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Experimental dynamic assessment of a cable-stayed bridge

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ABSTRACT: This work presents the results of the experimental dynamic assessment of the cable-stayed bridge that crosses the Garigliano river (Italy), a strategic infrastructure built in 1993 along the high-speed road connecting Naples and Rome. The bridge consists of two equal spans whose length and width are equal to 90 m and 26.1 m, respectively. The precast pre-stressed concrete box girder is supported by 18 couples of cables (nine couples for each span). These cables start from a central pylon whose height above the deck is equal to 30 m. The dynamic response of the deck subjected to ambient vibrations was monitored by placing several accelerometers along its longitudinal axis in order to identify the global modal characteristics (i.e., natural frequencies, mode shapes and damping ratios). The cables were equipped with accelerometers as well and the acquired measurements have been elaborated by means of some simplified analytical formulations for estimating the traction force. Finally, experimental results have been compared with initial design requirements and the outcomes of a previous dynamic monitoring campaign.

1 INTRODUCTION

Among the current civil engineering applications of structural dynamic monitoring techniques, bridges undoubtedly attract the greatest interest worldwide. The implementation of such non-destructive experimental testing method can be required to confirm design assumptions and results, to calibrate numerical models (Bayraktar et al. 2009), to assess the performance and the health of the structure (Reynders et al. 2007, Quaranta et al. 2012), to plan retrofitting interventions (Hag-Elsafi et al. 2012). The installation of a sensor network for dynamic monitoring requires proper solutions about several technological issues, including sensor selection and placement, power supply, data transmission and analysis (Lynch 2007, Quaranta et al. 2014). The presented work is concerned with the experimental dynamic testing of an existing cable-stayed bridge that crosses the Garigliano river (Italy). Once the bridge structure and the monitoring network have been illustrated, the modal characteristics of the deck extracted from ambient vibrations are provided. Tensions and forces acting in the cables have been also estimated using their dynamic response under impulse force. Finally, experimental results have been compared with initial design requirements and the outcomes of a previous dynamic monitoring campaign.

2 BRIDGE DESCRIPTION AND DYNAMIC MONITORING

2.1 Description of the bridge structure

An overview of the bridge is given in Fig. 1 whereas its geometry is illustrated in Fig. 2.

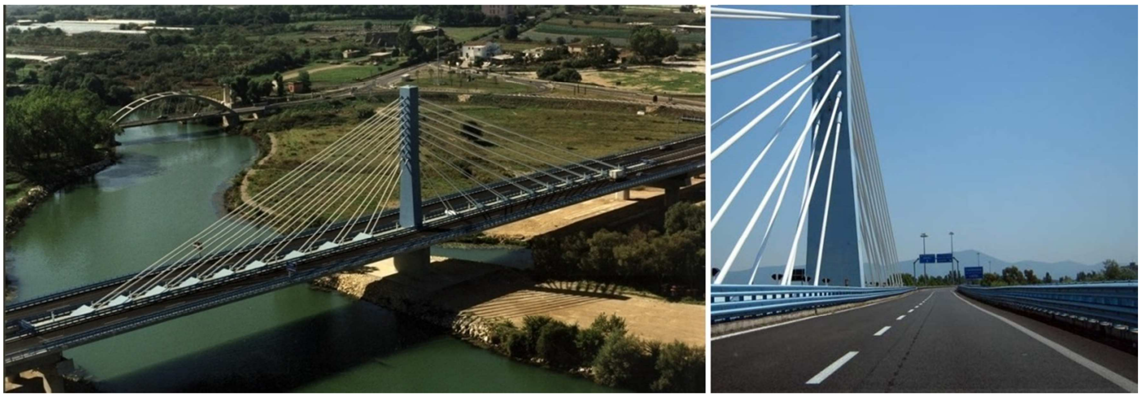

Figure 1. The cable-stayed bridge crossing the Garigliano river.

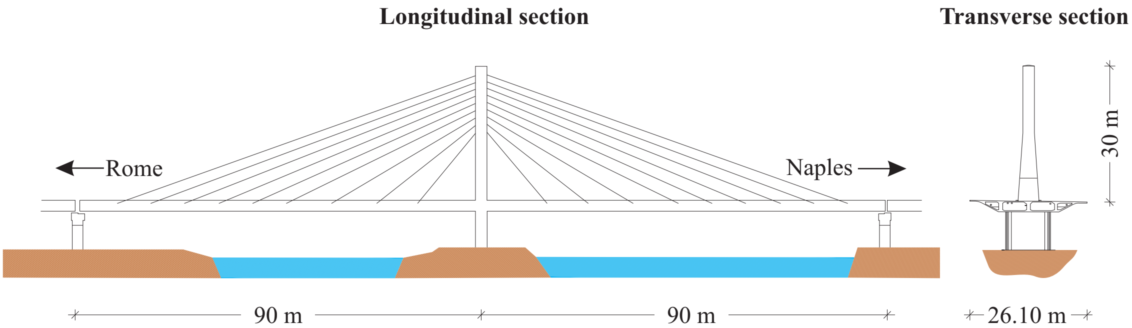

Figure 2. Main geometrical characteristics of the bridge.

The bridge was built on 1993 and consists of two equal spans whose length is 90 m. The width of the deck is 26.1 m and its height is 2.45 m. The precast multicell box girder deck was built by assembling *in situ* 35 sections for each span. The bridge deck is provided with four pre-stressing systems, of which three acting in the longitudinal direction and one in the transversal direction. The girder is constrained at the central tower, simply supported at the other ends and suspended by 18 couples of cables (nine couples for each span). The cables start at different sections of the pylon. The height of the central pylon is 10.85 m from the foundation to the deck extrados and 30 m from the deck. The lower part of the pylon (from the foundation to 5 m above the deck) is made of concrete, with a transversal section 4.6 m × 2.5 m at the base and 4 m × 2.5 m at 5 m above the deck. The central part of the pylon (starting from 5 m above the deck, up to 10 m) is a steel box beam with a transversal cross section varying from 4 m × 2.5 m to 2.9 m × 2.5 m. The remaining part of the pylon has a constant transversal section. The cross sectional areas of the two shortest cables is 67.5 cm^2. The longest cable has a cross sectional area of 70.5 cm^2. The others have a cross sectional area equal to 82.5 cm^2.

2.2 *Dynamic monitoring*

The dynamic monitoring was conducted on January, 2014. The layout of the dynamic monitoring campaign is illustrated in Fig. 3.

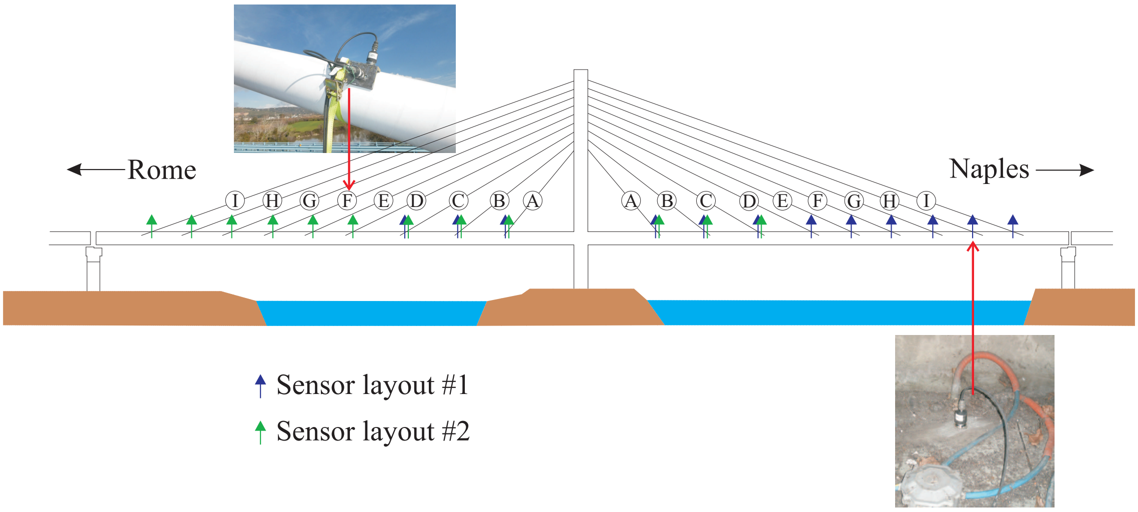

Figure 3. Sensor network layouts for the dynamic monitoring of deck and cables.

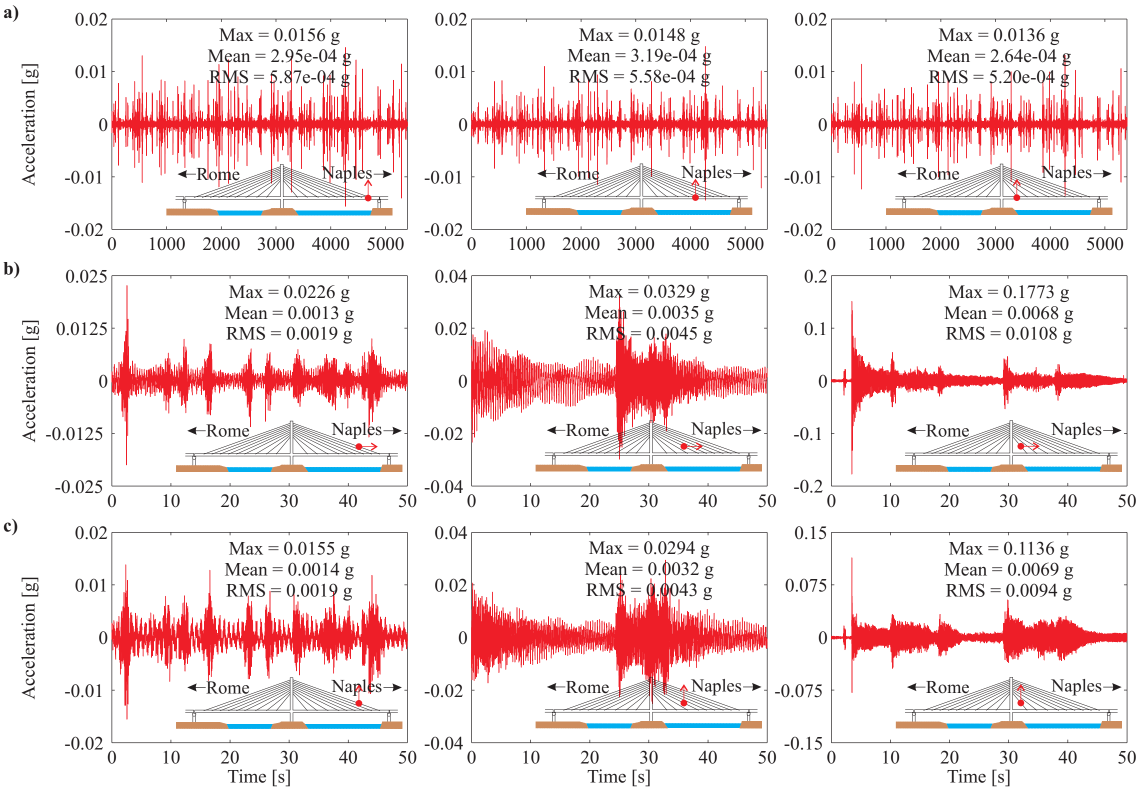

Figure 4. Some recordings of the bridge under ambient vibrations: a) vertical deck response, b) horizontal cable response, c) vertical cable response.

The sensor network consists of piezoelectric uniaxial accelerometers PCB 393B12 with a sensitivity equal to 10 V/g and a frequency range 0.15 Hz - 1000 Hz. The sampling rate is 200 Hz. The vertical vibrations of the deck were acquired by installing the accelerometers within the structure along the longitudinal axis. The dynamic monitoring of the deck was implemented by means of two sensor layouts because of the limited number of sensors, whereby six accelerometers were left on the previous positions to keep track of the phase. Both horizontal and vertical accelerations were acquired for each cable by placing two accelerometers at an average height of 3.8 m above the deck extrados. The length of the recordings for the bridge deck is about 90 min in order to perform a reliable output-only operational modal analysis. The time window of the measured cable response is about 50 s because the frequency-domain analysis is considered to estimate the cable tension. Some recordings of deck and cables response under ambient vibrations are shown in Fig. 4.

3 EXPERIMENTAL DYNAMIC CHARACTERIZATION OF THE BRIDGE

3.1 Identified natural frequencies, mode shapes and damping ratios of the deck

The output-only operational modal analysis of the deck response under ambient vibrations was performed by means of Enhanced Frequency Domain Decomposition (Brincker et al. 2001) and Stochastic Subspace Identification (Van Overschee and De Moor 1996) techniques, see for instance (Nisticò et al. 2015) for a short review. The analysis of the obtained results has allowed the identification of six bending mode shapes, three symmetric modes and three anti-symmetric modes (Fig. 5). Identified natural frequencies range from 0.92 Hz to 6.39 Hz whereas the damping ratios are equal or less than 3%. Since the accelerometers were placed along the longitudinal axis of the bridge, torsional modes cannot be identified.

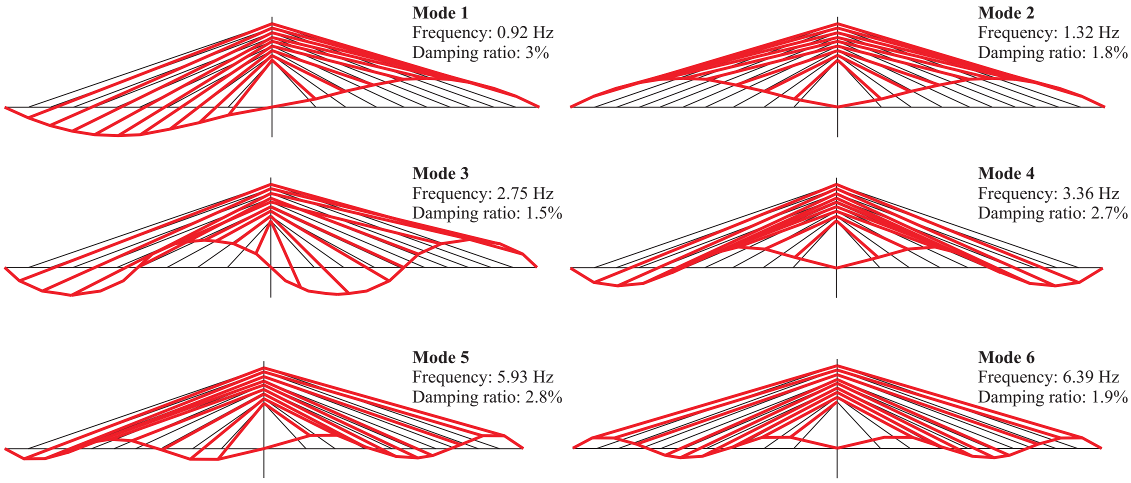

Figure 5. Identified modal characteristics of the bridge deck.

3.2 Estimation of the cables tension

Vibration-based methods for estimating the cable tension are widely adopted in practical applications. Their simplicity stems from the fact that a frequency-domain analysis only is required. Herein, the cable tension is estimated from vibrations induced by an impulse force. The classical bent beam model as well as the practical formulations presented in (Zui et al. 1996) and (Fang and Wang 2012) have been considered. Final results are listed in Tab. 1.

Table 1. Estimated tension in the cables using impulse-induced response.

Side	Cable	First frequency [Hz]	Design tension [MPa] Lower bound	Design tension [MPa] Upper bound	Estimated tension [MPa] Beam model	Estimated tension [MPa] Fang and Wang (2012)	Estimated tension [MPa] Zui et al. (1996)
Rome	A	6.42	645.6	733.5	748.2	593.5	613.0
	B	4.2	635	724.5	687.9	587.4	597.4
	C	3.24	622	723	673.5	599.1	605.3
	D	2.54	635	728,5	668.0	609.8	614.1
	E	2.16	643	727	657.9	608.5	611.8
	F	1.88	652	724.5	650.7	607.7	610.4
	G	1.66	651	711.5	643.9	606.0	608.2
	H	1.5	637.5	707	650.2	616.0	618.0
	I	1.26	600	685.5	572.6	543.4	545.1
Naples	A	6.14	645.6	733.5	683.1	535.0	554.3
	B	4.2	635	724.5	687.9	587.4	597.4
	C	3.22	622	723	665.2	591.2	597.4
	D	2.54	635	728.5	668.0	609.8	614.1
	E	2.16	643	727	657.9	608.5	611.8
	F	1.9	652	724.5	664.6	621.2	624.0
	G	1.68	651	711.5	659.5	621.1	623.4
	H	1.5	637.5	707	650.2	616.0	618.0
	I	1.28	600	685.5	591.0	561.3	563.0

Based on the results given in Tab. 1, the horizontal and vertical components of the cables force have been determined (Fig. 6).

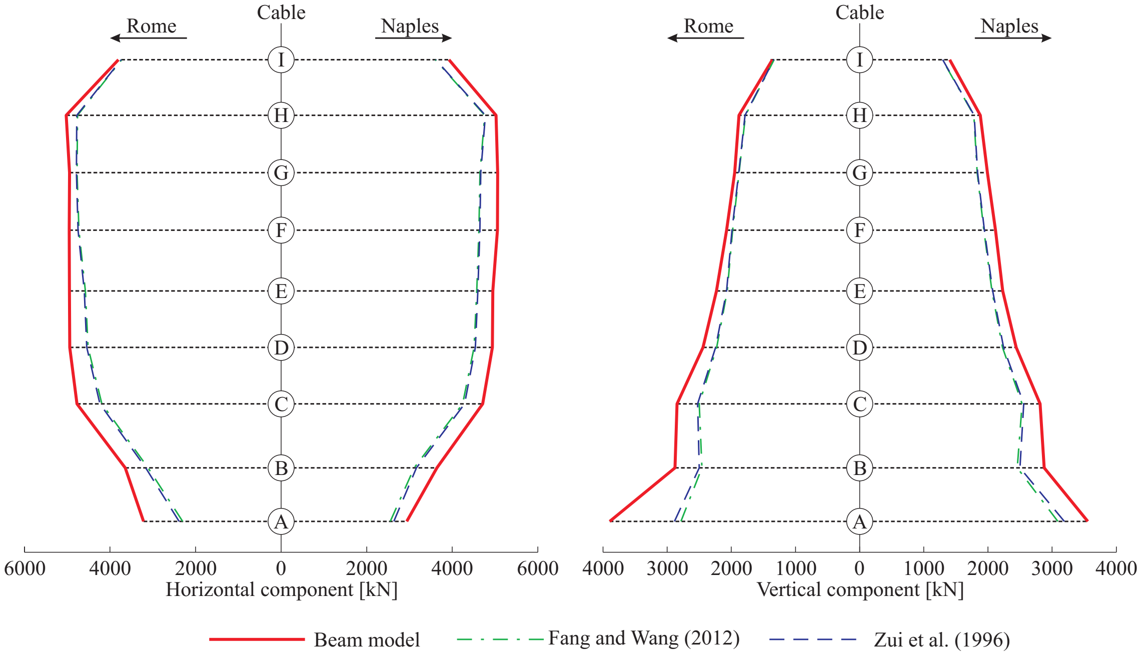

Figure 6. Horizontal and vertical components of the cable force using impulse-induced response.

Results in Tab. 1 and Fig. 6 shown that the formulations by (Zui et al. 1996) and (Fang and Wang 2012) lead to similar results whereas the bent beam model provides a slightly larger estimate of the tension. It can be noted in Tab. 1 that the estimated tensions lie within the prescribed design ranges or are very close to them. It is possible to observe from Fig. 6 that symmetric cables are subjected to a near identical traction force.

3.3 Comparison with previous dynamic monitoring campaign

Since a large number of dynamic monitoring campaign have been conducted in the last decades, ambient vibration re-testing is becoming a feasible way in order to identify changes in the modal parameter (Brownjohn et al. 2010) which, in turn, can be eventually adopted for damage detection (Quaranta et al. 2012, 2015). In this case, another dynamic characterization was performed for the same cable-stayed bridge by Clemente et al. (1998), who deployed the dynamic monitoring campaign in two different layouts. In the first configuration, four sensors were installed on the girder and three sensors were placed on the pylon (two at the top and one at half height). In the second configuration, thee sensors were used on the girder, two at the top of the pylon and three on the cable D. Among the sensors installed on the girder, two of them were placed on the bridge sides and the others along the longitudinal axis. Therefore, compared to this study, Clemente et al. (1998) were able to place some sensors on the bridge sides, which are useful for the identification of the torsional motion. Conversely, a finest spatial resolution of the deck motion can be obtained in this study because of the larger number of measurement points along the longitudinal axis. Additionally, only one cable was monitored by (Clemente et al. 1998) whereas all the cables were examined in this study. Another difference between this study and the one by Clemente et al. (1998) deals with dynamic loading conditions. While ambient vibrations were considered in this study for the identification of the bridge deck, within the work by Clemente et al. (1998) the structure was subjected to the impact due to a lorry of about 300 kN driving over a bar placed on the paving. Three wooden bars of different height (3 cm, 8 cm and 10 cm) were used. In each test, three impacts were caused by the three wheel axes of the lorry. The comparison between the results obtained in this study and the ones presented in (Clemente et al. 1998) is provided in Tab. 2.

Table 2. Comparison between the modal characteristics identified in this study and the results by Clemente et al. (1998): “B” denotes a bending mode, “T” indicates a torsional mode and “P” denotes a local mode of the central pylon.

Mode	This study		Clemente at al. (1998)	
	Frequency	Damping ratio	Frequency	Damping ratio
B1	0.92 Hz	3%	0.90 Hz	3.1%
B2	1.32 Hz	1.8%	1.30 Hz	1.6%
P1	-	-	2.50 Hz	-
T1	-	-	2.69 Hz	1.5%
B3	2.75 Hz	1.5%	-	-
T2	-	-	2.78 Hz	1.5%
B4	3.36 Hz	2.7%	-	-
B5	5.93 Hz	2.8%	-	-
B6	6.39 Hz	1.9%	-	-

Natural frequencies, damping ratios and mode shapes of the modes B1 and B2 calculated in this study are in good agreement with the results obtained by Clemente et al. (1998). A local mode with a natural frequency equal to 2.50 Hz was identified in (Clemente et al. 1998). This is a local bending mode involving the pylon, and its experimental identification was performed in Clemente et al. (1998) by exploiting two sensors installed along its height. The study by Clemente et al. (1998) was also able to identify two torsional modes with natural frequencies equal to 2.69 Hz and 2.78 Hz through the measurement points along the bridge sides. Because of the lack of sensors on the centered tower and along the bridge sides, the presented study was not able to identify the local mode of the pylon and the torsional modes of the deck. Differently from (Clemente et al. 1998), this study was able to identify a mode with a natural frequency equal to 2.75 Hz. It seems that Clemente et al. (1998) were not able to recognize experimentally such mode, but their finite element analysis confirm its existence. Specifically, the finite element model by Clemente et al. (1998) confirms that it is a bending mode, and the corresponding numerical natural frequency was found equal to 2.68 Hz. With respect to (Clemente et al. 1998), three further bending modes were identified experimentally in this study.

4 CONCLUSIONS

This study has illustrated the results of the experimental dynamic testing performed on the cable-stayed bridge that crosses the Garigliano river (Italy). Ambient vibrations-based re-testing and operational modal analysis of the deck have allowed the identification of six bending mode shapes. Natural frequencies and damping ratios of the first two modes do not show significant variations with respect to a previous work dating back to 1998. Impulse-induced dynamic response of the cables was examined in the frequency domain in order to estimate the tension by means of practical formulations. The estimated tensions are in good agreement with initial design provisions.

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