Advanced Materials Research Vols. 133-134 (2010) pp 709-714

doi:10.4028/www.scientific.net/AMR.133-134.709

Dynamic Assessment of the Iron Bridge at Paderno d'Adda (1889)

GENTILE Carmelo[1,a] and SAISI Antonella[1,b]

[1] Dept. of Structural Engineering, Politecnico di Milano, PMilan, Italy

[a]gentile@stru.polimi.it, [b]saisi@ stru.polimi.it

Abstract The paper presents the experimental modal analysis recently carried out on the historic iron bridge at Paderno d'Adda (1889). The dynamic tests were performed in operational conditions (i.e. under traffic and wind-induced excitation) between June and October 2009 and different output-only identification techniques were used to extract the modal parameters from ambient vibration data. The described tests represent the first experimental investigation carried out on the global characteristics of the bridge, since the load reception tests of 1889 and 1892.

Keywords: Dynamic testing, iron arch bridge, monitoring, operational modal analysis

Introduction

The iron arch bridge over the Adda River at Paderno (Fig. 1), built between 1887 and 1889 by the Società Nazionale delle Officine di Savigliano (SNOS), is one of the most important monuments of 19th century iron architecture and a symbol of Italian industrial archaeology (SNOS 1889, Nascè et al. 1984). The historic bridge is protected by the Italian Ministry of Cultural Heritage since 1980 and included in UNESCO heritage.

Notwithstanding the lack of maintenance and the poor state of preservation, the Paderno bridge is still used as a combined road and rail bridge, with the top deck carrying one lane of alternate roadway traffic and the bottom deck housing the tracks of a single-line railway.

In order to address a Structural Health Monitoring program based on the continuous dynamic monitoring of the bridge, ambient vibration tests were recently carried out. The first test, performed over 2 days, was aimed at investigating the vertical dynamic characteristics; subsequently, two other tests were carried out to check the variation over time of the previously identified resonant frequencies and to investigate the transverse dynamic behaviour (Gentile and Saisi 2010).

The paper focuses on the vertical dynamic characteristics of the bridge and their variation over time. In order to assess the accuracy of the identified modal parameters in view of the future monitoring, the most significant mode shapes and associated natural frequencies were determined in the frequency range 0−10 Hz by using two complementary output-only identification techniques with different theoretical bases: the *Frequency Domain Decomposition* (FDD, Brincker et al. 2001, developed in the frequency domain) and the data-driven *Stochastic Subspace Identification* (SSI, van Overschee and De Moor 1996, developed in the time domain).

Figure 1: *View of the iron bridge at Paderno d'Adda (1889)*

The Iron Bridge at Paderno d'Adda (1889)

The iron arch bridge over the Adda River (Fig. 1), designed in 1886 by J. Rothlisberger, consists of a single span parabolic web arch and an upper trussed box girder, supported by a series of piers. Three piers are erected from masonry basements while the others are supported by the parabolic arch; all the piers are battered in both directions according to the usual European practice of the late 19th century. The arch consists of two ribs, with a span of about 150.0 m and rise of 37.5 m; each of the two ribs is composed of double members 1.0 m apart and has a variable height, of between 8.0 m near the supports and 4.0 m at the crown. Since the two parabolic arch ribs are canted inward, the distance between the ribs is variable between 5.0 m at the crown and about 16.35 m at the shoes.

The deck is 266.0 m long and comprises 8 equal spans. The deck vertical trusses, 6.25 m high and 5.0 m apart, support two roadways: the upper one (originally with macadam pavement) for roadway and pedestrian traffic, and the lower for a single line of railroad.

All the iron members of the bridge have T or C shaped composite section and are formed by riveted flats and angles.

About 2,600 tons of iron were used in the bridge construction; according to the international classification of Philadelphia (1876), the bridge material can be classified as "wrought iron". Tests carried out on few samples of the bridge members between 1955 and 1972 (see e.g. Nascè et al. 1984) revealed rather poor metallurgical, chemical and mechanical characteristics. As it has to be expected for a wrought iron, the material is characterised by a stratified structure along the rolling plane and frequent non-metallic inclusions; the yield strength is generally larger than 240 MPa, with a tensile strength often less than 300 MPa and rather low (4-12%) elongation.

The bridge, opened to traffic on May 20th 1889, underwent major modifications and repairs during its history; in particular:

1. an important retrofit intervention was carried out between 1953 and 1956, aimed at repairing the structural damages suffered during the bomb attacks of the II World War and at re-painting the entire structure;
2. in 1972, the roadway deck (originally with Zorès beams) was entirely replaced by a steel orthotropic deck connected by rivets to the main structures;
3. the last intervention dates back to the early '90s and involved mainly the deck (replacement of damaged structural members, stiffening of the trussed box girder, sand-blasting and painting of the structural elements).

Further details of the bridge history and structural characteristics are reported in (Nascè et al. 1984).

Ambient Vibration Testing and Operational Modal Analysis

After 120 years of use and several structural modifications, and notwithstanding the lack of maintenance and the poor state of preservation of the iron members, the Paderno bridge is still in service as a combined road and rail bridge.

Since the last experimental investigation of the global behaviour of the structure dates back to the reception load tests performed in 1889 and in 1892, ambient vibration tests (AVT) were recently carried out, with the main objectives of identifying the dynamic characteristics (i.e. natural frequencies, mode shapes and damping ratios) of the bridge and their variation over time.

The first AVT, aimed at investigating the vertical modal behavior of the upper girder over the arch, was performed over 2 days (June 29-30, 2009); vibration measurements were performed according to the standard procedures for AVT, with the acceleration responses at the opposite sides of 13 cross-sections of the roadway deck (Fig. 2) being measured in two set-ups and considering 4 sensors (placed at the opposite sides of two reference cross sections) as reference transducers. A subsequent test was carried out on September 22, 2009 by using only two accelerometers (shown as red arrows in Fig. 2) to check the variation over time of the previously identified resonant frequencies and to investigate the importance of the transverse dynamic behaviour. The last experimental test was

carried out on October 26, 2009 and was aimed at checking again the variation over time of the natural frequencies and at identifying the transverse modes of vibration (Gentile and Saisi 2010). The AVTs were conducted using:

- a 24-channel data acquisition system, consisting of 6 NI 9234 4-channel dynamic signal acquisition modules (24-bit resolution, 102 dB dynamic range and anti-aliasing filters);
- uniaxial WR 731A piezoelectric accelerometers (Fig. 2); each WR 731A sensor, capable of measuring accelerations of up to ±0.50 g with a sensitivity of 10 V/g, was connected with a short cable (1 m) to a WR P31 power unit/amplifier.

The modal identification was performed by using the accelerations induced only by roadway traffic (Fig. 3) and at least two time windows of 2500 s were collected in each test. The sampling frequency was 200 Hz, which is much higher than that required for the tested bridge, as the natural frequencies of the dominant modes are below 10 Hz. Hence, a decimation was applied to the data before the use of the identification tools, reducing the sampling frequency from 200 Hz to 50 Hz.

The extraction of modal parameters from ambient vibration data was carried out by using two complementary output-only techniques: the FDD method (Brincker et al. 2000) in the frequency domain and the data driven SSI (van Overschee & De Moor 1996) in the time domain; these techniques are available in the commercial software ARTeMIS (SVS 2010).

The FDD technique is based on the Singular Value Decomposition (SVD) of the spectral matrix ***G(f)*** at each frequency: since the first singular value at each frequency represents the strength of the dominating vibration mode at that frequency, the first singular value can be suitably used as a modal indication function (yielding the resonant frequencies as local maxima), while the corresponding singular vector is an estimate of the mode shape. In the present application, the re-sampled time-series were processed in order to estimate ***G(f)*** with a frequency resolution of about 0.012 Hz.

In the application of SSI technique, stochastic state space models are identified of different order *N*, ranging from 2 to 120 in steps of 2, using the *Canonical Variate Analysis* (CVA) algorithm; inspection of the stabilization diagrams highlights that the observed dynamic behaviour is well represented using model orders between 70 and 100.

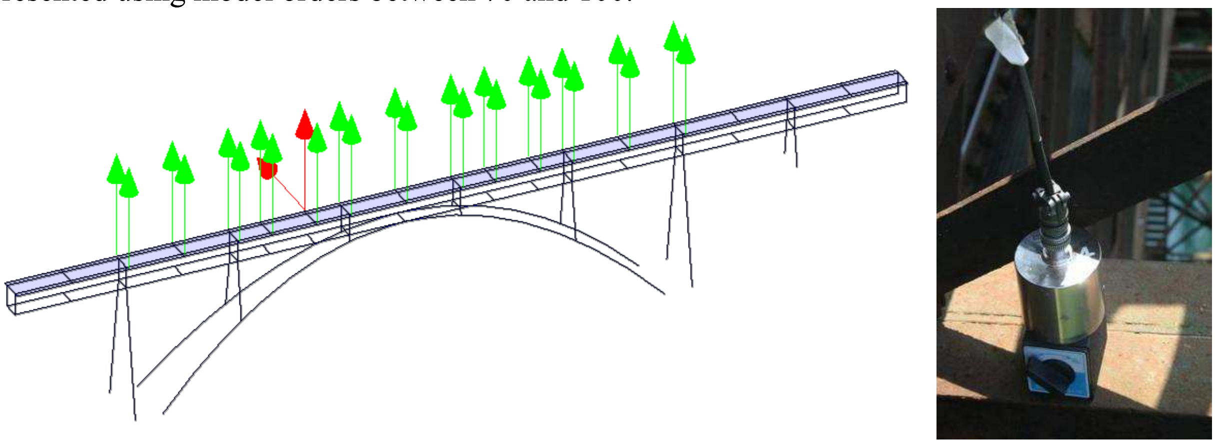

Figure 2: *Layout of the sensors on the bridge in the tests of June 2009 (green arrows) and September 2009 (red arrows) and image of one accelerometer*

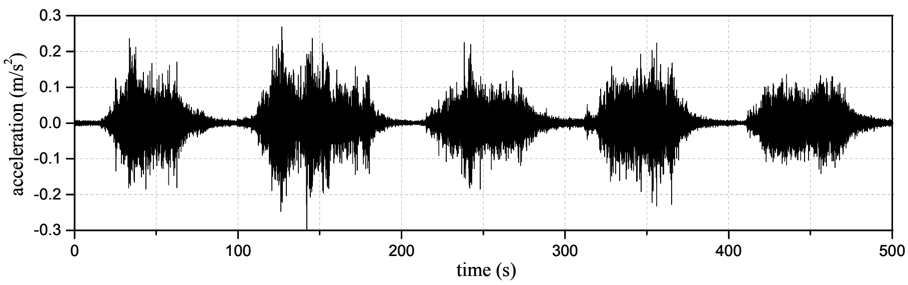

Figure 3: *Sample of measured vertical acceleration time series (roadway traffic)*

The two sets of mode shapes resulting from the application of FDD and SSI techniques were compared using the *Modal Assurance Criterion* (MAC, Allemang and Brown 1983).

The vertical modes of the Paderno bridge, identified in June 2009, are presented in Table 1. Eight modes were identified from the SVD plot (Fig. 4), using the peak-picking technique, and the corresponding mode shapes are shown in Fig. 5; all those modes, except the first one, were clearly identified by the SSI method as well.

Table 1 summarizes the results obtained by applying the FDD and the SSI identification methods through: (a) the natural frequencies (f_{FDD}) identified by the FDD method; (b) the average and the standard deviation values of the natural frequencies (f_{SSI}, σ_f) and modal damping ratios (ζ_{SSI}, σ_ζ) identified by the SSI technique. Furthermore, Table 1 compares the corresponding mode shapes provided by the two techniques through the MAC.

The natural frequencies estimated by both methods are almost coincident. A similar correspondence is found also for most mode shapes, except for the vertical bending mode B7; for this mode, the FDD technique seems to provide a better estimation of the mode shape.

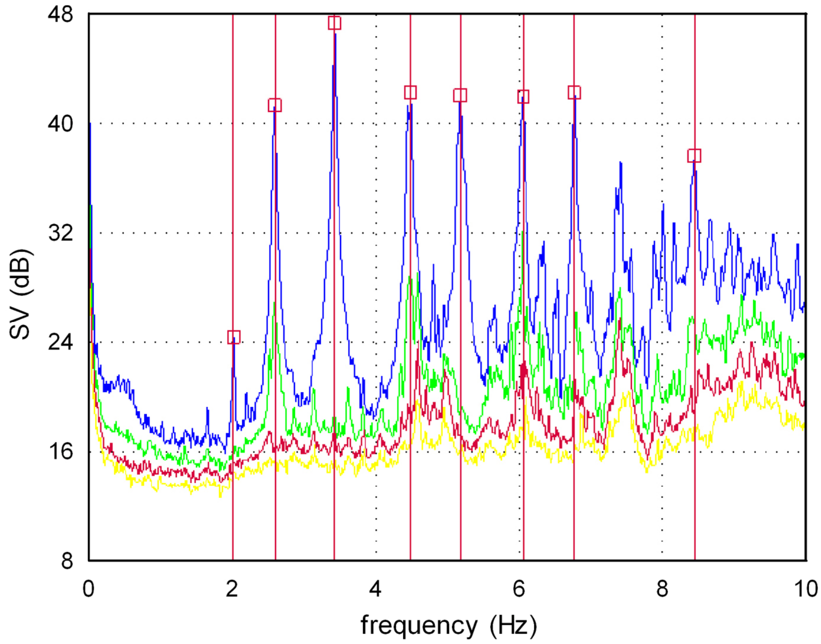

Figure 4: *Identification of natural frequencies (FDD): Singular value curves and peak picking*

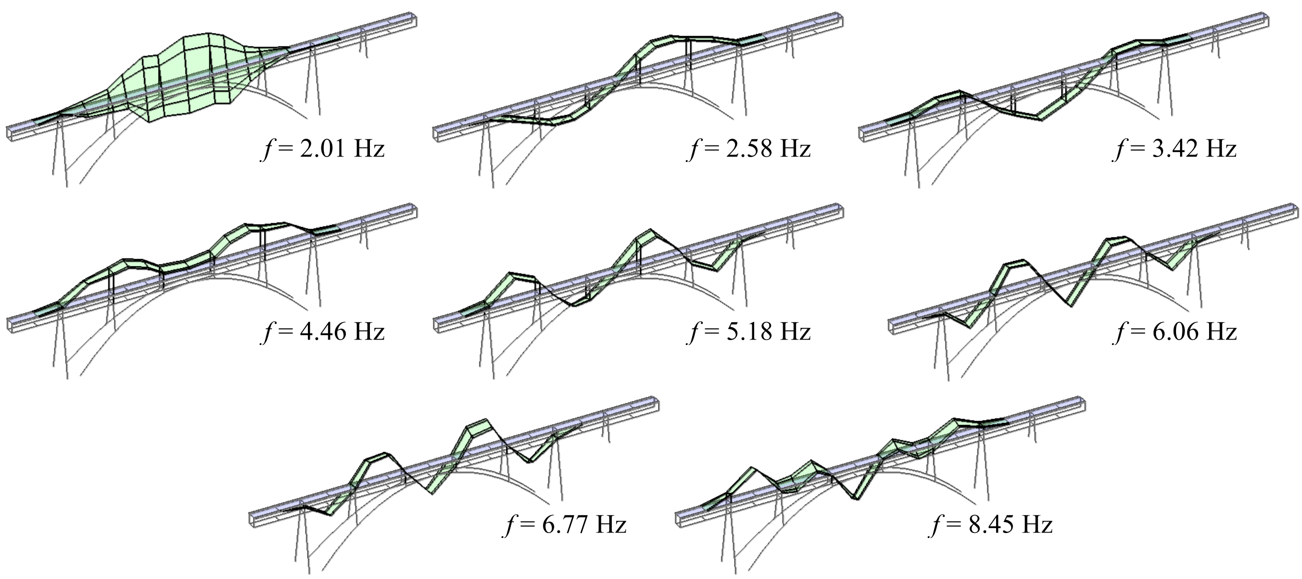

Figure 5: *Mode shapes identified by the FDD method*

Table 1: Summary of the modal parameters identified by FDD and SSI methods

Mode Type [(*)]	FDD	SSI				MAC
	f_{FDD} (Hz)	f_{SSI} (Hz)	σ_f (Hz)	ζ_{SSI} (%)	σ_ζ (%)	
T1	2.014	–	–	–	–	–
B1	2.582	2.579	0.0062	0.67	0.0560	0.9992
B2	3.424	3.418	0.0028	0.39	0.0380	0.9997
B3	4.462	4.464	0.0129	0.50	0.0983	0.9979
B4	5.176	5.175	0.0020	0.51	0.0301	0.9966
B5	6.055	6.046	0.0122	0.43	0.1986	0.9836
B6	6.769	6.764	0.0075	0.30	0.1313	0.9891
B7	8.453	8.420	0.0606	0.79	0.4019	0.8639

[(*)] V: Vertical Bending; T: Torsion

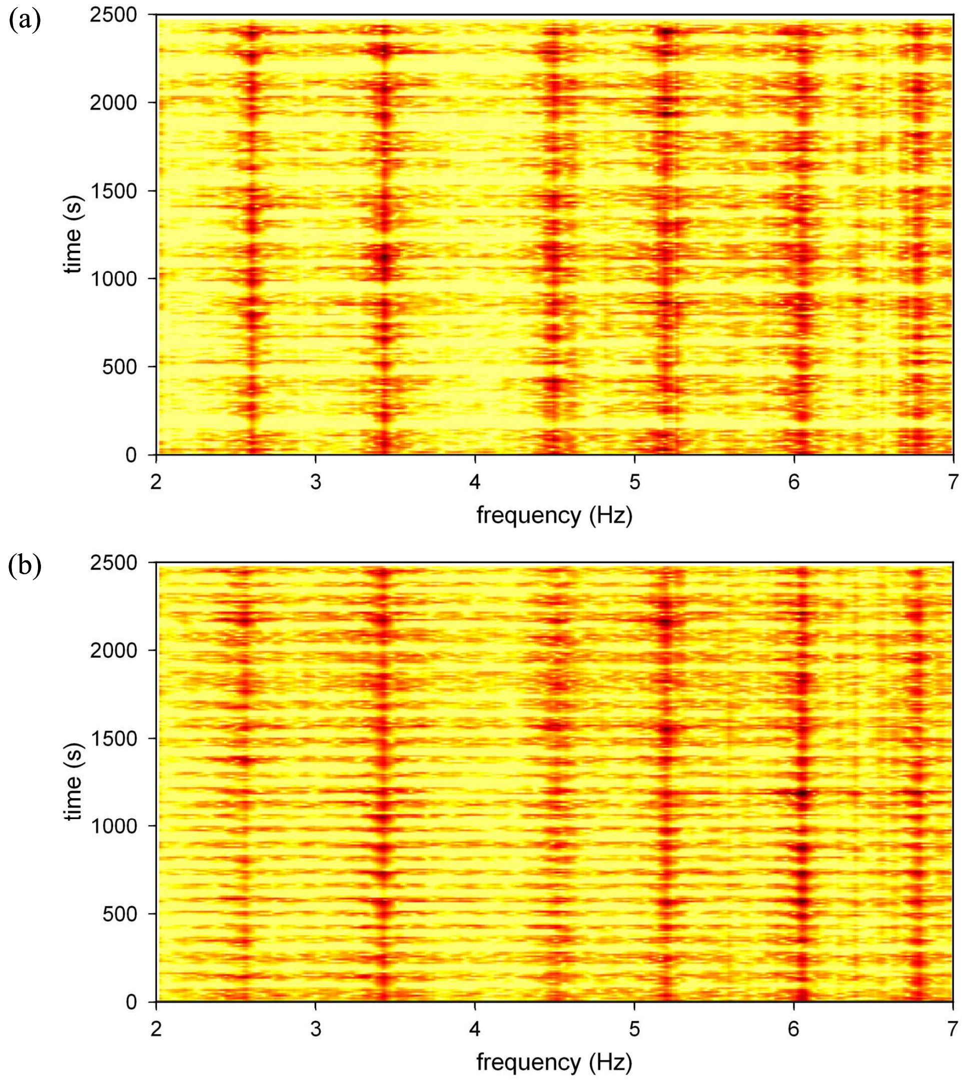

Figure 6: Colour map (spectrograms) with the variation of the signal frequency content over time: (a) 2009/06/29; (b) 2009/09/22

The identified modal behavior of the vertical modes suggests the following comments:

a) since the top deck is loaded by one lane of alternate roadway traffic, with the cars' passage being almost centered, the application of FDD and SSI methods provided the identification of seven vertical bending modes. One torsion mode was identified in the SVD plot (Figs. 4-5) only;

b) although the damping estimates are characterized by large standard deviations for the higher modes, the bridge generally exhibits low values of the damping ratios (0.3% < ζ < 0.8%, Table1);

c) the vertical bending modes are characterized by local violations of the symmetry condition expected with respect to the vertical plane containing the longitudinal axis of the bridge and the non-symmetric behavior is more clearly identified as the mode order increases. It should be noticed that similar shape uneven was not detected in the dynamic assessment of bridges similar to the Paderno bridge, such as the Luiz I bridge (1885) over the Douro river in Porto (Calçada et al. 2002); hence the mode shapes clearly highlight the different state of preservation of the structural elements on the downstream and upstream sides.

Furthermore, the inspection of the autospectra of the data acquired on June 29th and June 30th reveals slight variations of the natural frequencies. These variations are quite clear in the time-frequency plot of Fig. 6(a) and provided strong motivations for further experimental checks, carried out in September and October 2009. Fig. 6(b) refers to the September test and shows that, although the variation of the signal frequency content over time seems to be less significant than in Fig. 6(a) (probably as a consequence of lighter traffic flow), the resonant frequency of the first vertical bending mode decreased with respect to the previous test. Although no further variation was observed in subsequent test (October 2009), in which the transverse dynamic characteristics of the bridge (Gentile and Saisi 2010) were identified, the experimental investigation confirmed the opportunity of installing a permanent dynamic monitoring system on the bridge with Structural Health Monitoring purposes.

Conclusions

The paper focuses on the operational modal analysis of the Paderno bridge. The most significant mode shapes and associated natural frequencies of the vertical modes were determined in the frequency range 0–10 Hz by using two different output-only identification techniques, in order to assess the accuracy of the identified modal parameters.

The results clearly show time variance of the dynamic characteristics of the bridge, with resonant frequencies generally exhibiting slight variation depending on the excitation level; in addition, the vertical bending modes are characterized by local violations of the symmetry condition expected with respect to the vertical plane containing the longitudinal axis of the bridge. As a consequence, permanent dynamic monitoring has to be considered mandatory to accurately survey the possible evolution of the actual bridge condition.

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