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Dynamic load tests on the North-South axis cable-stayed bridge with a non-symmetric central pylon.

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Abstract

The new cable-stayed bridge built for the North-South axis road of Bari in order to overpass the railway of RFI and Ferrotramviaria s.p.a. has been recently built and opened to the traffic. The bridge is 626 m long and the central cable-stayed bays have a total length of 225 m. They are supported by cables connected to a central upside down Y-shaped pylon. The peculiarity is that this column is about 60° rotated with respect to the axis of the bridge deck. A dynamic load tests was developed previously to open the bridge to conventional traffic. 26 piezoelectric accelerometers have been utilized in different positions of the cables-stayed bays to record the accelerations produced by environmental forces and by the impact produced by a loaded truck passing over a bump. Operational Modal Analysis has been applied by mean of Artemis software to determine the first fundamental frequencies and the mode shapes. The main frequency of this non-symmetric pylon is the main frequency of all the stayed bridge.

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Keywords: cable-stayed bridges, antisymmetric deck, dynamic identification tests, FE modelling.

1. Introduction

In the field of structural engineering it is very current the interest towards the realization of lighter and slender structures. This trend is quite common in bridges for their flexible structure and gives the designers the awareness of the central role played by the study of the dynamic structural behavior among the usual design parameters. Dynamic identification techniques allow to determine the modal structural parameters by mean of data experimentally acquired. The values of the dynamic characteristics of the structure, that is the frequencies, modal shapes and damping factors,

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are then compared with those numerically obtained from finite element (FE) models. Dynamic identification has been applied to footbridges assuming different velocity speeds of a group of people passing over it [1-3]. Free vibration was also detected measuring the accelerations after the group had passed over.

The technique utilized in presence of an unknown environmental force is the operational modal analysis (OMA). It has been successfully applied in slender structures such as towers [4-11], chimneys [12], minarets [13-14] and bridges [15-16] in order to determine their dynamic characteristics (i.e. natural frequencies and mode shapes). Beyond the sustainability of the test (that does not interfere with the normal use of the infrastructure since it is performed by just measuring the response in operational conditions), ambient vibration tests turns out to be especially suitable to flexible systems since the most significant modes of vibration in the low range of frequencies are excited with sufficient energy by the environmental actions and a large number of normal modes can be identified from ambient vibration tests of these bridges.

The present paper analyses the dynamic behavior of a new cable-stayed bridge of Bari, Italy, realized on the North-South road axis, with the aim of a dynamic characterization of the structure. The specific geometry of the pylon, situated in the centre, with two non-symmetric supports on the bridge generated a non-symmetric dynamic behavior to be analyzed. A series of tests on the central cable-stayed decks have been performed. Ambient vibrations (due to wind) and the vibrations due to a fully loaded truck passing over a bump (i.e. impact loading) have been considered to obtain the behavior of the bridge. In order to check the dynamic design analysis, frequencies and vibration modes were experimentally determined.

2. Dynamic load tests

In the dynamic load tests, the accelerations were recorded in the 26 channels according to the marked positions and directions in Figure 1(a). The positions that do not match any arrow indicates a vertical accelerometer. With the aim of introducing an excitation and register the free vibration of the deck, two set of bumpers (one made in concrete and the other elastomeric) were casted as shown in Figure 1(b), and a series of tests were carried out with a 38t truck transiting on the deck at a variable speed (between 10 and 50 km/h depending on the test) and passing over a bumper to produce an impact force on the deck, Figure 1(c).

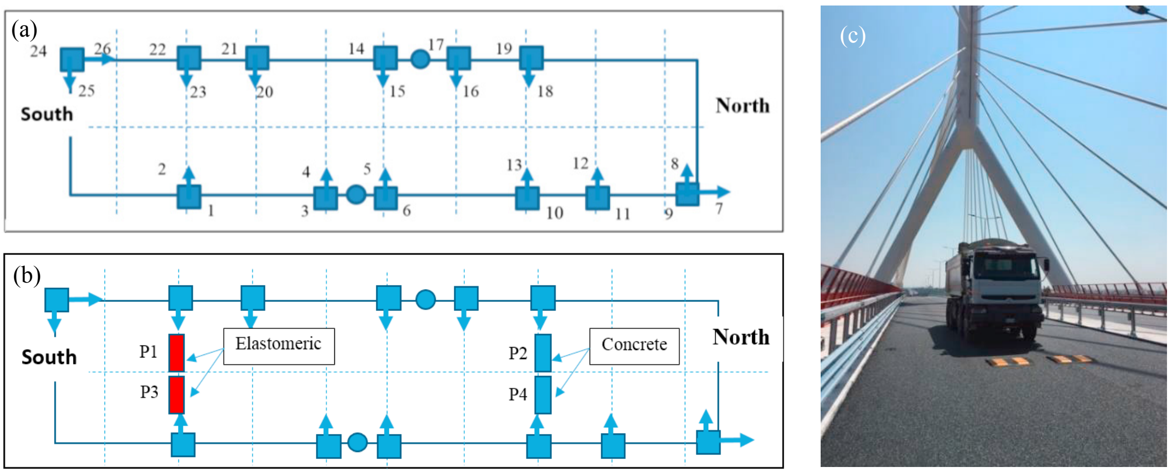

Fig. 1. (a) Accelerometers, and (b) bumpers location on the bridge. (c) 38t truck in the elastomeric bumpers of position P1.

2.1. Results

From the time-history plots, the maximum values of the accelerations at each test have been determined. They were used to perform an Operational Modal Analysis (OMA) and thus to determine the principal frequencies of the bridge, its mode shapes and the damping factors of each vibration mode.

The Environmental Modal Identification, also known as OMA, aims to identify the modal properties of a structure based on vibration data collected when the structure is in service, and neither initial excitation is applied nor any known artificial excitations. In an environmental vibration test, the structure may be subject to different types of excitation not measured, but assumed as 'broadband random'. The latter is a notation that must applied when an environmental identification method is utilized. Regardless of the method, however, the correct modal identification technique requires that the spectral characteristics of the measured response reflect the properties of the modes, rather than those of the excitation.

OMA methods can be classified according to two aspects: 1) perform in the frequency domain or in the time domain, 2) are methods of Bayesian or non-Bayesian type. They make use of some statistical estimating theoretical properties with notes for identification, such as, for example, the correlation function or the spectral density of the measured vibrations. The common non-Bayesian methods consider the stochastic subspace identification (time domain) and the decomposition of the frequency domain (frequency domain). Bayesian methods have been developed both in the time and in the frequency domain.

With the data recorded during the different stages of loading, the frequency analyses were performed through the Fourier analysis. Figure 2(a) shows the frequency response after the impact of the truck on bumper P1, identifying the predominant frequencies. Figure 2(b) is obtained applying EFDD (Enhanced Frequency Domain Decomposition) method, and determining the stable vibration frequencies, which values are collected in Table 1. In addition to the value of 0.576 Hz totally identified in the analysis, other frequencies are also identified: 2.7 Hz is the value of the predominant frequency in all the analyses. These values differ with respect to the analysis carried out in the structural design analysis of the bridge; therefore it was necessary to revise the analysis from a dynamic point of view.

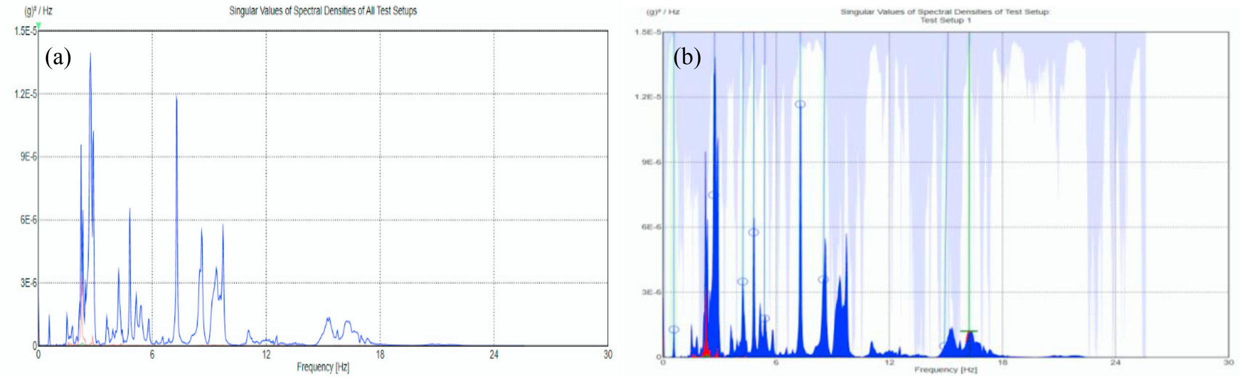

Fig. 2. (a) FFT analysis results after an impact generated by a 38t truck at 25 km/h. (b) EFDD analysis, stabilized frequencies in time.

Table 1. Fundamental frequencies obtained by EFDD identification technique.

Mode	1	2	3	4	5	6	7	8	9
Frequency (Hz)	0.576	2.738	4.238	4.815	5.396	7.286	8.591	14.926	16.232

The results of the dynamic tests, applying EFDD gave different modes of vibration associated with each identified frequency. The plots included in Figure 3(a) show the vibration modes; the latter can be identified in the cases of both environmental vibration and truck impact excitation. The 1st mode of vibration, with a frequency of 0.576 Hz, is an antisymmetric mode of bending of a two spans continuous beam subject to torsion, no doubt originated from the antisymmetry of the central supports of the bridge. The 2nd mode of vibration (2.738 Hz) is associated with a global mode of bending of the deck with a torsion component. The 3rd mode of vibration (4.238 Hz) is associated with a global mode of bending of the deck with a torsion component. The 4th mode of vibration (4.815 Hz) is basically a bending mode.

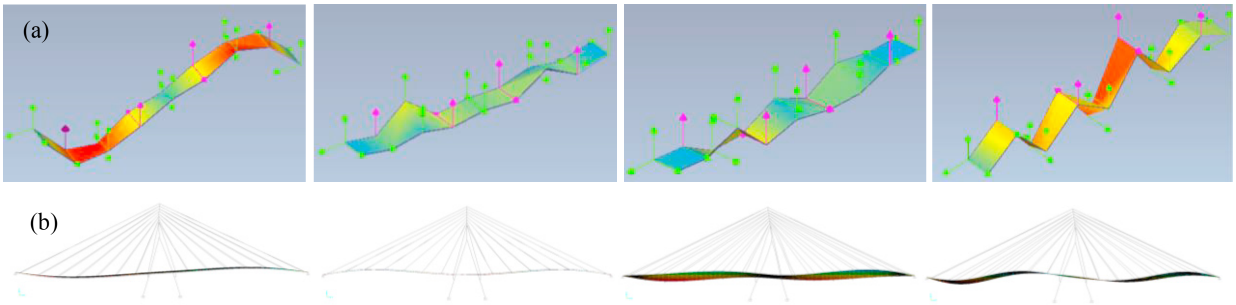

Fig. 3. (a) Experimental Modal shapes of modes 1 to 4. (b) Numerical modes: 1-2 pure bending; 2, pure bending; 3-4 torsion-bending.

3. Simplified structural model

The present analysis was developed with the aim of comparing the results of stiffness and masses considered in the design analysis of the bridge with the results obtained from the experimental test. The initial design numerical Finite Element Model (FEM) was realized with FEM software. In the initial model, Figure 4(a) all the spans of the bridge have been considered. In this model, 30 vibration modes were considered for the modal analysis, actually driving 86.39% of the mass in the X direction, 91.04% in the Y direction and 39.85% in the Z direction.

Practically, all the vibration modes are associated with a torsion component due to the asymmetry of the supports of the central pylon of the bridge. In order to provide more useful information about the first mode of vibration. Since the modal analysis, results of the complete bridge are difficult to identify with the dynamic tests; distinct parts of the bridge have been analyzed separately.

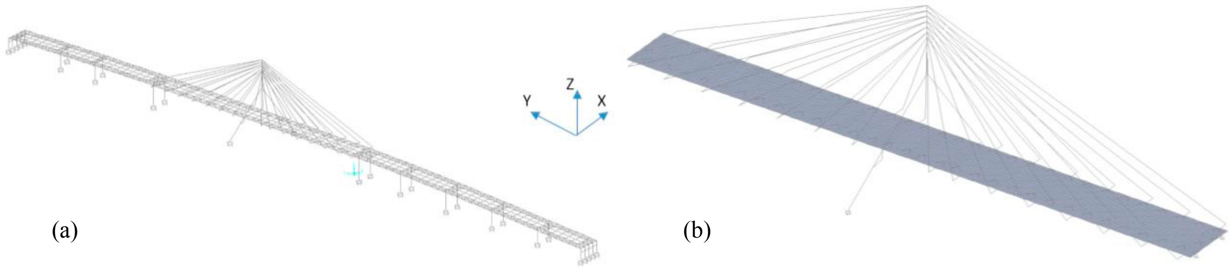

Fig. 4. Structural numerical models: (a) full bridge model; (b) simplified model for cable-stayed span..

Figure 5 shows the model and the vibration modes of the central pylon of the bridge. Table 2 shows the results of the modal mass mobilized for each mode. From the numerical model provided all elements have been eliminated except those corresponding to the central pylon, thereby maintaining the stiffness indicated by the designer. It is observed that the main frequency of the pylon is equal to about 0.45 Hz, the same order of the first frequency of the bridge; for this reason it is identified that globally the first frequency of the bridge is affected by the stiffness of the pylon and the effect of the force of the cables on the deck.

These results are consistent with those obtained in the experimental test for a frequency of 0.576 Hz; the mode shapes are very similar, despite that, for this mode, the constructed bridge is a bit stiffer than the analysis model. The results obtained in the dynamic tests are between 60% and 115% of those obtained in the dynamic calculation; therefore they can be accepted as valid.

Fig. 5. Simplified model of the central bridge pylon.

Table 2. Main frequencies and modal participating mass ratios of the central pylon.

Mode	Period	Frequency	UX	UY	UZ	RX	RY	RZ
1	2.25 s	0.45 Hz	13.3%	39.5%	0.0%	7.6%	20.4%	9.0%
3	0.38 s	2.61 Hz	4.7%	10.5%	0.0%	0.1%	1.4%	3.3%
5	0.19 s	5.16 Hz	36.8%	7.5%	0.0%	0.0%	1.1%	39.0%
8	0.11 s	9.37 Hz	23.0%	7.2%	0.0%	1.2%	29.9%	24.9%

To define the position and the number of the accelerometers to utilize, before the dynamic on-site tests a simplified model of the bridge has been realized by mean of FEM software. Based on the model used by the designer, the side bays have been eliminated, and only the cable-stayed central ones have been considered. A reinforced concrete slab of 30 cm thickness on all the deck is then introduced, eliminating the masses corresponding to the weight of the slab. The model is shown in Figure 4(b).

Table 6 shows the first four vibration frequencies obtained from the dynamic load test, the frequencies presented in the original design, and those obtained from the simplified numerical model. In Table 6 the values of the first experimental fundamental frequency of the bridge is very close to the one numerically obtained, differently from what happens for the other vibrational modes that probably are conditioned by factors such as the stress values in the cables.

Table 7. Numerical and experimental vibration frequencies.

Model	Mode 1	Mode 2	Mode 3	Mode 4
Experimental	0.576 Hz	2.738 Hz	4.238 Hz	4.815 Hz
Design	0.40 Hz	0.58 Hz	0.73 Hz	0.83 Hz
Numerical with slab	0.41 Hz	0.69 Hz	0.90 Hz	1.24 Hz

4. Conclusions

In the present paper, the results relative to the dynamic load tests on the new cable-stayed bridge in Bari (Italy) are shown and discussed. All the data obtained during the dynamic tests have been shown, as well as data processing: natural frequencies, damping factors and first mode shapes of vibration of the structure. The maximum recorded acceleration value does not exceed 0.152 g.

The recorded values of the natural frequencies of the structure at the points where the accelerometers are placed, correspond to modes of bending-torsion, pure bending and/or torsion. These values are in agreement with the dynamic numerical model, where the contribution to the stiffness and the mass of the mixed structure are considered. The results obtained for the damping factor show relatively low values, equal to 0.294% for the predominant mode and 2.43 % for the lowest mode. These values are close to the values considered to be normal in the scientific literature, between 0.5% and 2%.

The effect of the non-symmetric pylon generates a main frequency with bending and torsion effects coupled, non-usually when the pylon supports on the bridge are symmetric.

It is important to note that in the original design of the cable the analysis model does not present any pre-assigned stress and the axial force comes exclusively from their weight. Furthermore, it should be noted that in the analysis the maximum number of modes considered (30 modes) could not mobilize more than 40% of the vertical mass of the bridge. Finally, it is important to point out that in the design model analysis only a 5% damping factor value has been assumed for the response spectrum, a value indicated by the code exclusively for the ultimate limit state, not for the service state. No other damping values have been considered for the other analyses.

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