests.

The FE model was tuned according to the modal identification from the dynamic tests. However the modulus of elasticity of the structural concrete was kept with the value previously referred (35 GPa). The mass density of the structural elements is 2291 kg/m^3 and the additional mass is 400 kg/m, corresponding to the mass of the lateral sidewalks and the guard rail. The stiffness of the link elements, for modelling the supports at the abutments, was adjusted to fit the computed modal characteristics to the ones identified from the tests.

The natural frequencies computed by the updated FE model were compared to the frequencies identified from the tests and are also presented in Table 2. A graphical comparison between the experimentally identified frequencies (EFDD method) and the frequencies computed with the FE model is also presented in Figure 18. In the case that a perfect agreement had been achieved, all the data points represented in Figure 18 would be in the 45 degrees line that is also represented in that figure. Figure 18 shows, therefore, that a good agreement exists between the experimental and the FE model frequencies, especially in what concerns the vertical and transverse modes.

Table 2. Dynamic characteristics of PS3

Nº	Frequency			Damping	Type of mode
	FDD	EFDD	Model	ξ (%)	
1	2.09	2.11	2.07	1.74	Fundamental transverse mode
2	3.30	3.31	3.23	1.05	Fundamental vertical mode
3	3.91	3.90	3.87	1.38	Vertical mode
4	5.01	4.99	5.11	1.90	Transverse mode
5	6.12	6.12	6.06	1.25	Fundamental torsional mode
6	6.70	6.70	7.90	1.44	Torsional local mode
7	7.10	7.11	7.22	1.20	Torsional mode
8	7.62	7.60	9.15	0.92	Coupled (torsional / vertical)
9	9.61	9.62	--	0.98	Coupled (torsional / vertical)
10	10.12	10.12	9.92	1.15	Vertical mode
11	10.41	10.40	11.06	1.18	Vertical mode
12	11.60	11.62	11.93	1.03	Torsional mode
13	12.14	12.09	14.76	1.20	Torsional mode
14	14.42	14.42	17.69	1.49	Torsional mode
15	15.70	15.75	21.79	1.43	Torsional mode
16	17.23	17.31	24.09	0.92	Torsional mode

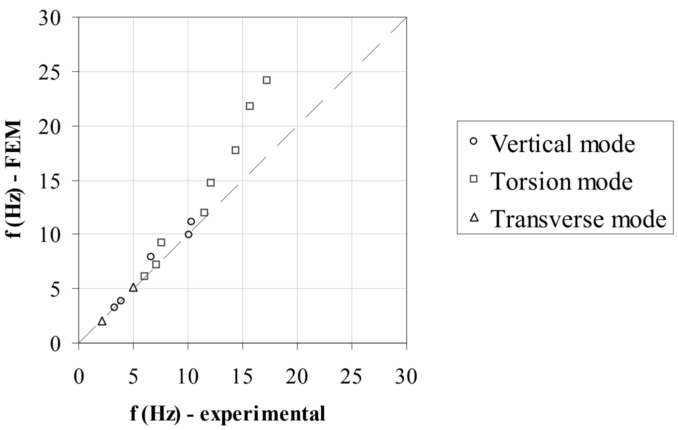

Figure 18. FE model vs. experimental frequencies

5 CONCLUSIONS

The experimental results obtained in the static load test and the dynamic tests have a good correlation with the analytical values computed by FE model.

The information that was experimentally obtained about the structural behaviour of this overpass, is an important contribution to the characterization of its actual condition at the end of the construction and before its opening to the traffic. It is important to note that dynamic tests, similar to the ones that were presented here, can be performed during the lifetime of the structure, without the need to impose traffic restrictions.

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