Available online at www.sciencedirect.com

ScienceDirect

Procedia Engineering 199 (2017) 3015–3020

Procedia Engineering

www.elsevier.com/locate/procedia

X International Conference on Structural Dynamics, EURODYN 2017

Experimental results in damping evaluation of a high-speed railway bridge

M. Brunetti[a], J. Ciambella[a], L. Evangelista[b], E. Lofrano[a], A. Paolone[a], A. Vittozzi[b]

[a]*Dept. of Structural and Geotechnical Engng., Univ. "La Sapienza", Via Eudossiana 18, Rome 00184, Italy*
[b]*Italferr S.p.A., Via Vito Giuseppe Galati 71, Rome 00155, Italy*

Abstract

Structural damping in high-speed railway bridges is a key parameter that affects significantly the performance of the structure in terms of fatigue life and comfort of the passengers and hence has a significant impact on the design procedures. For these reasons its accurate assessment is a paramount task to be performed and hence the results of an experimental campaign carried out on a prestressed concrete (four box-girder) beam bridge of the new Italian high-speed network are presented in this work. A total amount of fifteen accelerometers were used to ensure redundancy of measurements. Both a time domain technique and a Frequency-Domain Decomposition (FDD) technique are used to analyse the vibration signals and extract the modal frequencies, shapes and damping ratio of the first mode of the structure. The results are compared with the design provisions, and with other expectation provided by the technical literature. The paper from one hand provides a contribution in the framework of the damping evaluation of high-speed railway bridges, and from the other hand would furnish a first assessment of those damping sources that are not usually accounted for in the modelling process.

Peer-review under responsibility of the organizing committee of EURODYN 2017.

Keywords: Damping identification, high-speed railway, experimental test, modal analysis, energy dissipation, beam bridge, prestressed concrete

1. Introduction

The accurate evaluation of the structural damping in railway bridges is a crucial task to be performed to correctly assess the structural performance and evaluate the maximum vibration amplitude and vertical acceleration expected under dynamic loading conditions. The quantification of damping is not an easy task due to the interaction among tracks, structures, bearings and foundations. A quantitative evaluation can be gained from the bridge components (tracks with ballast or ballastless, steel or concrete structure...) but large safety margins are usually prescribed by the design codes [5,13] or guidelines [12]. Indeed, in real-world railway bridges there are a number of non-structural *secondary* components that can increase the stiffness and damping capacity without being accounted for by the design models. In terms of additional damping capacity these secondary elements are: tracks, joints, bearings, others, e.g., the handrails. In the open literature, a number of papers has dealt with the evaluation of stiffness increase provided by the non-structural components [1,4], whereas the assessment of the dissipation has so far received less attention and only few contributions are available [11]. It is further remarked that the damping ratio could have a significant effect on the identification of structural damages [2,3,9].

1877-7058 © 2017 The Authors. Published by Elsevier Ltd.
Peer-review under responsibility of the organizing committee of EURODYN 2017.
10.1016/j.proeng.2017.09.402

This paper summarises the results of the experimental campaign carried out by the authors and it is organized as follows: section 2 recalls the damping values suggested by the literature (both design codes and guidelines are considered), section 3 shows the experimental setup and results, section 4 highlights the main conclusions of the work.

2. Damping values prescribed by design codes and guidelines

In the technical literature several estimate of the damping ratio for railway bridges can be found yet they are sometimes conflicting. While the Italian 2008 national code [5] does not provide any indication of the dissipative capacity of railway bridges, the table 6.6 of the Eurocode 1 "Actions on structures" - Part 2 "Traffic loads on bridges" [13] furnishes the damping values according to the type of structural system and the span length of the bridge. As a further indication, the internal guideline DTC-INC-PO-SP-IFS-001-A of the Italian society "Rete Ferroviaria Italiana S.p.A." (RFI) [10] gives the damping ratios starting from the type of connections among structure and track (ballast or ballastless), table 1.4.2.6.3.1-1. Table 1 summarises these values.

Table 1. Damping ratios suggested by the Eurocode 1-2 (on the left) and the RFI guideline (on the right).

Bridge type	Damping ratio, %	
	Span L < 20 m	Span L > 20 m
Steel and composite	0.5+0.125(20-L)	0.5
Prestressed concrete	1.0+0.070(20-L)	1.0
Filler beam and reinf. concr.	1.5+0.070(20-L)	1.5

Connection type	Damping ratio, %
Ballastless	1.5
Ballast	4.0

The European Commission in the "Guideline for estimating structural damping of railway bridges" [12] gives an estimate of the damping ratio provided by the different sources. In this guideline, according to [8], the damping ratios are evaluated dividing the difference sources of dissipation into three main contributions: the damping due to the material, the nonmaterial structural damping and the friction of the supports. The overall damping ratio will be the sum of the three contributions listed in Tab. 2.

Table 2. Damping ratios suggested by the European Commission's project D5.2-S2.

Material damping	Damping ratio, ‰
Steel	0.8
Reinforced concrete: uncraked, craked	4.0, 7.2
Prestressed concrete	4.0
Masonry	8.0
Natural stone	8.0
Nonmaterial structural damping	**Damping ratio, ‰**
Steel bridge: welded, bolted, riveted	2.4, 3.2, 3.2
Composite bridges, steel girders and concrete deck	6.4
Reinforced and prestressed concrete bridges	3.2
Masonry and natural stone bridges	(not available)
Ballast	3.2
Interaction damping	**Damping ratio, ‰**
Pendulum and roller bearings	0.8
Standard sliding bearings (not base isolation devices)	2.4
Monolithic concrete bearings	1.6

Although, previous tables furnish an initial estimate of the damping capacity of the bridge (constant for all the vibration modes), the EC document itself highlights that those values are affected by a certain degree of uncertainty and are likely to be smaller than the real ones.

3. Experimental evaluation

The experiments have been carried out on the Cave's viaduct of the Italian high-speed line RM-NA (Rome-Naples); it is located between the km 13+410.80 and 14+032.40, near the new station of "Ponte di Nona", in the Rome suburb, Fig. 1. The tests, performed in collaboration with "Italferr S.p.A." (supervisor of the network), were carried out in operational conditions, guaranteeing the functionality of the bridge.

Fig. 1. The Cave's viaduct: top view (top) and lateral view (bottom).

The main structural properties of the bridge (isostatic and with continuous track) are:

- total length of 621.60 m, width of 13.60 m, track with ballast;
- 26 spans, span length of 24.00 m (first and last spans of 22.80 m);
- reinforced cast-in-place concrete deck on 4 V-shaped girders, prefabricated and prestressed, connected by 4 cast-in-place cross girders;
- hollow bridge-piers of height variable between 5.00 and 9.50 m and cross-section of 3.60 x 8.80 m;
- abutments of height 4.50 m, Rome side, and 7.00 m, Naples side;
- piers and abutments foundations are slab foundations with piles; these have a diameter of 1200 mm, and a variable length (up to the lithoid substrate).

For the Cave's viaduct, both the Eurocode 1-2 and the project of the European Commission furnish a damping value of the 1 %, while the RFI guideline suggests a value four times greater (see section 2).

3.1. Experimental setup

The bridge has been tested the 28th of September 2016. The equipment was a "Bridge XP" unit, consisting of a 15 channels data acquisition system and 15 3-axis accelerometers (sensitivity of 5 V/g and sampling frequency of 10 kHz). Fig. 2 shows two pictures of the acquisition system. Two adjacent spans of the bridge have been instrumented to check the potential onset of relative couplings due to secondary components, see Fig. 3; all the sensors have been installed on the V-shaped girders, on the inner face of the bottom edge.

As stated before, the tests were carried out under operational conditions (none *ad hoc* excitation system has been used). Three family of high-speed trains crossed the line during the tests: ETR 500 and ETR 1000 of "Ferrovie dello Stato S.p.A.", AGV 575 (the so called "Italo") of "Nuovo Trasporto Viaggiatori S.p.A.". These trains have different properties in terms of length, mass and maximum speed. Therefore, even if the passing speed on the Cave's viaduct is set close to 200 km/h (due to the proximity to an urbanized area), the excitation transmitted to the structure was different. A total amount of 15 train passages were recorded, getting 225 acceleration signals (15 accelerometers have

Fig. 2. The acquisition system: one of the sensor (left) and the data acquisition (right).

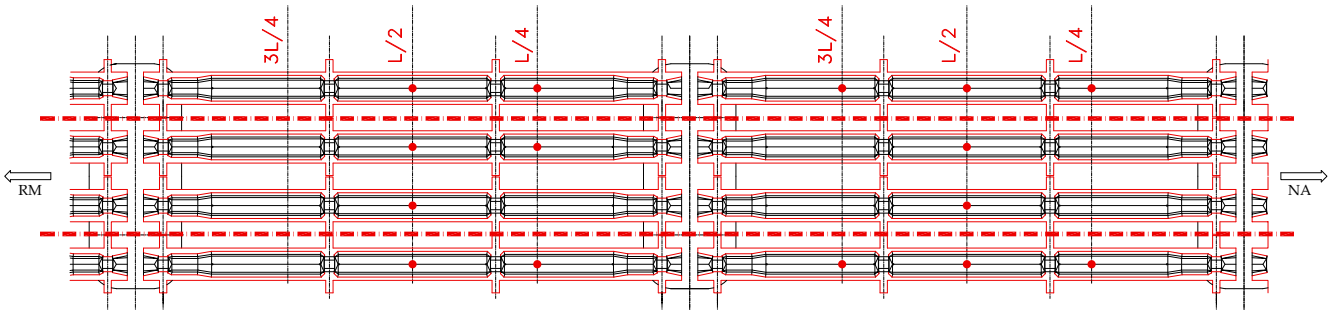

Fig. 3. Sensors placement (red dots); lengths are expressed as distances from the bearings alignment on Naples side, being *L* the distance between the two alignments of the span bearings.

been used). For the identification purposes, the free oscillation parts of the vertical accelerations were considered, Fig. 4.

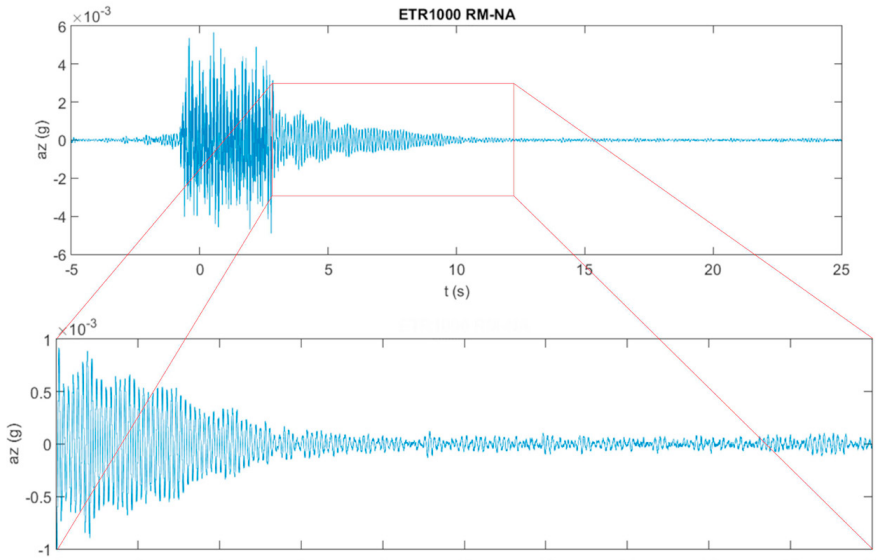

Fig. 4. Time history of a vertical acceleration: full (top) and windowed (bottom) signal.

3.2. Results

The main dynamic features of bridge under investigation were assessed by a coarse numerical model implemented in the FE-based software SAP2000 (version 18.1.1). "Shell-thick" elements with 4 nodes (24 dof) or 3 nodes (18 dof) are used to discretize a single span of the bridge (in truth the viaduct is curved, but the curvature is so small that can be considered negligible); the maximum dimension of the edge is fixed to 50 cm. The material is assumed linearly elastic, homogeneous and isotropic, while rigid links are used among the medium planes to simulate the inter-connections. The structural mass is directly evaluated by the code, while the mass sources related to non-structural weights are estimated as 18 kN/m^3. The resulting model has 4234 nodes, 4364 elements and 25404 dof, see Fig. 5. In the same figure the bearings are sketched: 2 fixed (F) and 2 multi-directional (M) devices on one side, 1 one-directional (U) and 3 multi-directional (M) devices on the opposite one. Lastly, also the first and second mode-shapes obtained with a modal analysis are showed: the first one is a flexural mode (period of 0.139 s and frequency of 7.18 Hz), while the second shape is a torsional mode (period of 0.093 s and frequency of 10.73 Hz).

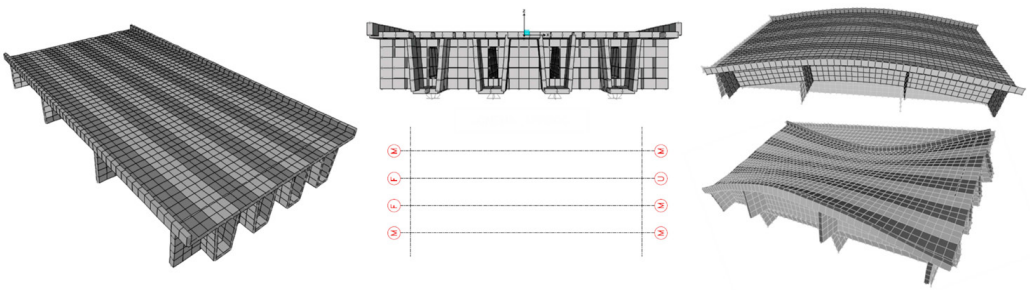

Fig. 5. FE model of a span: lateral view (left), cross-section (middle-top) and bearings (middle-bottom), first (right-top) and second (right-bottom) mode-shape.

Each time history of the 225 experimental accelerations was then analyzed. First of all, a preprocessing of the data is carried out, recurring to operations of signal windowing, detrending, filtering and fitting. Hence, the damping ratios are gathered using the two following techniques [6,7]: the logarithmic decrement method (in the time domain) and the half-power bandwidth method after a Frequency-Domain Decomposition technique was used to assess the modal shapes and frequencies. As an example, Fig. 6 shows the application of the half-power bandwidth method for the signal in Fig. 4; in the plot, the first frequency and the relevant damping ratio are detected. The skeleton of the identified first mode-shape (obtained considering the simultaneous vibrations of all the sensors) is also sketched in the figure: the coupling between the two adjacent (instrumented) spans is clearly highlighted, as well as the coupling among the first (flexural) and the second (torsional) mode-shapes provided by the numerical simulation. This means that secondary components behave like an internal constraints between the spans.

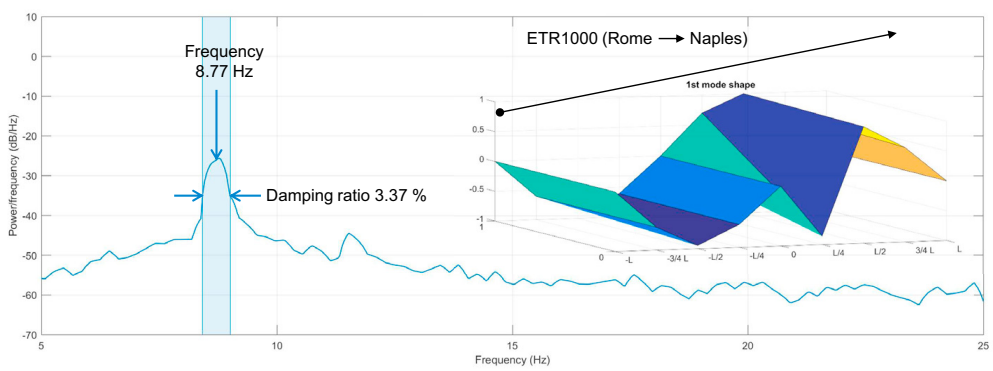

Fig. 6. Application of the half-power bandwidth method and skeleton of the identified first mode-shape.

Table 3 summarizes the statistics of frequencies and damping ratios by type of train (for the sake of brevity, only the results related to the first mode-shape are here showed): μ is the mean value, σ the standard deviation and CV the coefficient of variation; for the damping ratios the table shows also μ_5, the fractile 5 %.

Table 3. Experimental results for the first mode-shape.

Train	Frequency			Damping ratio			
	μ, Hz	σ, Hz	CV, %	μ, %	σ, %	CV, %	μ_5, %
ETR 500	8.77	0.15	2.80	4.80	1.07	22.30	3.04
ETR 1000	8.77	0.16	1.80	4.85	1.28	26.40	2.76
AGV 575	8.77	0.02	0.20	4.10	0.24	5.90	3.70

The results point out that the frequency is well identified, since all the trains provide the same mean value, with a small uncertainty. The average experimental frequency (8.77 Hz) is the 22.1 % higher than the numerical prediction (7.18 Hz), confirming the stiffening effect of the secondary elements discussed in section 1. About the damping ratio, the mean values are roughly similar for all the trains, and in good agreement with the previsions of the RFI guideline (4 %); the value of 1 % provided by the Eurocode 1-2 and by the European Commission seems too precautionary instead. It should be noted how the dispersion obtained for the damping is quite high, especially for ETR 500 and ETR 1000. However, looking at the fractile 5 %, a value equal to the 2.76 % appears a trustworthy and reliable estimate of the actual dissipative capacity of the bridge. Also the damping dependence on acceleration magnitude should be analyzed, in order to check if the results can be assumed amplitude-independent or not.

4. Conclusions

This paper has dealt with the experimental evaluation of the dissipative capacity of a high-speed railway bridge (Cave's viaduct) of the Italian high-speed line RM-NA. As a result of the measurements, by considering a fractile of 5 %, a damping value of 2.76 % has been obtained. Such a value is higher than the damping ratio of 1% provided by the Eurocode 1-2 [13] and by the EC guidelines [12], yet is less than the 4 % considered in the RFI guidelines [10]. This discrepancy suggests that an accurate evaluation of the different damping sources is still needed but confirms that the damping ratios provided by the design codes has large and too limiting safety margin. It is therefore suggested that a deep revision of the topic is needed and as a first instance a broader experimental campaign taking into account different bridge structural topologies should be carried out.

References

[1] L. Bornet, A. Andersson, J. Zwolski, J.M. Battini, Influence of the ballasted track on the dynamic properties of a truss railway bridge, Struct Infrastruct E, 11(6): 796–803, 2015.
[2] D. Capecchi, J. Ciambella, A. Pau, F. Vestroni, Damage identification in a parabolic arch by means of natural frequencies, modal shapes and curvatures, Meccanica 51(11): 2847-2859, 2016.
[3] J. Ciambella, F. Vestroni, The use of modal curvatures for damage localization in beam-type structures, J Sound Vib 340: 126-137, 2015.
[4] G. Chellini, L. Nardini, W. Salvatore, Dynamical identification and modelling of steel-concrete composite high-speed railway bridges, Struct Infrastruct E 7(11): 823–841, 2011.
[5] Consiglio Superiore dei Lavori Pubblici, Norme Tecniche per le Costruzioni - Decreto Ministeriale 14/01/2008 (in Italian), 2008.
[6] D.J. Ewins, Modal testing: theory and practice, second ed., Research Studies Press, Philadelphia, 2000.
[7] Z.F. Fu, J. He, Modal analysis, Elsevier Science, Woburn, 2001.
[8] O. Klingmller, M. Lawo, G. Thierauf, G., Stabtragwerke: Matrizenmethoden Der Statik Und Dynamik, Braunschweig, Vieweg, 1983.
[9] E. Lofrano, A. Paolone, M. Vasta, Dynamic identification of classically damped uncertain structures, Proc. Int. Conf. IMAC 2016, Orlando (USA), 25-28 January 2016
[10] R.F.I. S.p.A., Specifice per la progettazione e l'esecuzione dei ponti ferroviari e di altre opere minori sotto binario DTC-INC-PO-SP-IFS-001-A (in Italian), 2011.
[11] P. Salcher, H. Pradlwarter, C. Adam, Reliability of high-speed railway bridges with respect to uncertain characteristics, Proc. Int. Conf. EURO-DYN 2014, Porto (Portugal), 30 June - 2 July 2014.
[12] Skanska Teknic (project co-funded by the EC), Guideline for estimating structural damping of railway bridges, Back. Doc. D5.2-S2, 2007.
[13] Technical Committee CEN/TC 250, Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges, EN 1991-2, 2003.