Continuous dynamic monitoring of a centenary iron bridge for structural modification assessment

Carmelo GENTILE*, **Antonella SAISI**

Department of Architecture, Built environment and Construction engineering (ABC), Piazza Leonardo da Vinci, 32, Milan 20133, Italy

**Corresponding author. E-mail: carmelo.gentile@polimi.it*

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1 Introduction

In the last few years, the concern with the aging and structural degradation of bridges and civil infrastructures (see e.g., [1].) have contributed to significantly increase the interest on vibration-based Structural Health Monitoring (SHM) and the installation of dynamic monitoring systems, especially on bridges, has become more common [2]. The vibration-based approach to SHM has been favored also by the technological advances, allowing more economical installation and operation of permanent monitoring systems as well as efficient transmission and processing of the recorded data. Examples of well-known bridges equipped with continuous monitoring systems include Akashi Kaikyo bridge [3] in Japan, Commodore Barry bridge [4] in the United States, Infante D. Enrique bridge [5,6] in Portugal, Øresund Bridge [7] in Denmark, Tamar bridge [8] in South-west England and Tsing Ma bridge [9] in Hong Kong.

A multi-channel continuous dynamic monitoring system has been recently installed in a iron arch bridge, built between 1887 and 1889 by the Società Nazionale delle Officine di Savigliano (SNOS) to complete one of the first Italian railway lines [10,11]: the San Michele bridge. The historic bridge, protected by the Italian Ministry of Cultural Heritage since 1980, is a symbol of Italian industrial archeology heritage and is still used as a combined road and railway bridge. However, the aging of material and the difficulty of carrying out regular maintenance have caused a poor state of preservation of the structure, that appears nowadays significantly damaged by corrosion. In addition, although the weight and speed of vehicles are limited (180 kN/axis and 15 km/h for the trains, 35 kN and 20 km/h for the road vehicles), the bridge has not been saved from the progressive traffic increase, generally experienced by the infrastructures during last decades.

To assess the structural condition of the bridge, three different ambient vibration tests (AVTs) were performed in 2009 [12] by Politecnico di Milano. Those tests represented the first global experimental survey performed on the bridge since the load reception tests (1889 and 1892) and some peculiar aspects concerning the dynamic behavior of the bridge suggested the opportunity of installing a permanent dynamic monitoring system with Structural Health Monitoring (SHM) purposes. Hence, a joint research started between the Italian Railway Authority (RFI) — the main institutional owner of the historic infrastructure — and Politecnico di Milano, including the installation of a continuous dynamic monitoring system on the railway deck, in order to accurately survey the possible evolution of the actual bridge condition. The SHM program, based on the observation over time of the modal parameters automatically identified from the collected data, started in late November 2011.

The first part of the paper, after a description of the bridge, summarizes the main conclusions inferred from the AVTs carried out in 2009 [12] and providing the motivations of a SHM program. Subsequently, the main results of the AVTs performed between March 2010 [13] and June 2011 [14,15], with the aim of defining the position of the sensors to be permanently installed in the bridge and verifying the design of the monitoring system, are presented. Subsequently, full details are given on the second phase of the research, including the following main tasks:

1) Installation of the continuous dynamic monitoring system with remote control and data transmission via Internet.

2) Development of efficient toolkits to process the raw data regularly received and to automatically extract the modal parameters [16,17].

3) Tracking of the time variation of natural frequency estimates and analysis of the influence of temperature and (road) traffic intensity on those estimates.

4) Identification of structural performance anomalies.

2 The iron arch bridge at Paderno d'Adda

The San Michele bridge (Figs. 12), better known as Paderno bridge, was designed in 1886 by the head of SNOS technical division, the Swiss engineer Julius Röthlisberger (1851–1911), and its construction was completed on March 1889.

The bridge (Figs. 1–2) spans the gorge cut by the river Adda between Calusco and Paderno d'Adda enabling the single-track railway line between Ponte S. Pietro and Seregno (which was one of the first Italian railway lines) to cross the deep divide at a height of about 80.0 m with respect to the water level.

The bridge consists of a single span parabolic arch, an upper trussed box girder and a series of piers (Figs. 1–2). Three piers are erected from masonry basements while the others are supported by the parabolic arch (Fig. 2); all the piers are battered in both directions according to the usual European practice of the late XIX century.

The total length of the bridge is 266.0 m and the trussed arch (consisting of two ribs composed of double members 1.0 m apart, Fig. 3(a)) has the following dimensions [10,11]:

- Span: 150.0 m;
- Rise: 37.5 m;
- Depth of the arch cross-section at crown: 4.00 m;

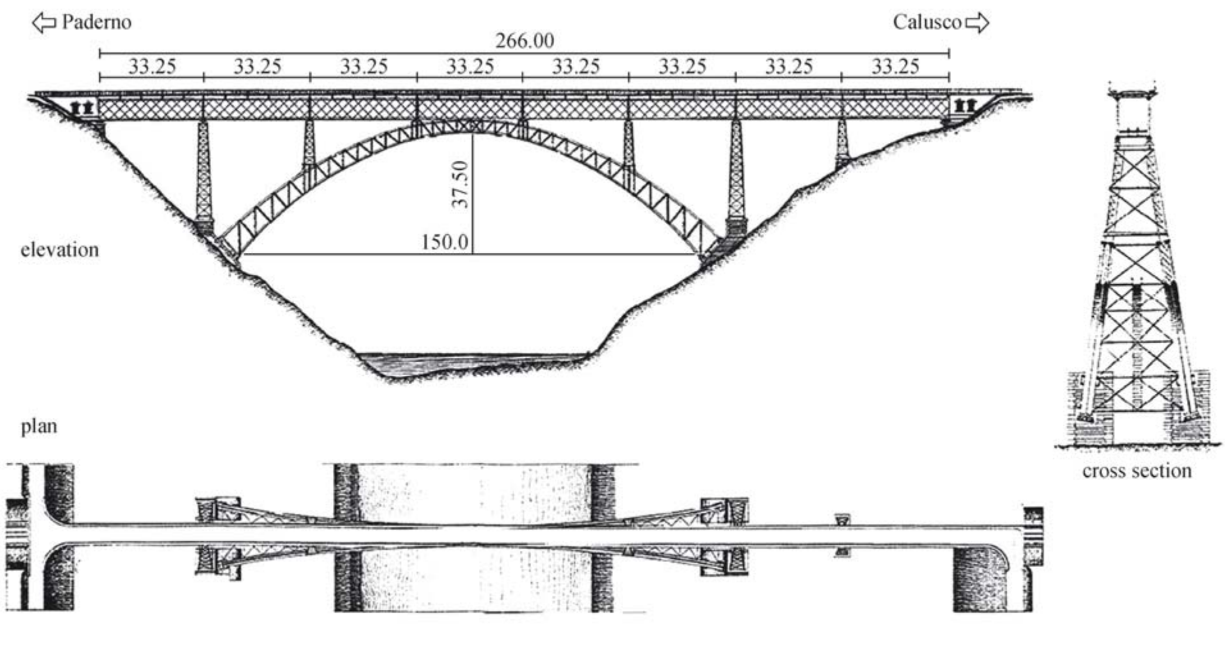
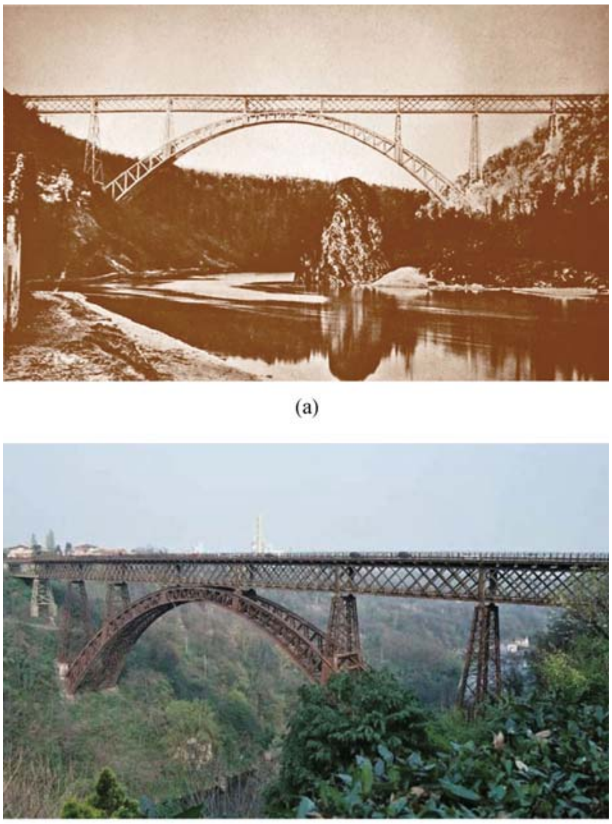


Fig. 1 Elevation, plan and cross-section of the San Michele bridge (unit: m) [10]



(a)

(b)

Fig. 2 (a) Historic and (b) present view of the San Michele bridge

- Depth of the arch cross-section at springing: 8.00 m;
- Width of the arch cross-section at crown: 5.00 m;
- Width of the arch cross-section at springing: 16.35 m.

The upper girder is 266.0 m long and is supported by 9 bearings, equally spaced every 33.25 m, so that the girder behaves as a beam bridge defined between abutments and with equally spaced elastic supports. The girder vertical trusses (6.25 m high and 5.0 m apart) consist of T-shaped chords connected by multiple lattices and support two decks: the upper one for roadway and pedestrian traffic, and the lower one for a single line of railroad (Fig. 3(b)).

The piers, shaped like truncated pyramids, are composed by 2 truss-box inclined posts, connected by horizontal and bracing elements (Fig. 3(c)); each inclined post consists of 4 angular elements connected by transversal and diagonal members in the 4 main planes.

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 [11] revealed rather poor metallurgical, chemical and mechanical characteristics. The material is characterized 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 20/05/1889, underwent major modifications and repairs during its history:

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 1990s and involved mainly the deck (replacement of damaged structural members, stiffening of the trussed box girder, sand-blasting and painting of the structural elements).

Due to the historic importance of the bridge, almost all the original drawings are available in the State Archives, whereas the repair interventions are well documented in the RFI archives. In addition, a comprehensive and valuable study of the bridge history and structural characteristics is reported in [11].

As previously stated, the state of preservation of the bridge is rather poor; Fig. 4 exemplifies typical damages induced by the corrosion, observed on a huge number of structural members on both downstream and upstream sides. It should be noticed that the large spacing of the rivets, not adequate to the thickness of the iron plates, easily allows the penetration of moisture between the contact planes so that the subsequent oxide expansion induces deformations of the iron plates and sections at the connections of the composite struts.

3 Preliminary tests and modal parameters of the bridge

Three AVTs were carried out on the roadway deck of the Paderno bridge between June 2009 and October 2009, in order to evaluate the dynamic characteristics of the historic infrastructure [12]. The first test (June 2009) was aimed at investigating the vertical dynamic characteristics, whereas the subsequent two tests were performed to check the possible variation over time of the previously identified resonant frequencies (September 2009) and to investigate the transverse dynamic characteristics (October 2009), respectively. The experimental survey clearly highlighted [12] that:

1) A large number of normal modes were determined in the frequency range 0–10 Hz. More specifically, 4 vertical bending modes and 17 transversal bending modes were identified in the range 0–6 Hz, whereas 3 vertical bending were identified in the range 6–9 Hz.

2) The bridge generally exhibited low values of the damping ratios ($\zeta_i < 1\%$) in both vertical and transverse dynamic response.

3) Under road traffic loads, the natural frequencies of vertical bending modes exhibited slight variations, possibly depending on the excitation/response level.

(a) (b) (c)

Fig. 3 Structural details of the bridge

(a) (b) (c)

Fig. 4 Examples of the damage inflicted by the corrosion to the bridge members

4) The vertical bending modes exhibit non-symmetric modal deflections on the upstream and downstream sides of the deck and the non-symmetric behavior is more clearly identified as the mode order increases. The average difference between the modal deflections on the upstream and downstream sides ranges between 4.25% and 22.26%. Since the available drawings and documents, concerning both the original design and the refurbishments, do not show any significant lack of symmetry between the two sides of the bridge, the observed non-symmetric mode shapes, revealing a different stiffness of the downstream and upstream sides, are conceivably related with the different state of preservation of the structural elements on the two sides. The observation of the typical corrosion damages (Fig. 4), unevenly distributed on structural elements of the deck and the arch, strengthens and corroborates this conclusion. In addition, the mode shapes uneven was not detected in the dynamic assessment of

similar bridges, where regular maintenance has been carried out, such as the Luiz I bridge (1885) over the Douro river in Porto [18].

The results of the above tests provided the evidence that AVT and operational modal analysis (OMA) are effective tools for assessing the structural condition of the historic bridge and, at the same time, suggested the continuous dynamic monitoring as a suitable strategy for SHM. Hence, the RFI decided to install a dynamic monitoring system on the railway deck of the bridge.

To properly design a monitoring system, prior knowledge of the modal parameters of the structure (natural frequencies and mode shapes) is generally required [5]. Although the tests performed in 2009 provided valuable information on natural frequencies and mode shapes of the roadway deck, no information on modal deflections of the railway deck were available. On the other hand, the railway deck turned out to be more suitable for installing the monitoring system since mounting, wiring and even inspecting the sensors and all other devices is relatively easy on the railway deck with the support of RFI technical staff. Hence, further dynamic tests [13–15] were performed between March 2010 and June 2011.

The test performed on March 2010 involved a large number of instrumented points on both roadway and railway deck [13] and was mainly aimed at: (a) better understanding the dynamic characteristics of the bridge and (b) checking the number of points to be permanently instrumented on the railway deck. During the test, again slight variation of the natural frequencies was observed and variation of few mode shapes between the two tests of June 2009 and March 2010 was also detected. It is further noticed that the sensor set-up mounted on the railway deck (Fig. 5) allowed the identification of almost all the normal modes previously detected.

In the subsequent tests, performed on 25–26 November 2010 and 13–14 June 2011, the response was continuously recorded at the points scheduled for permanent monitoring (Fig. 5) for 15 and 24 h, respectively. During these tests the temperature was not measured since the main objectives were: a) evaluating some aspects useful in the future monitoring and especially related to the automated identification of the time-histories corresponding to train passages and the length of "train-free" time window to be extracted from each hour for the subsequent OMA; b) defining a baseline list of modes for monitoring (i.e., those modes that generally exhibited a significant occurrence over several hours of continuous recording); c) testing the robustness and reliability of the tools developed for data acquisition, storage, signal processing and automated OMA [16,17].

The two tests of November 2010 and June 2011 demonstrated that each 1-h data set generally provides a "train-free" time window of length T larger than 2400 s. On one hand, this observation implies that each 1-h data set collected during the day will provide enough data for a good accuracy of the modal identification (the length of the time window acquired should be 1000 to 2000 times the period of the fundamental mode); on the other hand, applying the modal identification tools to time windows of constant length has surely to be preferred in permanent dynamic monitoring. Hence, it was decided to extract one time window of 2400 s from each 1-h data set and to use this time window for identifying the modal parameters as well as for describing the average intensity of road traffic.

The automated OMA was initially performed in the frequency domain by using the well-known Frequency Domain Decomposition (FDD) method [19]. The reliability and accuracy of the developed automated OMA tools [16,17] was verified through extensive comparison with

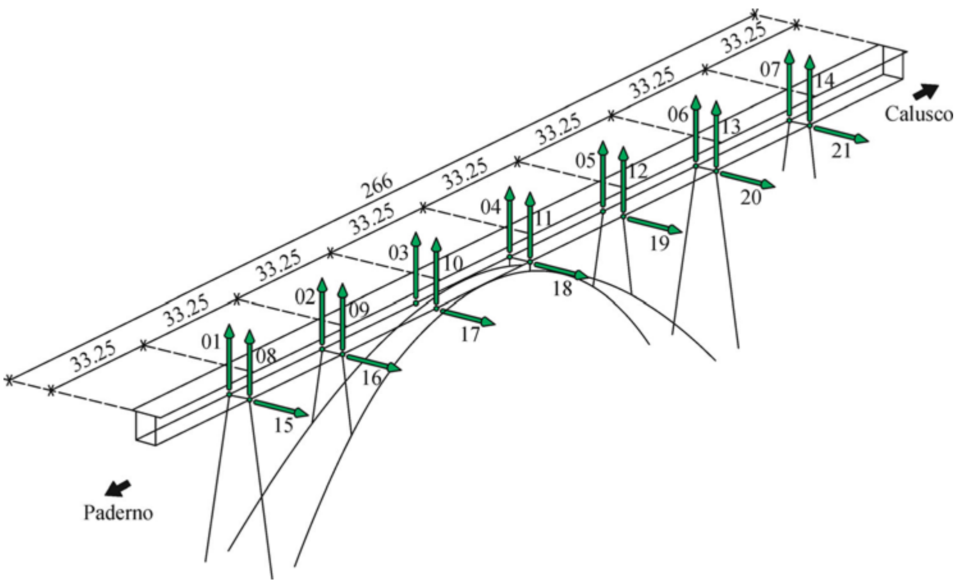


Fig. 5 Measurement points on the railway deck adopted in the tests of November 2010, June 2011 and for the monitoring system (unit: m)

the results obtained using the OMA methods available in the commercial software ARTeMIS [20]. For example, typical results, in terms of identified natural frequencies, obtained by applying the developed automatic FDD to the data collected on 14/06/2011, 07:00–08:00 are shown in Fig. 6. The corresponding transversal and vertical modes of the bridge are shown in Figs. 7 and 8.

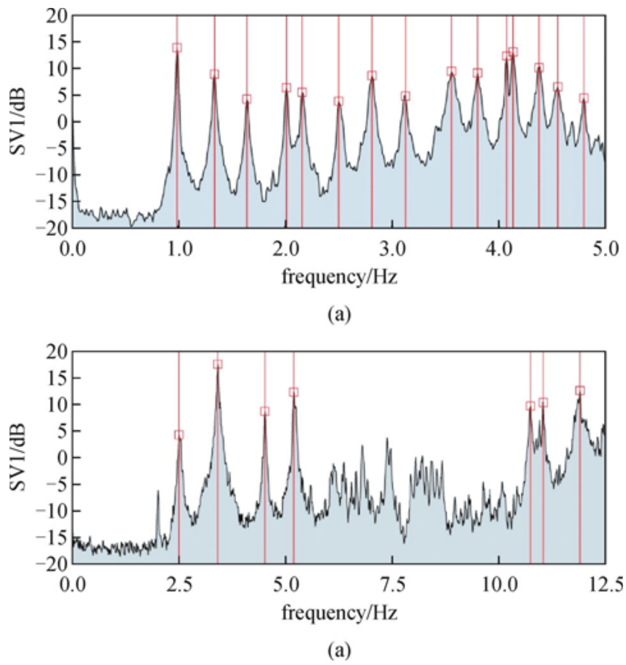


Fig. 6 First Singular Value line (SV1) and identification of natural frequencies (14/06/2011, 07:00–08:00). (a) Transversal bending modes; (b) vertical bending modes

All the modes in Figs. 7 and 8 exhibited a high occurrence during the tests of continuous acquisition and were included in the baseline list of the modes to be monitored. It has to be observed that: (1) the vertical modes VB1-VB4 (Fig. 8(a)–(d)) and the transversal bending modes TB1-TB15 (Fig. 7(a)–(o)) were identified in all previous tests, as well; (2) three vertical modes, with frequency ranging between 6 and 9 Hz, that were identified in the previous tests [12] performed on the roadway deck, are not clearly detected (Fig. 6(b)) when only the railway deck is instrumented; (3) conversely, the vertical bending modes VB5-VB7 (Fig. 8(e)–(g)) were not detected in the previous tests of June 2009 and March 2010.

4 Dynamic monitoring system and signal processing tools

The continuous dynamic monitoring system installed on the San Michele bridge consists of 21 MEMS accelerometers, 2 thermocouples, 7 Ethernet data acquisition (DAQ) modules, 2 Ethernet switch devices and 1 industrial PC. The global arrangement of sensors and hardware components along the bridge is schematically illustrated in Fig. 9. As previously pointed out, the sensor layout was verified in various AVTs and turned out to be adequate for identifying all the lateral modes in the frequency range 0–5 Hz (Fig. 7) and the lower modes of vertical bending (Fig. 8).

A distributed architecture is adopted and 7 instrumented cross-sections, corresponding to the bearings of the truss-box girder between the abutments, are equipped with (Fig. 9): (a) One NI Ethernet DAQ unit, which is incorporated in a steel box installed between the rails and (b) three MEMS accelerometers, measuring the vertical accelerations on the downstream/upstream sides and the lateral acceleration. Temperature sensors are placed at the second and the fifth instrumented cross sections (Fig. 9) and measure the air temperature nearby the structure on the upstream and the downstream sides, respectively.

Each DAQ unit serves three accelerometers (or three accelerometers and one temperature sensor), so that wiring from the sensors to the analog-to-digital converter is minimized. Two switch devices collect the Ethernet cables from each group of channels and the digitized data are transmitted to an industrial PC on site.

A new binary file, containing 21 acceleration time series (sampled at 200 Hz) and the temperature data, is created every hour, stored on the local PC and transmitted via Internet to Politecnico di Milano for the being processed.

The continuous dynamic monitoring system is now active since 28 November 2011. Each data file received from the monitoring system was processed by a software developed in LabVIEW, including the (online or off-line) execution of the following tasks:

1) Creation of a database with the original data in compact format for later developments.

2) Data pre-processing, i.e., de-trending, automatic recognition and extraction of the time series associated to the railway traffic. Since the response induced by the railway traffic does not comply with some basic assumptions required for the OMA application, such as white noise and stationarity, it is mandatory that the signals corresponding to the train passages are automatically identified, extracted and collected separately. Hence, each 1-h data file is divided in 1 main data set, having duration of 2400 s and not containing train passages, and a certain number of other data sets, each collecting one train passage.

Figure 10(a) shows a sample of vertical acceleration time series with four train passages. The automatic identification of train passages is performed by computing an “average RMS function”. For each channel, the root mean square (RMS) value of the acceleration time series is evaluated over the time interval of 1 s; subsequently the RMS values are averaged over all channels to obtain the plot in Fig. 10(b), where the peaks and the valleys correspond to the train passages and to the temporary stops

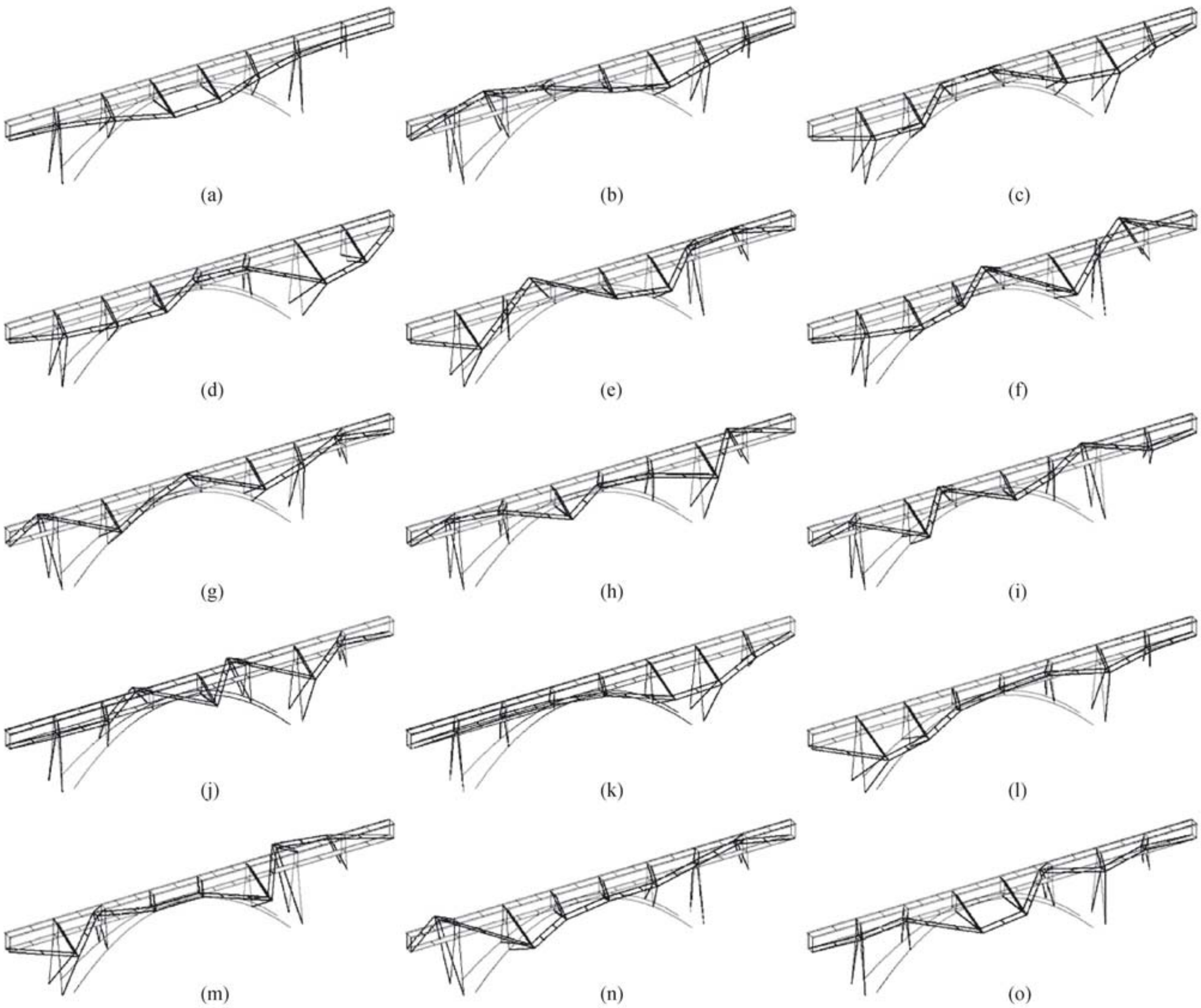


Fig. 7 Transversal modes generally identified from ambient vibration data (14/06/2011, 07:00–08:00). (a) TB1: f_{FDD} = 0.986 Hz; (b) TB2: f_{FDD} = 1.338 Hz; (c) TB3: f_{FDD} = 1.641 Hz; (d) TB4: f_{FDD} = 2.012 Hz; (e) TB5: f_{FDD} = 2.158 Hz; (f) TB6: f_{FDD} = 2.500 Hz; (g) TB7: f_{FDD} = 2.813 Hz; (h) TB8: f_{FDD} = 3.125 Hz; (i) TB9: f_{FDD} = 3.555 Hz; (j) TB10: f_{FDD} = 3.799 Hz; (k) TB11: f_{FDD} = 4.072 Hz; (l) TB12: f_{FDD} = 4.131 Hz; (m) TB13: f_{FDD} = 4.375 Hz; (n) TB14: f_{FDD} = 4.551 Hz; (o) TB15: f_{FDD} = 4.795 Hz

of alternate road traffic, respectively. The extraction procedure consists in "cutting" the time series corresponding to the train passage between two subsequent local minima, that are detected before and after the peak.

Due to the combined rail and road use of the bridge, the length of the time windows, extracted during the day and associated to ambient excitation and road traffic, is generally not constant. As already stated in Section 3, one time windows of 2400 s is extracted from each 1-h record and used for modal identification.

3) Statistical analysis of data. This phase includes also the evaluation of hourly-averaged temperatures, the computation of the RMS value of accelerations associated to the time window of 2400 s not containing train passages, and the computation of the RMS value of accelerations associated to each train passage;

4) Low-pass filtering and decimation of the each data set of 2400 s containing the response to road traffic only and creation of a second database in ascii format (in order to have the possibility of performing spectral analysis and OMA using software different from LabVIEW).

5) Automated modal identification using the FDD method.

6) Visualizing the most significant results in a quick way by trends and graphs.

It is further noticed that low-pass filtering and decimation applied to the time series of 2400 s before the use of the identification tools, reduces the sampling frequency from 200 to 40 Hz; after decimation, the number of samples in each record is of 96000, with a sampling interval of 0.025 s. Subsequently, the spectral matrix (i.e., the matrix where the diagonal terms are the auto-spectral densities while the other terms are the cross-spectral densities, see e.g., Ref. [21].) of vertical and transverse

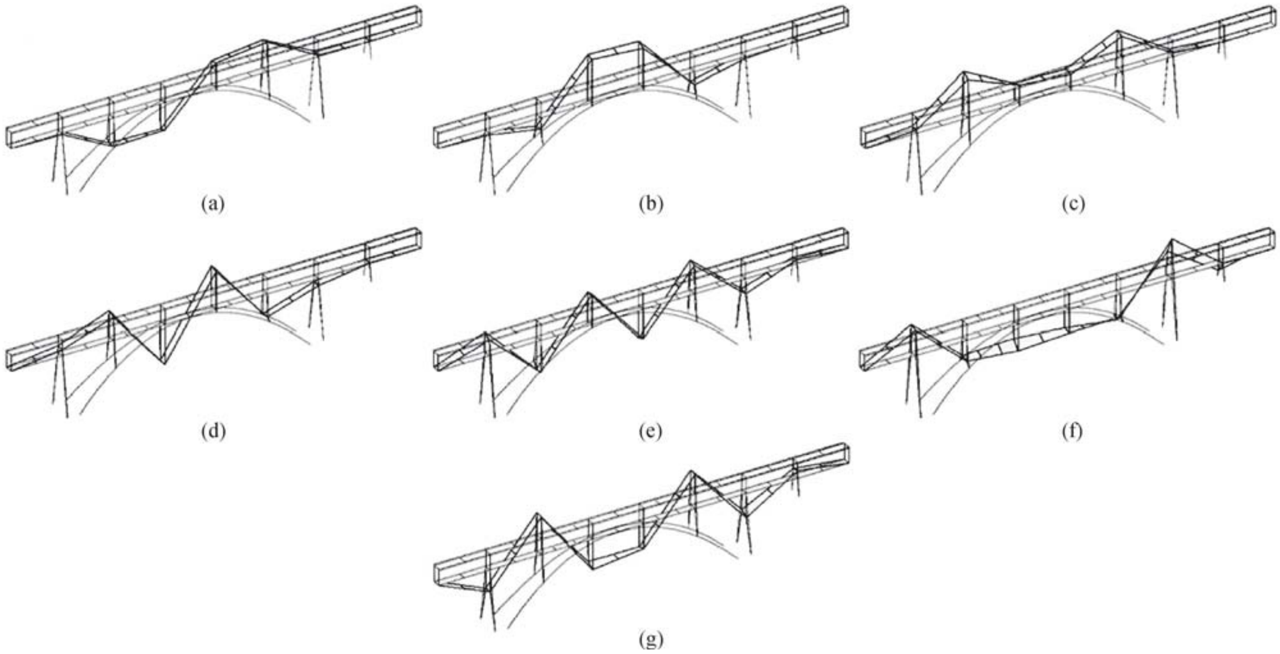


Fig. 8 Vertical modes generally identified from ambient vibration data (14/06/2011, 07:00–08:00). (a) VB1: $f_{\mathrm{FDD}} = 2.500$ Hz; (b) VB2: $f_{\mathrm{FDD}} = 3.408$ Hz; (c) VB3: $f_{\mathrm{FDD}} = 4.512$ Hz; (d) VB4: $f_{\mathrm{FDD}} = 5.186$Hz; (e) VB5: $f_{\mathrm{FDD}} = 10.73$ Hz; (f) VB6: $f_{\mathrm{FDD}} = 11.01$ Hz; (g) VB7: $f_{\mathrm{FDD}} = 11.87$ Hz

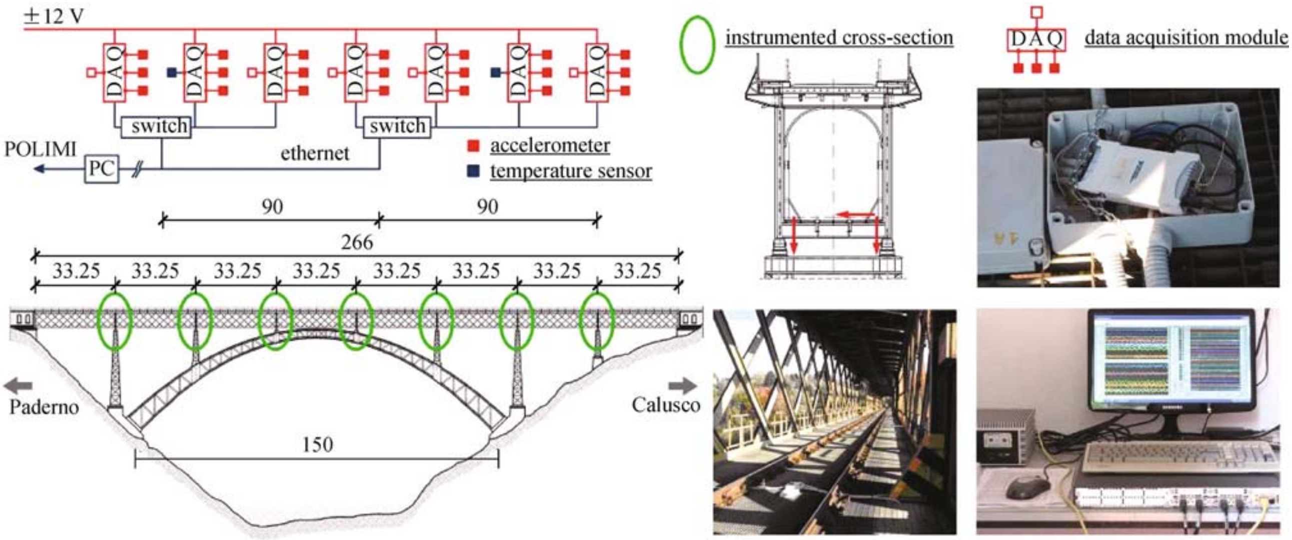


Fig. 9 Schematic of the continuous dynamic monitoring system installed in the San Michele bridge and components of the monitoring system (unit: mm)

accelerations were evaluated using the Welch procedure [22] with a frequency resolution of 0.009766 Hz.

5 Monitoring results

5.1 Evaluation of averaged vibration amplitude (roadway traffic)

To investigate the effect of roadway traffic on the modal characteristics of the bridge, the averaged RMS value of the acceleration time series is used. More specifically, for each data set of 2400 s, the RMS value of acceleration is computed for each channel (Figs. 11 and 12) and averaged over the vertical and the horizontal channels so that the intensity of road traffic associated to each set of identified modal parameters is approximately described by the two averaged RMS values associated to vertical and horizontal accelerations.

Figure 11 shows the variation over 4 weeks of the RMS

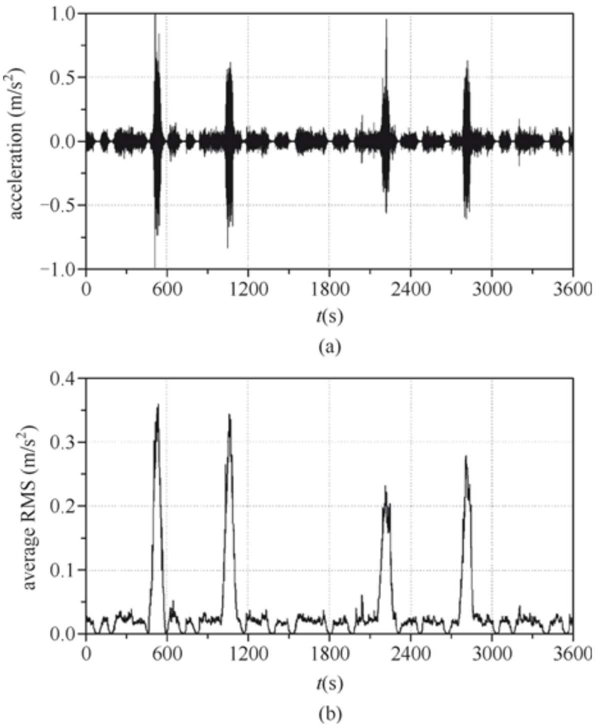


Fig. 10 (a) Sample of recorded signal ($T = 3600$ s) with four train passages; (b) average RMS function

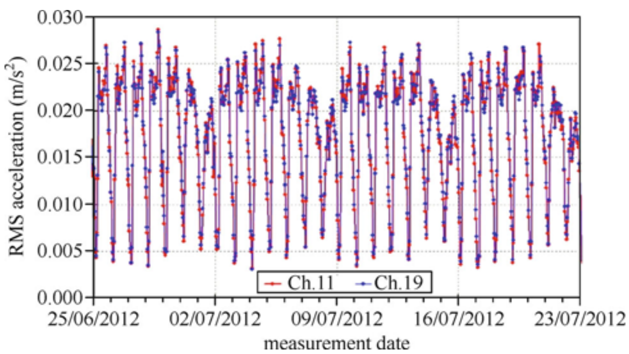


Fig. 11 Typical variation of the RMS value of vertical (red line) and transversal (blue line) acceleration

of the accelerations recorded in the vertical (red line) and lateral (blue line) channel, that are characterized by the largest RMS values, averaged over the entire day. It should be noticed that two rush hours of road traffic typically occur around 07:00 and 17:00; furthermore, the level of road traffic is quite high between the rush hours, whereas the traffic intensity significantly decreases at late evening and during the night. It is further noticed (Fig. 11) that, as it has to be expected, the road traffic decreases during the weekends (or in holydays) and the first rush hour moves around 12:00–13:00.

The variation of the RMS value of vertical accelerations recorded at channels 1–7 (Fig. 5) in the period between 28/11/2011 and 12/05/2013 is illustrated in Fig. 12, highlighting a similar evolution in each point of the bridge deck; furthermore Fig. 12 shows that the RMS of vertical accelerations (i.e., the road traffic intensity) tends to significantly increase in Spring and Summer and to decrease after the end of August.

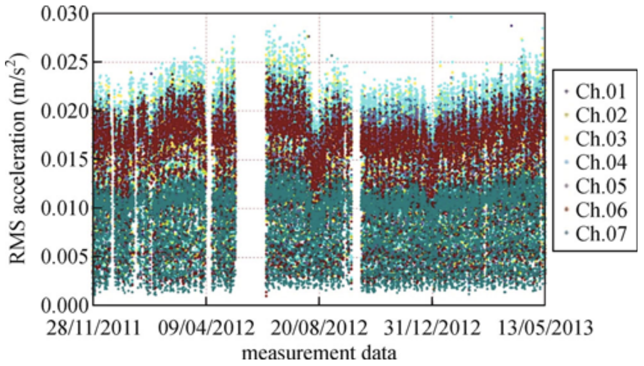


Fig. 12 RMS values of the vertical acceleration recorded by sensors 1-7 (Fig. 5) in the period from 28/11/2011 to 12/05/2013

5.2 Measurement of temperature

Figure 13 displays the temperature records from the two sensors installed in the upstream (blue line) and downstream side of the bridge between 28/11/2011 and 12/05/2013. It can be observed that the two temperature series are very similar, except in winter afternoons (i.e., between 13:00 and 16:00) when the temperature on downstream side of the bridge is few °C higher.

It is worth mentioning that the average temperature between the two sides was used to investigate the effects of temperature on the dynamic characteristics of the bridge.

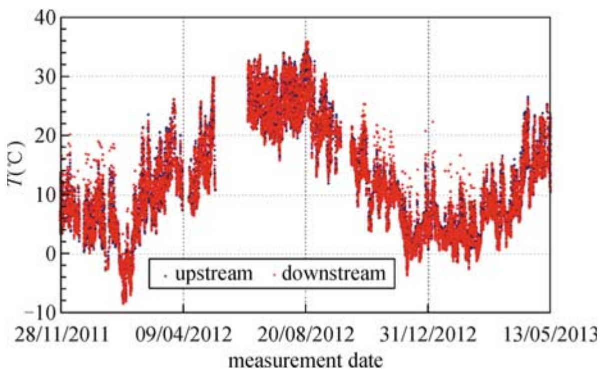


Fig. 13 Measured temperature in the period from 28/11/2011 to 12/05/2013

5.3 Tracking of natural frequencies and environmental/operational effects

The adopted sensor layout yields to the automated identification, with high occurrence, of the normal modes shown in Figs. 7 and 8: 15 transversal modes in the frequency range 0–5 Hz (Fig. 7) and of 7 vertical bending modes (Fig. 8).

Automated identification of the modal frequencies from the data sets collected during the period from 28/11/2011 to 12/05/2013 provided the frequency tracking shown in Figs. 14(a) (transversal modes) and 14(b) (vertical modes). The inspection of Figs. 14(a)–(b) suggests the following comments:

1) The algorithm for automatic modal identification exhibits good and robust performance, as all the expected modes (including the closely spaced modes) are clearly identified with high occurrence.

2) The natural frequencies exhibit relatively small but clear variations.

3) During the period under analysis, some interruptions of the monitoring occurred due to maintenance operations in the electric cabin providing power to the monitoring system and owned by the municipality of Paderno. The longest interruption occurred between 14/05/2012 and 17/06/2012 and conceivably allowed to solve the related issues.

4) Figures 14(a) and 14(b) (as well as Figs. 17(a)–(d)) show obvious abnormal increase of all modal frequencies in winter time. This phenomenon was almost constantly observed in the first two weeks of February 2012 and, more frequently but involving shorter time periods, between December 2012 and February 2013. In the corresponding periods, no clear change of the structural condition was observed but significant changes of the environmental conditions occurred. More specifically, heavy snowfalls occurred in the first half of February 2012 and several structural elements of the bridge's trussed girder and arches were covered with ice, since the temperature ranged between 0°C and – 8°C for more than two weeks (Fig. 13); between December 2012 and February 2013, snow-falls occurred almost every weeks, with the temperature being below 0°C for several hours (in correspondence with each snowfall). Hence, the temporary and anomalous increase of modal frequencies has to be associated to low temperatures, snowfalls and the observed presence of ice on the bridge members. On one hand, frequency peaks are due to the locking of the iron bearing of the girder, that is induced by low temperatures (and the possible ice formation); on the other hand, the presence of ice on the

Table 1 Statistics of the natural frequencies identified from 28/11/2011 to 12/05/2013 (excluding the periods with low temperature and ice/snow on the bridge)

mode	f_{ave} (Hz)	σ_{f} (Hz)	f_{min} (Hz)	f_{max} (Hz)
TB1	0.991	0.005	0.967	0.996
TB2	1.346	0.005	1.328	1.357
TB3	1.650	0.005	1.631	1.670
TB4	2.015	0.005	2.002	2.031
TB5	2.174	0.011	2.139	2.207
TB6	2.509	0.011	2.471	2.559
VB1	2.542	0.024	2.441	2.607
TB7	2.825	0.019	2.764	2.891
TB8	3.118	0.015	3.066	3.164
VB2	3.418	0.009	3.379	3.447
TB9	3.572	0.027	3.477	3.662
TB10	3.801	0.022	3.701	3.867
TB11	4.083	0.007	4.043	4.111
TB12	4.141	0.009	4.111	4.189
TB13	4.383	0.016	4.336	4.424
VB3	4.513	0.019	4.375	4.551
TB14	4.562	0.018	4.482	4.619
TB15	4.806	0.012	4.766	4.854
VB4	5.219	0.017	5.146	5.264
VB5	10.745	0.020	10.664	10.801
VB6	11.040	0.022	10.947	11.113
VB7	11.894	0.033	11.719	12.012

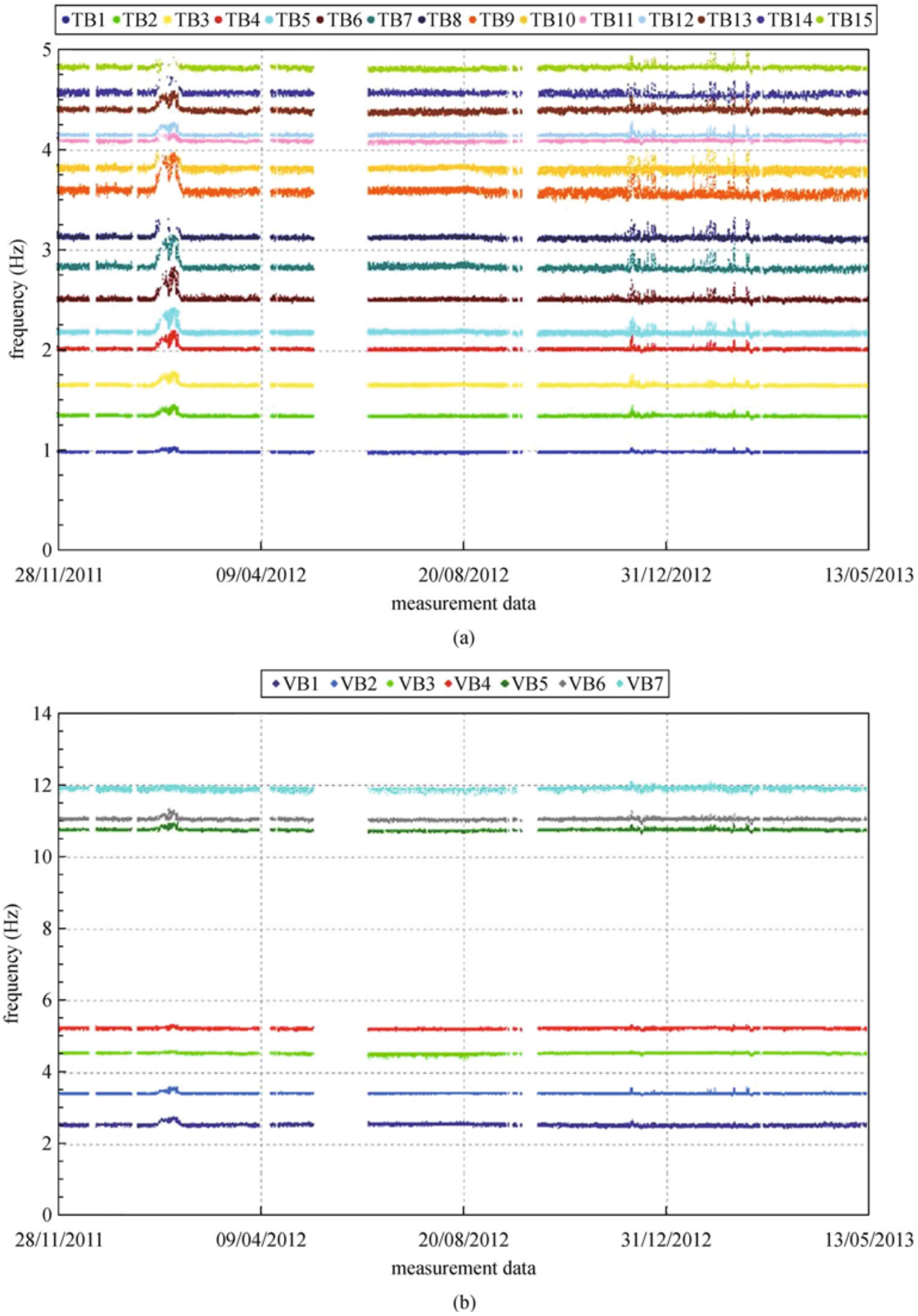


Fig. 14 Time evolution of the natural frequencies automatically identified with the FDD method, from 28/11/2011 to 12/05/2013. (a) Transversal modes; (b) vertical modes

structure conceivably involved an augmented cross-section of many structural elements and, hence, the increase of the local stiffness of iron members. Since the stiffening due to low temperature and ice has to be considered as temporary, the correlation analysis between modal frequencies and environmental/operational conditions was performed excluding the "ice periods" in order to consider the evolution referred only to a "normal" behavior of the structure.

Statistics of the natural frequencies for all modes that were identified between 28/11/2011 and 12/05/2013 (excluding the periods with low temperature and ice/

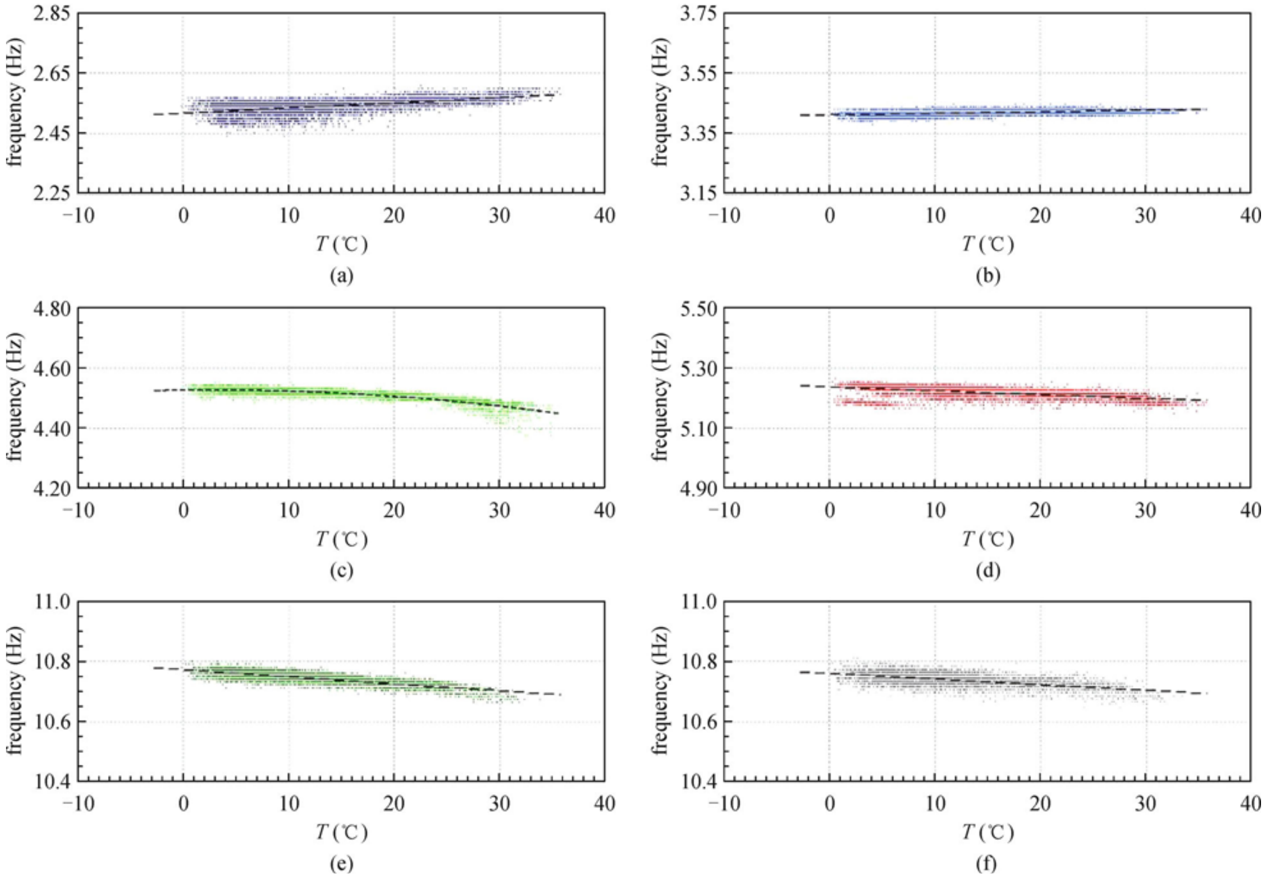


Fig. 15 Correlation between temperature and modal frequencies identified from 28/11/2011 to 12/05/2013 (excluding the periods with low temperature and ice/snow on the bridge). (a) Mode VB1; (b) mode VB2; (c) mode VB3; (d) mode VB4; (e) mode VB5; (f) mode VB6

snow on the bridge) are summarized in Table 1, including the mean values, the standard deviations, and the extreme values for all natural frequencies. It should be noticed that: (a) Standard deviation is lower than 0.01 for modes TB1–TB4, VB2, TB11,TB12 and (b) vertical modes generally exhibit larger standard deviations and changes in time of the corresponding natural frequency.

To evaluate the effect of changing environmental and operational variables on the fluctuations of the modal frequencies of the bridge, the simplest approach is to plot each frequency with respect to those variables. For example, Fig. 15 shows the natural frequencies of modes VB1–VB6 plotted with respect to temperature along with best fit lines; as in Table 1, the plots of Fig. 15 do not account for the periods with low temperature and ice/snow on the bridge. The inspection of Fig. 15 reveals a clear dependence of the investigated natural frequencies on temperature. Extending the analysis to all modes, the following conclusions can be outlined:

a) The natural frequencies of vertical modes are significantly affected by the temperature. Generally, the modal frequencies exhibit the tendency to decrease with increased temperature, with the exception of the frequency of modes VB1 (Fig. 15(a)) and VB2 (Fig. 15(b)), that increases as temperature increases.

b) The relationships between the natural frequencies of vertical modes and temperature is essentially linear, except for the frequency of mode VB3 (Fig. 15(c)) exhibiting nonlinear dependence on the environmental conditions.

c) The temperature impact on the frequencies of vertical modes tends to be more clear in the hot seasons, as the daily variation of temperature increases.

d) The effects of temperature on the natural frequencies of transversal modes are significant or very significant for modes TB1, TB2, TB4, TB5, TB7, TB9, TB11, TB13 and TB15 whereas are relatively small for modes TB6, TB8, TB10 and TB12. The impact of the environmental conditions seems practically negligible on natural frequencies of modes TB3 and TB14.

e) The natural frequencies of modes TB1–TB3, TB4, TB6, TB11–TB15 have the tendency to decrease with increased temperature and the relationship between frequency and temperature is essentially linear.

f) The relationships between the natural frequencies of

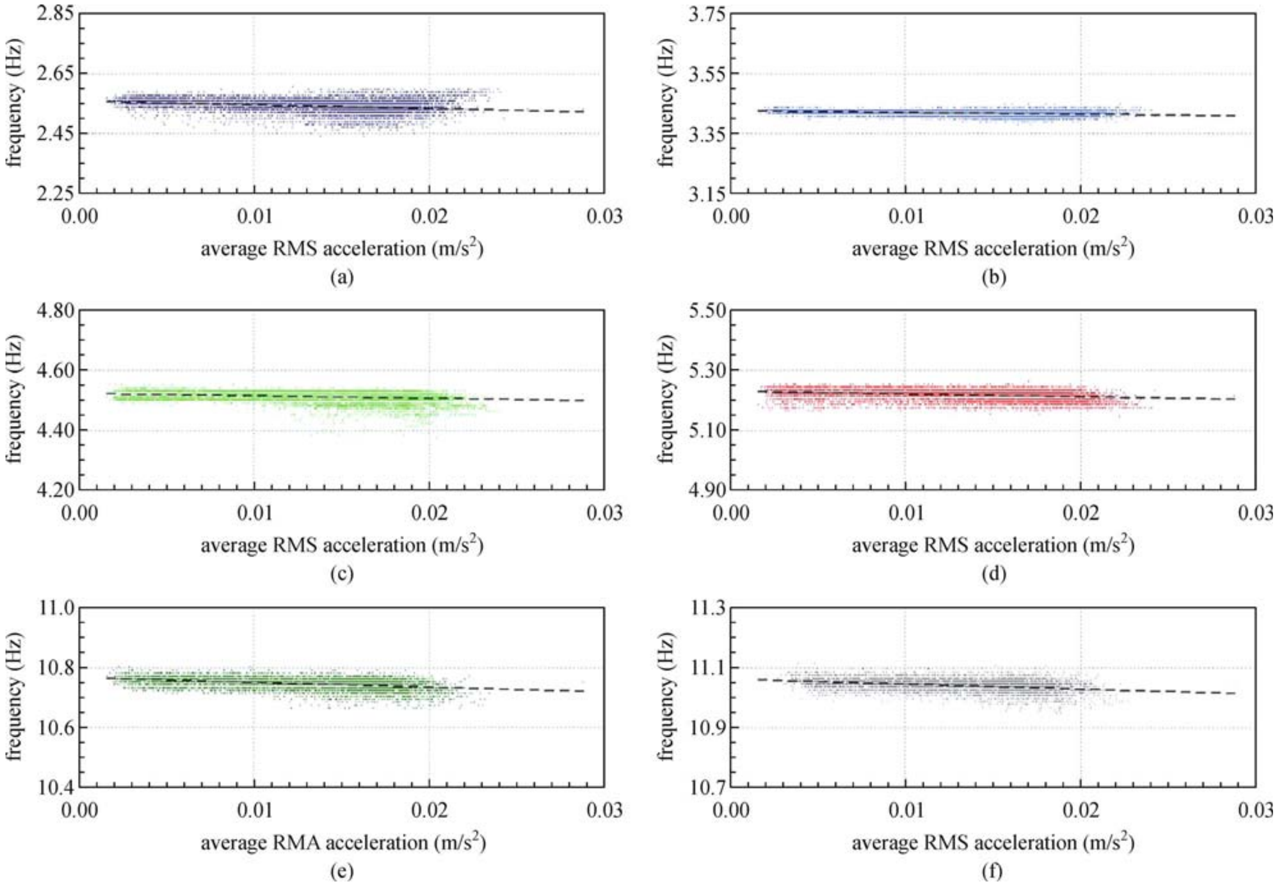


Fig. 16 Correlation between road traffic intensity and modal frequencies identified from 28/11/2011 to 12/05/2013 (excluding the periods with low temperature and ice/snow on the bridge). (a) Mode VB1; (b) mode VB2; (c) mode VB3; (d) mode VB4; (e) mode VB5; (f) mode VB6

modes TB5 and TB7–TB10 is nonlinear and the modal frequencies tend to increase with increased temperature.

Performing the same correlation analysis with respect to the intensity of road traffic (described by the averaged RMS values of vertical and transversal accelerations) provides information on the effects of the excitation/response amplitude. The correlation between natural frequencies and the traffic intensity (and best fit lines) is exemplified in Fig. 16 (where, again, the periods with low temperature and ice/snow on the bridge were not accounted for) for the same modes VB1–VB6 examined in Fig. 15. Since similar plots have been obtained for all modes, it can be stated that all modal frequencies turned out to decrease with increased (road) traffic intensity; furthermore, the transversal modes are often more sensitive to the traffic acceleration than the vertical ones. It is worth underlining that the dependence on the amplitude, already noticed since the tests performed in 2009 [12], suggests a nonlinear behavior of the bridge.

As a further remark, the data collected provide clear indication on the correlation between natural frequencies and environmental/operational conditions; hence, multivariate regression models predicting the evolution of each modal frequency under changing temperature and traffic intensity have been estimated as part of the SHM strategy [23].

To better understand the structural reason of environmental and operational effects on the investigated bridge, it is worth noting that: (1) The rheological characteristics of the metallic elements are not affected by temperature and the asphalt cover on the roadway deck has very limited thickness (as it generally happens for steel orthotropic decks). (2) The variation of modal masses due to road traffic has to be considered negligible (the mass of road vehicles is very small in comparison to the mass of the superstructure also because both weight and speed of road vehicles are limited). Hence, the variation of modal parameters due to temperature and traffic effects is conceivably associated to the same structural phenomenon. In fact, a huge number of truss elements belonging to the girder and the arches are very slender; in addition, a significant corrosion has reduced the area of their cross-

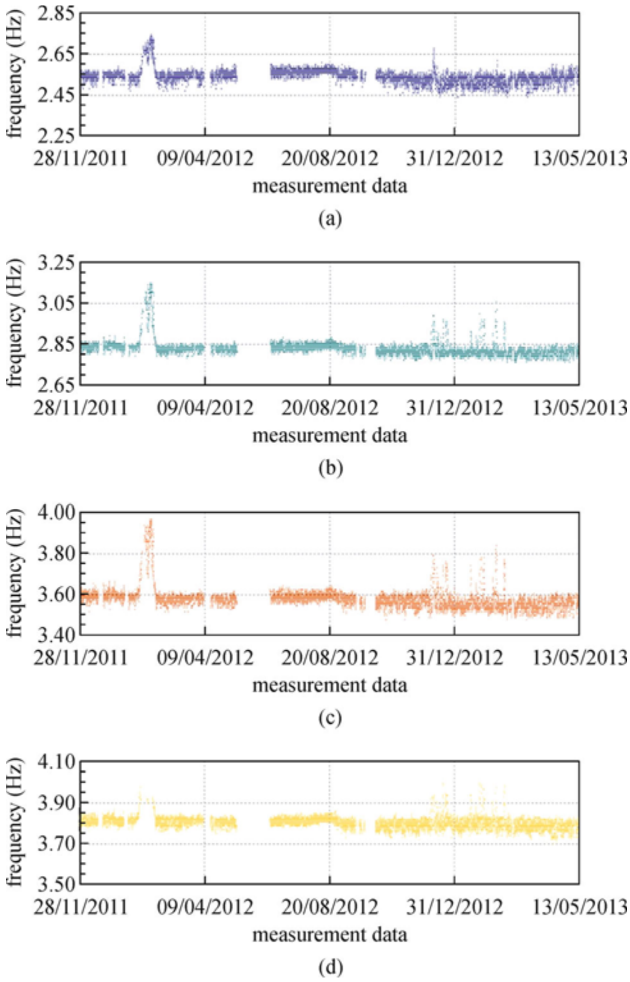


Fig. 17 Time evolution of the natural frequency of modes. (a) VB1; (b) TB7; (c) TB9; (d) TB10

sections (Fig. 4). Consequently, temperature and load variations induce slight variations of the truss axial loads, that might lead to slight changes of the second-order stiffness.

6 Detection of structural performance anomalies

A careful inspection of the frequency tracking (Figs. 14(a)–(b)) reveals that the natural frequencies of modes VB1, TB7 and TB9–TB10 exhibited a clear decrease in the last days of August 2012. As pointed out in section 5.3, the natural frequency of those modes increases with temperature; hence, the decrease has been initially associated with the normal behavior of the bridge since the temperature began to decrease exactly at the end of August (Fig. 13). After few weeks, the direct inspection of the frequency tracking, illustrated in Figs. 17(a)–(d), and data analysis highlighted that: (a) the modal frequency drops were slightly larger than expected and (b) the related standard deviations increased, as well. In addition, the analysis of mode shapes revealed clear changes, as shown in Fig. 18, where the mode shapes of the vertical modes identified on 09/03/2012 (23:00–24:00) and 30/11/2012 (17:00–18:00) are compared.

Figures 18(e)–(f) shows that the region mainly involved in the changes correspond to the crown of the arch on the Calusco side of the bridge (i.e., the region neighboring the cross-section where the Sensors 4, 11 and 18 are installed, see Fig. 5). Indeed, visual inspection of this region did not reveal any concentrated damage but highlighted that the arch crown exhibits a state of preservation worse than neighboring regions, with higher corrosion of the structural members. The higher corrosion, detected in the zones affected by the major changes of modes VB5–VB7 (Fig. 18), was conceivably determined by less accurate carrying out the last protective re-painting of those regions, dating back to the late 50s.

7 Conclusions

A dynamic monitoring system has been installed in a centenary iron arch bridge crossing the Adda river about 50 km far from Milan. The monitoring system and the software developed in LabVIEW for automated processing of the collected data are described in the paper.

An automatic version of the FDD method was developed and its application to the acceleration data sets collected in the period from 28/11/2011 to 12/05/2013 was presented and discussed. The automated OMA tool provided very good results, as all the expected modes (including the closely spaced ones) are clearly identified with high occurrence.

Furthermore, the work covered has addressed which environmental/operational conditions drive the changes observed in the identified modal frequencies. The natural frequencies of all modes turned out to decrease with increased (road) traffic intensity and the transversal modes are generally more sensitive to the traffic acceleration than the vertical ones. The temperature affects almost all the natural frequencies as well but its effect is nonlinear for some modal frequencies and not always characterized by a frequency decrease with increased temperature.

Notwithstanding the quite complex mechanisms that define the normal response of the structure under changing temperature and traffic conditions, multivariate regression models turned out to be appropriate to predict the modal frequency changes of the bridge, given the measured environmental/operational conditions, and to address the SHM strategy of the bridge [23].

Furthermore, careful data analysis and inspection of frequency tracking, together with the information provided by the mode shapes, allowed to detect structural

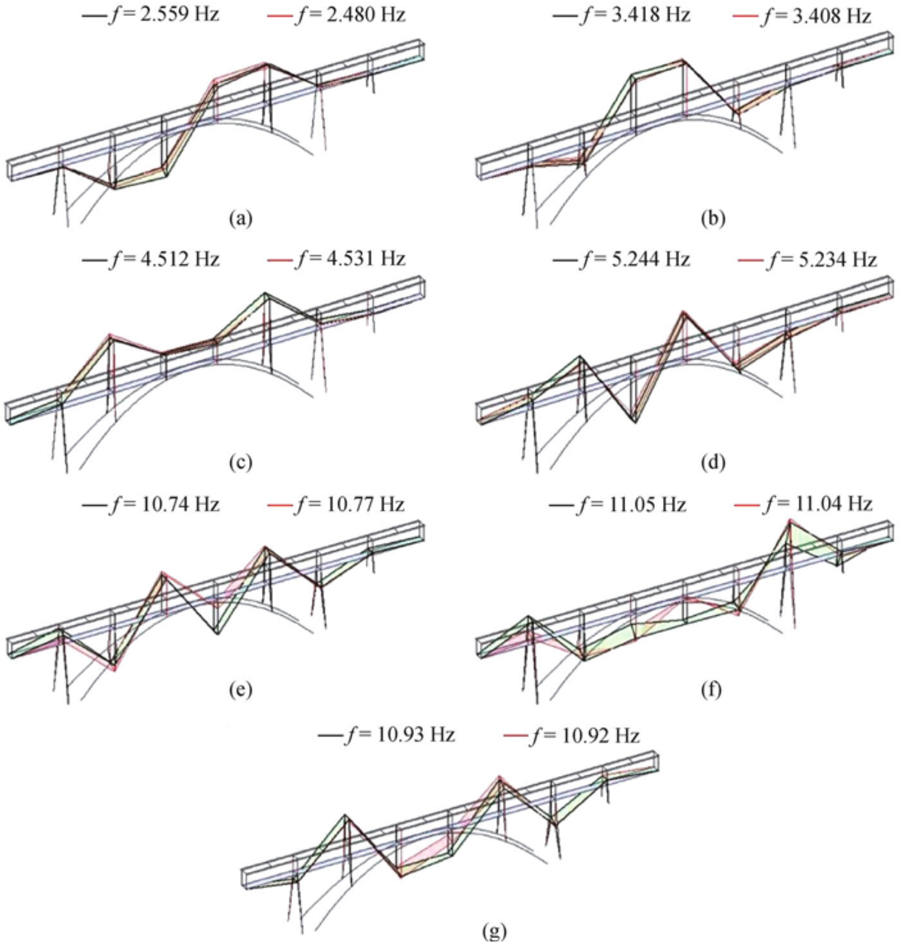


Fig. 18 Vertical bending modes: comparison between the mode shapes identified on 09/03/2012 (black line**)** and 30/11/2012 (red line). (a) VB1; (b) VB2; (c) VB3; (d) VB4; (e) VB5; (f) VB6; (g) VB7

performance anomalies and changes in the dynamic characteristics of the bridge, conceivably related to the progress of the damage due to corrosion.

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