

Available online at www.sciencedirect.com**ScienceDirect**

Procedia Engineering 199 (2017) 3021–3026

**Procedia
Engineering**www.elsevier.com/locate/procedia

X International Conference on Structural Dynamics, EURODYN 2017

Investigation of the dynamic response and effect of soil properties of Arroyo Bracea II bridge in Madrid-Sevilla High-Speed railway line through experimental analyses

P. Galvín^a, E. Moliner^{b,*}, A. Romero^a, M.D Martínez-Rodrigo^b^a*Escuela Técnica Superior de Ingeniería, Universidad de Sevilla, 41092, Spain*^b*Mechanical Engineering and Construction Department, Universitat Jaume I, Castellón, 12071, Spain*

Abstract

In this contribution the authors include results and conclusions from an experimental and numerical analysis of a railway bridge belonging to the Madrid-Sevilla High-Speed railway line in Spain. This structure is monitored due to its short length and typology, which make it susceptible to experience high transverse vibration levels. During the in-situ tests the soil properties at the site were obtained. Also, the response of the structure under the circulation of railway convoys was measured at several points of the deck and at the abutments. From the experimental measurements the modal parameters of the bridge are identified. Finally the experimental results are compared to those provided by a finite element numerical model in the time and frequency domains. Conclusions are extracted regarding the structure performance and the adequacy of the numerical model implemented.

© 2017 The Authors. Published by Elsevier Ltd.

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

Keywords: Railway bridges; experimental measurements; resonance; traffic induced vibrations; bridge dynamics; soil-structure interaction

1. Introduction

With the advent of High-Speed trains, the Serviceability Limit State of vertical acceleration is one of the most demanding specifications for the design or upgrading of many railway bridges. At speeds above 200 km/h, resonance effects caused by the regular nature of train axle loads may entail inadmissible levels of vertical vibrations at the deck level, leading to harmful consequences in the superstructure. Especially critical in this regard are short-to-medium span simply-supported (S-S) bridges with usually low associated structural damping and mass [1,2]. These facts point out the importance of developing accurate numerical models, able to realistically predict the vibration levels in the bridge with reasonable computational costs. In this regard, the calibration of these models with in situ dynamic testing becomes crucial. With this purpose a number of researchers have performed experimental campaigns on railway bridges in the past [3–5]. In these studies, usually performed on structures of long spans, the attention is focused on the dynamic response of the bridge deck under ambient or train induced vibrations, with the aim of

* Corresponding author. Tel.: +34-964387468; fax: +34-964728106.

E-mail address: molinere@uji.es

identifying its modal properties. However, as indicated in [6], the dynamic response of short-to-medium span S-S railway bridges is difficult to predict during the design or upgrading stages, since the influence of the environment and the super-structure components (rails, ballast) can be significant and it is not well known. Despite this fact, the number of reported experimental campaigns on these structures is low. In [6] the authors perform in situ dynamic testing in short-to medium span S-S slab bridges belonging to a conventional railway line to be upgraded for higher operating speeds. Although some studies reveal that soil-structure interaction (SSI) may significantly affect the bridge dynamic response [7], especially in short S-S structures, the properties of the soil are seldom measured during the tests nor included in the numerical FE models. Only when the interaction between the super-structure and the soil is more evident due to the bridge typology, as is the case with portal frames or soil-steel composite railway bridges [8,9], the SSI is taken into account. In this work the results and conclusions from an experimental campaign performed on a short S-S bridge belonging to a High-Speed line in Spain are included. The main novelty is the measurement of both the soil properties and the super-structure accelerations during the tests, and the implementation of a suitable numerical model according to the results of the experimental campaign. Also this bridge is particularly interesting as (i) the structure has a skew angle of 45° with a span length similar to the deck width, therefore its dynamic response substantially differs from that of a beam-like structure; and (ii) in a preliminary numerical evaluation [10], using a three dimensional finite element (FE) model that follows the main simplifications adopted by practitioners in accordance with the European Standards [11,12], an important transverse vibration response was predicted at the deck level. Conclusions are extracted regarding the structure performance and the adequacy of the numerical model implemented for the particular soil and loading conditions.

2. Structure description

The structure under study, designated as Arroyo Bracea II bridge, belongs to the Madrid-Sevilla High-Speed railway, opened in 1992. It is composed by two identical S-S spans of 15.25 m, with a 45° skew angle as shown in Fig.1. Each deck consists of a concrete slab resting over five pre-stressed concrete I girders. The girders lean on the supports through laminated rubber bearings. As per the substructure, the bridge deck is supported on reinforced concrete abutments in its outermost sections and the inner sections of both bays lean on a pile foundation.

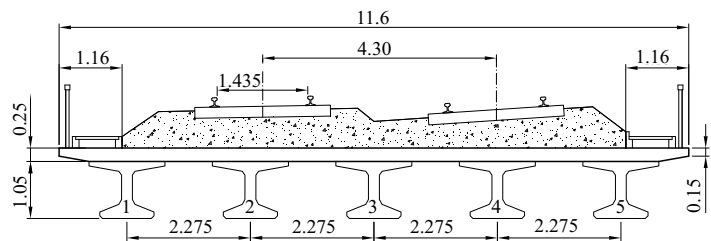


Fig. 1. Arroyo Bracea II Bridge (from left to right): accelerometers set up and bridge deck cross section

3. Experimental campaign

The experimental program, carried out in July of 2016, included a dynamic characterization of the structure and of the soil at the bridge site. The acquisition equipment was a portable system LAN-XI of Bruel & Kjaer. The acquisition system fed the accelerometers and the impact hammer used in the tests, and also performed the Analog/Digital conversion (A/D). The A/D was performed at a high sampling frequency and aliasing effect was avoided applying a low-pass filter with constant cut-off frequency to the digital signals. The sampling frequency was $f_s = 4096$ Hz. The acquisition equipment was connected to a laptop for data storage. The dynamic characterization of the soil was done

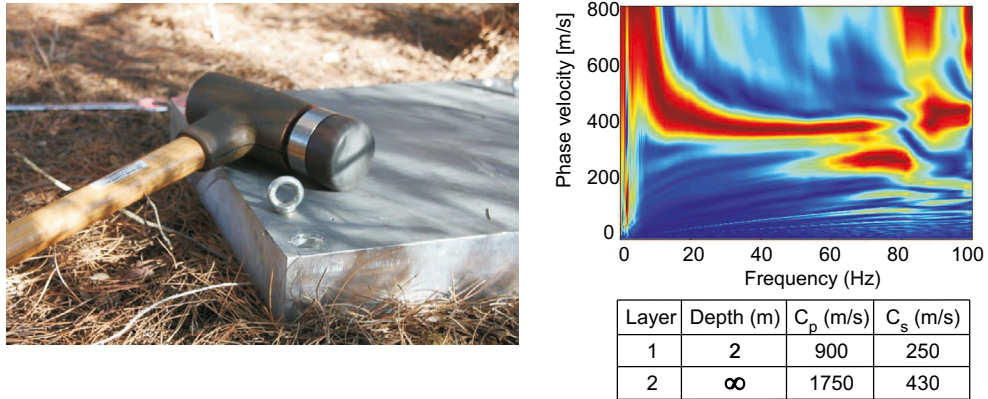


Fig. 2. From left to right: impact hammer and aluminum foundation; dispersion curve and soil properties

by (i) the seismic refraction test, that allowed the identification of the P-wave velocity (C_p) of the soil layers; and (ii) the spectral analysis of surface waves (SASW) test to determine the S-wave velocity (C_s) and the material damping ratio of the soil layers [13].

The measurements were performed in two setups. In each setup, 100 impacts were applied to a 50 cm × 50 cm × 8 cm aluminium foundation anchored to the soil surface. The instrumented hammer was a PCB 086D50 force sensor. The vertical free field response was recorded by means of accelerometers anchored to the soil surface every 2 m, (2 to 68 m distant from the foundation). Steel stakes of cruciform section and 0.3 m length were driven into the ground for mounting each of the accelerometers. After each impact, a time signal of 16348 samples (4 s) was stored. The force channel was used as a trigger, with a trigger level of 10 N, a pre-trigger of 4096 samples (or 1 s), and a post-trigger of 12288 samples (or 3 s). The signals were decimated (order 4), filtered with a third-order Chebyshev filter with a high-pass frequency of 5 Hz and with a third-order Chebyshev filter with a low-pass frequency of 100 Hz. On the right hand side of Fig.2 the resulting dispersion curve along with the soil properties obtained from the resolution of the inverse problem are shown. The dynamic characterization of the soil revealed a very stiff soil at the site.

The acceleration response of the structure was measured at 11 points of the lower flange of the pre-stressed concrete girders and at the pile abutment upper horizontal surface, close to the girders support. For the sake of conciseness, only the results at accelerometer 4, located at a quarter of the span center of girder 3, and at point 5, located at the span center of girder 1 (see girders in Fig.1) will be shown. Endevco model 86 piezoelectric accelerometers were used with a nominal sensitivity of 1000 mV/g and a low frequency limit of approximately 0.1 Hz. The sensors were attached to the deck using circular aluminum plates of 0.09 m diameter and 0.006 m thickness, glued with epoxy resin to the girders.

The ambient vibration response was acquired in 4184 seconds per channel. Data were decimated (order 16) to carry out data analysis in the frequency range of interest (0 to 30 Hz). The signals were filtered applying two third-order Chebyshev filters with high-pass and low-pass frequencies of 1 Hz and 30 Hz, respectively. The same filter is applied to the numerical response. During the recording time, several trains crossed the bridge with circulating velocities in the range [100, 280] km/h. In the next section the response of the structure is presented for two of these trains: a Talgo 350 with an axle scheme as that shown in Fig.3a and a coach distribution type L-1-10×2-1-L, and an ICE 350 (Fig.3b) with a coach distribution 1-6×2-1.

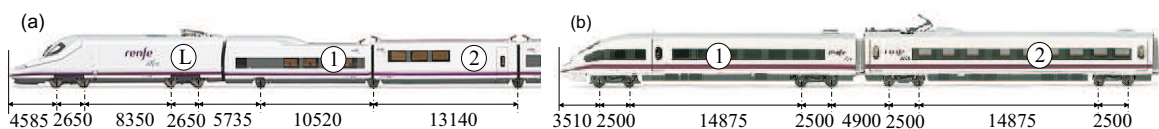


Fig. 3. (a) Talgo 350 and (b) ICE 350 trains. Coach distribution and axle distances

4. Numerical model description

A finite element numerical model of the bridge is used in order to compare the predicted response with the experimental measurements. In a first approach, and due to the stiff soil properties that were measured at the site, SSI is not included in the model. Fig.4 shows the main features of the model, which are the following: (i) the deck behavior is simulated by means of an orthotropic thin plate discretised in Linear Varying Curvature FEs. These are C1 compatible triangular elements with 12 degrees of freedom [14]; (ii) the laminated rubber bearings of the deck girders are included in the model as an equivalent vertical stiffness uniformly distributed along the supports lines; (iii) different mass density elements are employed in order to concentrate the weight of the ballast, sleepers and rails at the central portion of the plate; (iv) a moving load model is adopted for the railway excitation, therefore neglecting vehicle-structure interaction effects and also rail irregularities. As the track rigidity is not included in the numerical model and in order to avoid un-realistic high-frequency modal contributions when a load enters or exits the bridge due to the presence of the elastic bearings, the gradual nature of the wheel loads application process close to the abutments due to the distributive effect of rails, sleepers and ballast, is simulated. To this end, the value of each axle load is modulated at the deck entrance and exit vicinity applying a function based on the Zimmerman-Timoshenko solution for an infinite beam on Winkler foundation [10]. Finally, the dynamic equations of motion are transformed into modal space and numerically integrated by the Newmark-Linear Acceleration algorithm taking into account the first 20 natural modes of the structure. Initially, the plate orthotropic constants, mass and the supports flexibility are obtained from the bridge geometry and mechanical properties. Subsequently the model properties are updated to reproduce the proof load test performed on the structure prior to its opening [10]. In the following sections, conclusions regarding its effect under resonant and not-resonant conditions of the structure will be exposed.

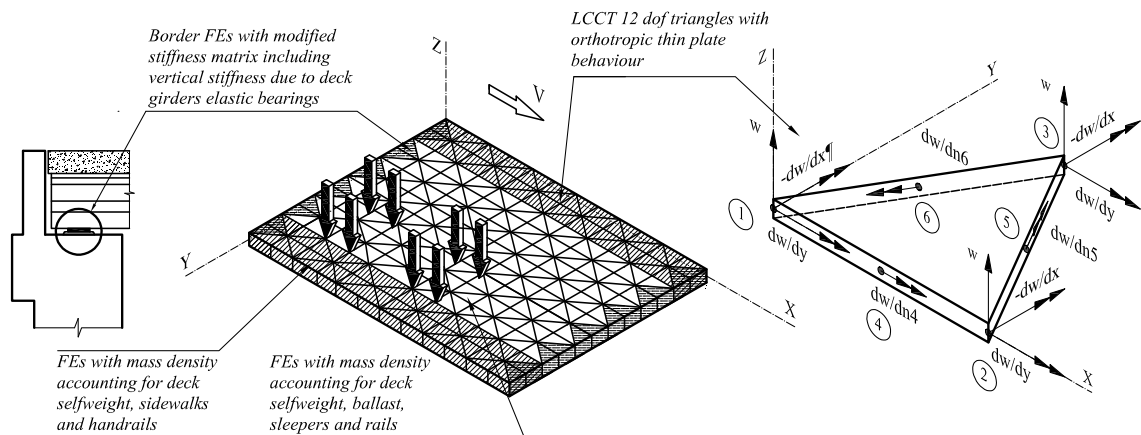


Fig. 4. Finite Element model main features

5. Experimental vs. numerical results

5.1. Modal parameters

First, the modal parameters of the bridge are identified from ambient vibration data by state-space models using MACEC software developed by the Structural Mechanics Section of KU Leuven [15]. The deck presents five modes in the frequency range [0, 30] Hz, where the lowest ones in frequency order correspond to the first longitudinal bending, first torsion and first transverse bending modes. Table 1 shows the identified natural frequencies and the damping ratios for the first five modes, the numerical frequencies provided by the FE model and the correlation numerical/experimental according to the MAC values [16]. The correspondence between them is reasonably accurate.

Table 1. Experimental and numerical natural frequencies, experimental modal damping and AutoMAC values.

| | f_{exp} (Hz) | f_{num} (Hz) | ζ_{exp} (%) | AutoMAC() | | f_{exp} (Hz) | f_{num} (Hz) | ζ_{exp} (%) | AutoMAC() |
|---|----------------|----------------|-------------------|-----------|---|----------------|----------------|-------------------|-----------|
| 1 | 9.25 | 9.19 | 3.64 | 0.98 | 4 | 17.94 | 17.20 | 0.80 | 0.96 |
| 2 | 10.63 | 10.34 | 1.66 | 0.96 | 5 | 24.58 | 24.81 | 0.95 | 0.98 |
| 3 | 12.75 | 12.65 | 1.30 | 0.97 | | | | | |

5.2. Transient structure response under the circulation of High-Speed trains

In Fig.5 the vertical acceleration time history response and its frequency contents are represented at sensors 4 and 5 for two train passages: Talgo-350 at $v = 263$ km/h and ICE-350 at $v = 279$ km/h. Only the load passage excitation is considered in the numerical model as it dominates the response at the track [17]. The computed results show a reasonably good agreement with those experimentally recorded. In the frequency domain peaks coinciding with the bogie passing frequency, the axle passing frequency and at the natural frequencies of the structure may be detected. It should be noted that the circulation of Talgo-350 train does not induce resonance on the bridge deck. In this case, several modal contributions are responsible for the deck response below 30 Hz. In this particular case, where no clear resonance takes place the experimental measurements are slightly higher than the numerical prediction. On the

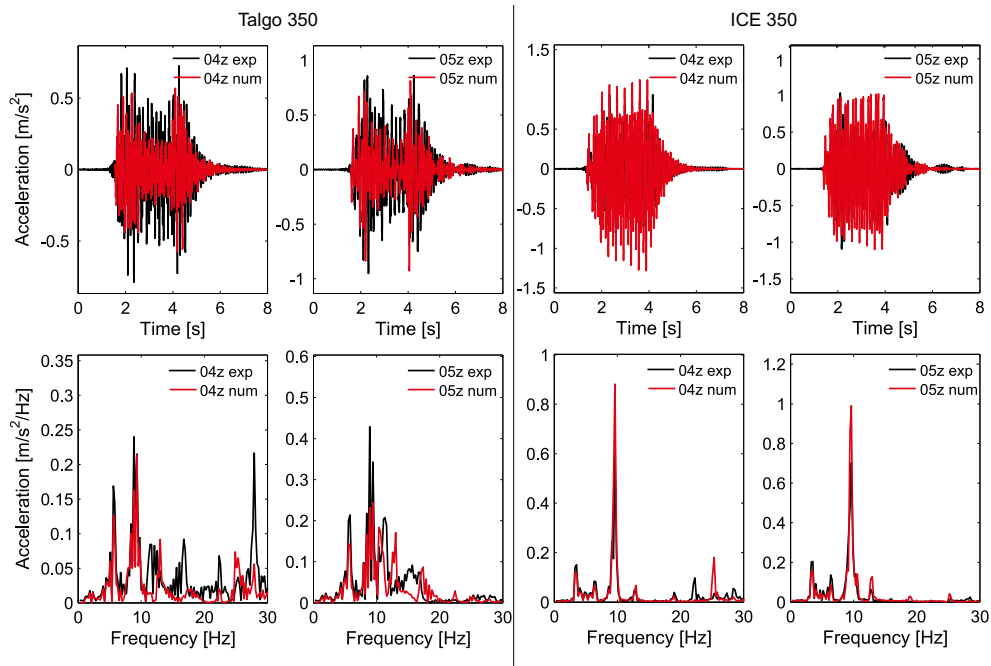


Fig. 5. Acceleration response in time and frequency domains under the passage of a S102 Talgo 350 train at $v=263$ km/h

other hand, train ICE-350 travelling at 279 km/h induces a third resonance of the fundamental mode of the bridge at a theoretical velocity of $v_{res,3} = 3.6 \cdot d_{car} \cdot f_1/3 = 275$ km/h associated to the length of the cars repetitive distance ($d=24.775$ m). In this case, the peak associated to the resonant mode predominates in the frequency response and the contribution of other frequency contents is less relevant in comparison. It should be noted that even though the correspondence between experimental and numerical results is even better in this case, the numerical model overestimates the real response, specially at the fundamental frequency of the structure, associated to the resonant response. This is consistent with the fact that the effect of wave radiation exerted by the soil and energy dissipation related with vehicle-structure interaction are both especially noticeable at resonant velocities [18].

6. Conclusions

The experimental dynamic response of a short S-S railway bridge under High-Speed traffic is presented and compared to numerical predictions. The structure modal parameters are identified from ambient vibration measurements showing good correspondence with numerical parameters. The response of the bridge under the circulation of a Talgo 350 train and an ICE 350 are shown. In the former case, the train does not induce resonance on the bridge deck, therefore the response is not dominated by a predominant modal contribution. Also, in this case the numerical model predicts quite accurately the experimental response of the bridge, being experimental amplitudes slightly higher than numerical. In the second case, the train ICE 250 induces a third resonance of the fundamental mode of the structure. The phenomenon is evident both in the experimental response and in the numerical predictions. In this case the FE model overestimates the real response, specially at the resonant frequency. This is consistent with the fact that the effect of wave radiation exerted by the soil and energy dissipation related with vehicle-structure interaction are both especially noticeable at resonant velocities.

Acknowledgements

The authors would like to acknowledge the financial support provided by (i) the Spanish Ministry of Economy and Competitiveness under the research project [BIA2013-43085-P], (ii) the Andalusian Scientific Computing Centre (CICA) and (iii) Universitat Jaume I under the research project [P1-1B2015-54]. This work falls within the initiation of research project BIA2016-75042-C2 financed by the Spanish Ministry of Economy and Competitiveness.

References

- [1] ERRI-D-214/RP9, Rail bridges for speeds > 200km/h. Final report. Part A. Synthesis of the results of D214 research, European Rail Research Institute, 1999.
- [2] L. Frýba, Dynamic behaviour of bridges due to high-speed trains, Workshop Bridges for High-Speed Railways Porto (June 2004) 137–158.
- [3] K. Liu, E. Reynders, G. D. Roeck, G. Lombaert, Experimental and numerical analysis of a composite bridge for high-speed trains, *Journal of Sound and Vibration* 320 (12) (2009) 201 – 220.
- [4] D. Ribeiro, R. Calada, R. Delgado, M. Brehm, V. Zabel, Finite element model updating of a bowstring-arch railway bridge based on experimental modal parameters, *Engineering Structures* 40 (2012) 413 – 435.
- [5] J. Malveiro, D. Ribeiro, R. Calada, R. Delgado, Updating and validation of the dynamic model of a railway viaduct with precast deck, *Structure and Infrastructure Engineering* 10 (11) (2014) 1484–1509.
- [6] C. Rebelo, L. S. da Silva, C. Rigueiro, M. Pircher, Dynamic behaviour of twin single-span ballasted railway viaducts field measurements and modal identification, *Engineering Structures* 30 (9) (2008) 2460 – 2469.
- [7] A. Romero, M. Sols, J. Domnguez, P. Galvín, Soilstructure interaction in resonant railway bridges, *Soil Dynamics and Earthquake Engineering* 47 (2013) 108 – 116.
- [8] M. Iker Kaustell, R. Karoumi, C. Pacoste, Simplified analysis of the dynamic soilstructure interaction of a portal frame railway bridge, *Engineering Structures* 32 (11) (2010) 3692 – 3698.
- [9] P. Mellat, A. Andersson, L. Pettersson, R. Karoumi, Dynamic behaviour of a short span soilsteel composite bridge for high-speed railways field measurements and fe-analysis, *Engineering Structures* 69 (2014) 49 – 61.
- [10] M. D. Martínez-Rodrigo, J. Lavado, P. Museros, Dynamic performance of existing high-speed railway bridges under resonant conditions retrofitted with fluid viscous dampers, *Eng Struct* 32 (3) (2010) 808 – 828.
- [11] CEN/TC250, Eurocode: Basis of structural design. Annex A2: Application for bridges. Final version, European Committee for Standardization, Brussels, 2005.
- [12] CEN, EN 1991-2, Eurocode 1: Actions on Structures - Part 2: Traffic loads on bridges, European Committee for Standardization, Brussels, 2002.
- [13] S. A. Badsar, M. Schevenels, W. Haegeman, G. Degrande, Determination of the material damping ratio in the soil from saw tests using the half-power bandwidth method, *Geophysical Journal International* 182 (3) (2010) 1493–1508.
- [14] C. A. Felippa, Refined finite element analysis of linear and nonlinear two dimensional structures, Ph.D. thesis, University of California, Berkeley (1966).
- [15] E. Reynders, System identification methods for (operational) modal analysis: Review and comparison, *Archives of Computational Methods in Engineering* 19 (1) (2012) 51–124.
- [16] R. J. Allemang, D. L. Brown, Correlation coefficient for modal vector analysis, in: *Proceedings of International Modal Analysis I*, 1983.
- [17] G. Lombaert, G. Degrande, Ground-borne vibration due to static and dynamic axle loads of intercity and high-speed trains, *Journal of Sound and Vibration* 319 (35) (2009) 1036 – 1066.
- [18] A. Doménech, M. Martínez-Rodrigo, A. Romero, P. Galvín, On the basic phenomenon of soil-structure interaction on the free vibration response of beams: Application to railway bridges, *Engineering Structures* 125 (2016) 254 – 265.