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Validation and calibration of a twin-deck bridge model under railway traffic

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Abstract

This article describes the experimental and numerical evaluation of the dynamic behaviour of Cascalheira bridge located on the Northern Line of the Portuguese railway network. The bridge has a short span formed by two filler-beam half-decks each one accommodating a railway track. The study includes the development of a finite element numerical model in ANSYS® software, as well as in situ dynamic characterization tests of the structure, namely an ambient vibration test, for the estimation of natural frequencies, modes shapes and damping coefficients, and a dynamic test under railway traffic, particularly for the passage of Alfa Pendular train. The calibration of the numerical model of the bridge was carried out using an iterative methodology based on a genetic algorithm, allowing an upgrade of the agreement between the numerical and experimental modal parameters. Finally, the validation of the numerical model is performed by comparing the accelerations response of the structure under traffic actions, by means of numerical dynamic analyses considering the vehicle-bridge interaction, with the ones obtained by the dynamic test under traffic actions. The results of the calibrated numerical model showed a better agreement with the experimental results based on the accelerations evaluated in several measurement points located in both half-decks. In the validation process the degradation of the ballast located over the longitudinal joint between half-decks, demonstrated to be relevant for the accuracy and effectiveness of the numerical models.

Keywords: railway bridge; numerical modelling; dynamic testing; model calibration and validation.

1 Introduction

The new operational demands for bridges require an accurate evaluation of the dynamic behaviour of the train-track-bridge system to guarantee the structural safety, train running safety as well as passenger's comfort [1-2]. For this purpose, advanced numerical models are developed for the train, track and bridge subsystems, including their interfaces. The accuracy of these models strongly depends on the experimental calibration and validation of the numerical results, which is usually performed using dynamic measurements based on ambient vibration tests and tests under traffic actions, respectively [3-4].

The most advanced bridge numerical models are typically three-dimensional FE models that include the track [4]. In these models, the ballast layer, sleepers and rail pads, are modelled using volume finite elements, while rails are modelled by beam finite elements. The inclusion of the ballasted track in these models has several advantages, particularly it: i) guarantees an efficient distribution of the trains' axle-loads, ii) acts as a filter removing the high-frequency content from the bridge's dynamic response [5], iii) considers the track-bridge composite effect due to the longitudinal shear stress transmission occurring between rails and bridge deck, through the ballast layer, and iv) considers the damping mechanisms due to energy dissipation on the track components caused by structure-induced movements [6].

In the specific case of twin-deck bridges, few authors [4,7] emphasize the partial continuity effect provided by the track between the adjacent decks and its relevance for understanding their dynamic response. However, given the large number of this type of bridges in the European network, more attention should be given to this topic. Therefore, this paper aims to add new contributes to this regard, namely the development of an advanced methodology capable of characterizing the degradation of the continuous ballast layer over bridges through shear modulus degradation curves [8], particularly in the longitudinal joints between adjacent decks. In these specific zones, the ballast is subjected to cyclic movements induced by rail traffic, which can significantly reduce the interaction effect between adjacent decks.

2 Methods

Train-Track-Bridge model

The Cascalheira bridge (Figure 1) is a 11.10 m length short-span bridge located in the Northern Railway Line in Portugal, that establishes the connection between Lisbon and Porto. The structural solution consists of two composite filler-beam half-decks composed by a reinforced concrete slab with embedded steel girders and separated by a longitudinal joint.

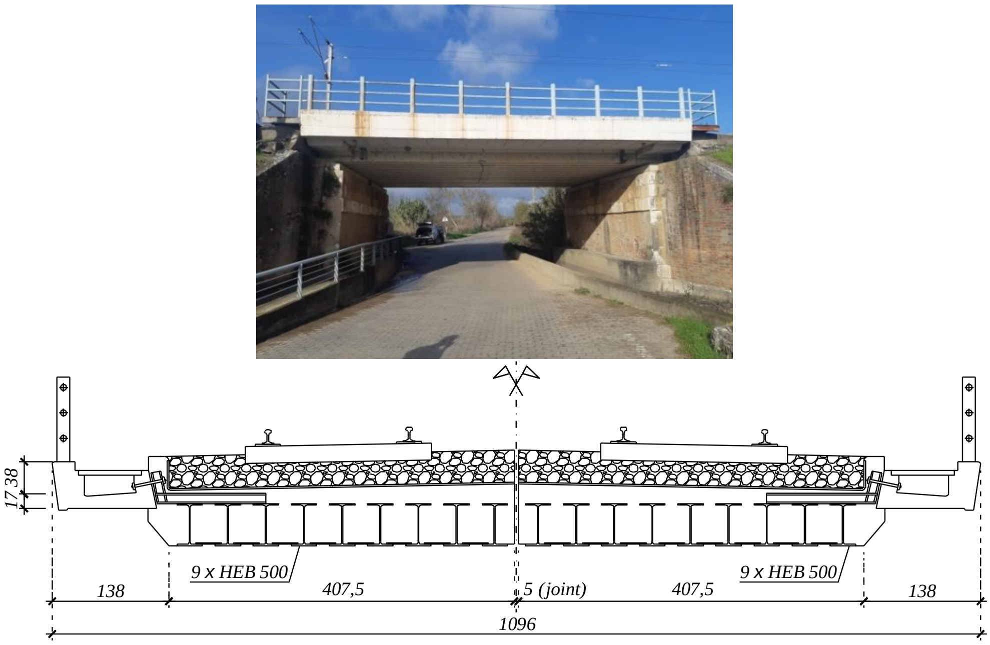


Figure 1: Cascalheira bridge: global view and cross-section.

A 3D model of the bridge (Figure 2), including the track, was developed in the finite element software ANSYS®. To better simulate the transition zone in the abutments, an extension of the track was also modelled. Moreover, different materials were used to model the ballast on the longitudinal and transversal joints to allow the study of the degradation of the track in these regions. The concrete slab was modelled with shell elements, while the steel girders and the rails were modelled with beam elements. Concerning the track, solid elements were used for the ballast, as well as for the sleepers and rail pads. The bearing supports were simulated through spring-dashpots to consider the vertical and longitudinal stiffness of the pot bearings.

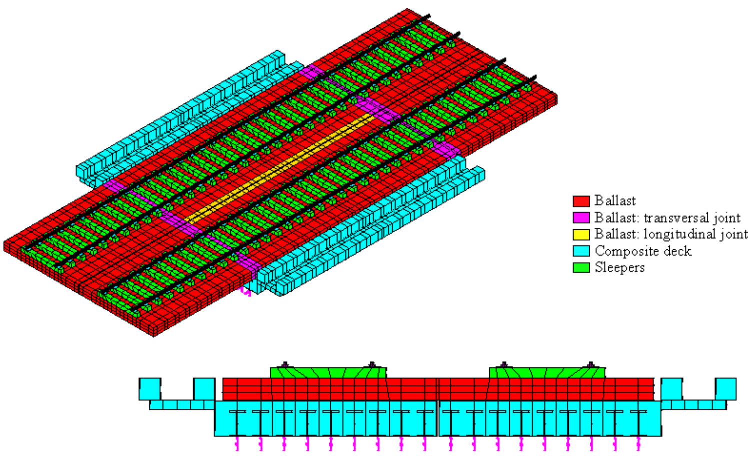


Figure 2: Numerical model of the Cascalheira bridge.

The vehicle considered in this work is the Alfa Pendular train. This conventional type train is composed by 5 cars with a loading scheme characterized by 24 axles with loads varying between 128.8 kN and 136.6 kN. A multibody model of the train has been developed to perform the train-bridge interaction analysis through the tool developed by Montenegro et al. [9].

Experimental tests

An ambient vibration (Figure 3) test was conducted to identify the modal parameters of the bridge, particularly, its natural frequencies and mode shapes, through the Enhanced Frequency Domain Decomposition method. The test setup adopted 12 measurement points, involving the use of accelerometers installed at the lower face of the bridge deck. Moreover, tests under railway traffic were also conducted to measure the dynamic response of the bridge in terms of accelerations caused by the passage of trains.

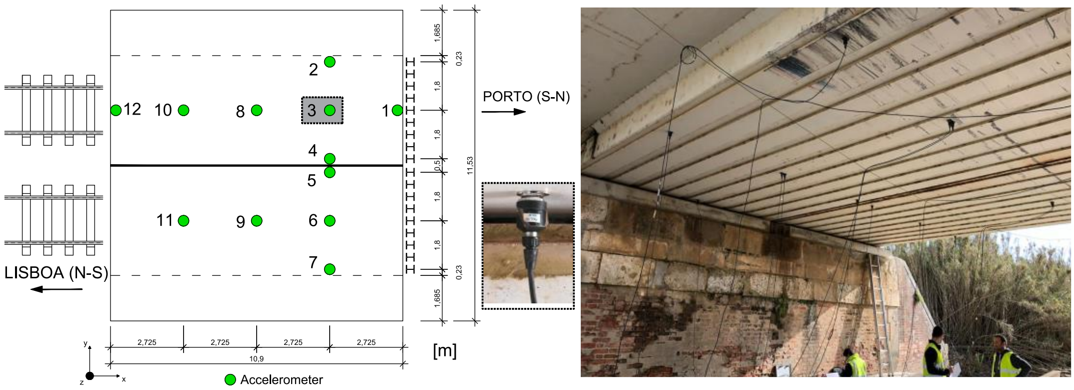


Figure 3: Ambient vibration test.

The calibration of the numerical model of the bridge was performed based on the results of the ambient vibration test and involved an optimization process based on a genetic algorithm. The validation was performed by comparing the vertical acceleration measurements obtained in the tests under railway traffic with the corresponding numerical results obtained with the dynamic analyses.

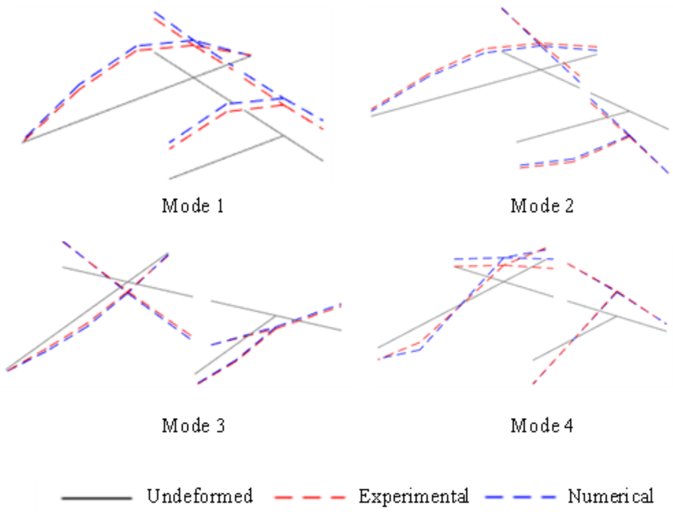
3 Results

Calibration

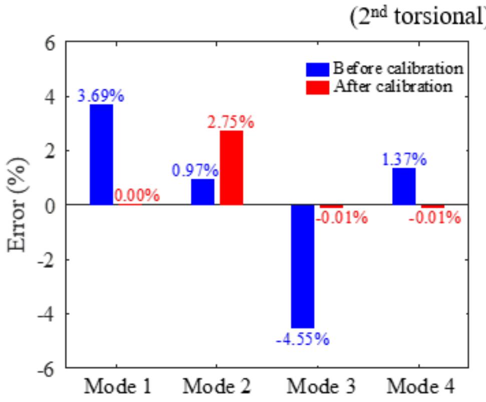
The model optimisation was performed with a genetic algorithm, involving the several bridge material parameters, 4 frequencies and 4 modal configurations. The objective function f_{obj} is expressed as:

$$f_{obj} = a\sum_{i=1}^{4}\frac{\left|f_i^{exp} - f_i^{num}\right|}{f_i^{exp}} + b\sum_{i=1}^{4}\left|MAC\left(\phi_i^{exp}, \phi_i^{num}\right) - 1\right| \tag{1}$$

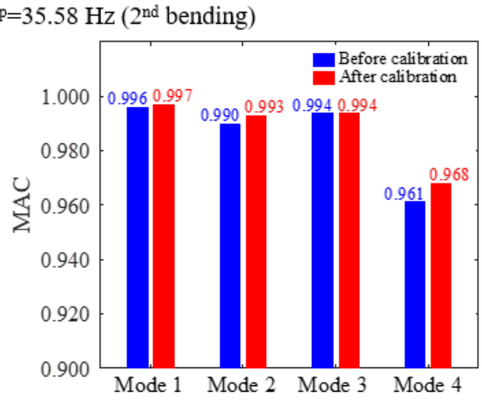
where a and b are weights (=1.0), f_i is the ith experimental frequency, *exp* and *num* indicate experimental and numerical and the MAC is related with the ith modal configurations ϕ_i. Figure 5 shows the comparison between the experimental mode shapes and the corresponding numerical ones after calibration, as well as the errors between the numerical frequencies, before and after the calibration, in relation to the corresponding experimental values. A good agreement is observed, especially after the calibration.



(a) Mode shapes: f_1^{exp}=10.10 Hz (1st bending), f_2^{exp}=12.04Hz (1st torsional), f_3^{exp}=16.59 Hz (2nd torsional), f_4^{exp}=35.58 Hz (2nd bending)



(b) Errors of natural frequencies.



(c) MAC values.

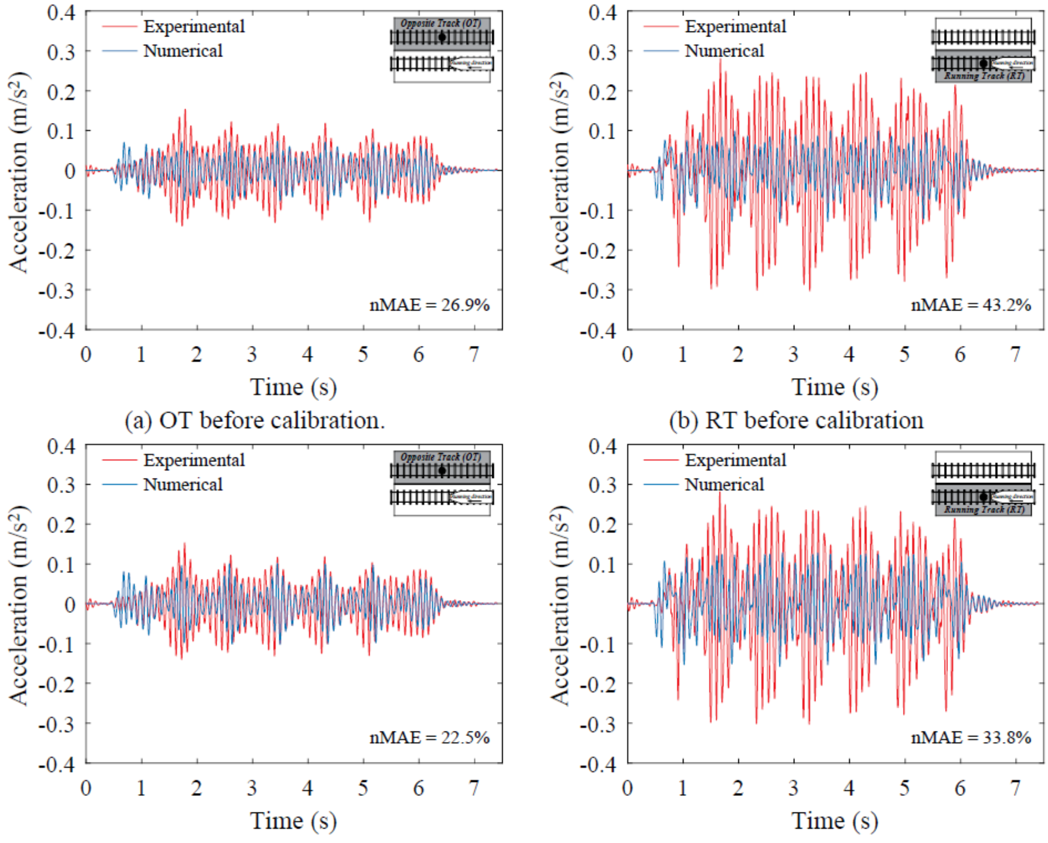
Figure 4: Results of the calibration process.

Validation

The validation was performed by comparing the experimental accelerations at the midspan in the opposite (OT) and running tracks (RT) with the corresponding numerical results. The normalized Mean Absolute Error indicator (nMAE) has also been used to evaluate the quality of the match between the two time-histories (||.|| is the L_1 norm and E and N represent experimental and numerical results):

$$nMAE = \frac{\|N - E\|_1}{\|E\|_1} \qquad (1)$$

Figure 5 presents the comparison between the experimental and numerical accelerations before and after calibration considering a passage of the Alfa Pendular at 110 km/h, where a good match is observed. However, the agreement is not so satisfactory in terms of amplitude (nMAE indicator of 26.9 % and 43.2 % in the OT and RT sides).

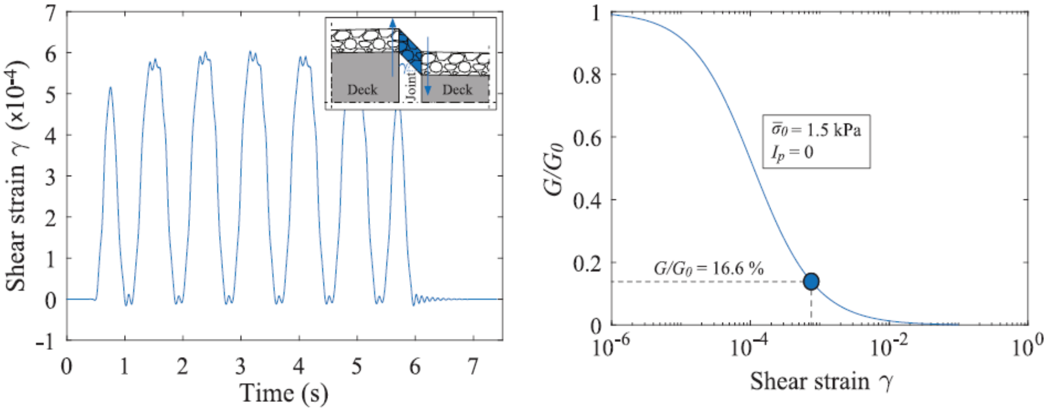


(a) OT before calibration. (b) RT before calibration

(c) OT after calibration (d) RT after calibration

Figure 5: Comparison between experimental and numerical vertical accelerations time-histories before and after calibration.

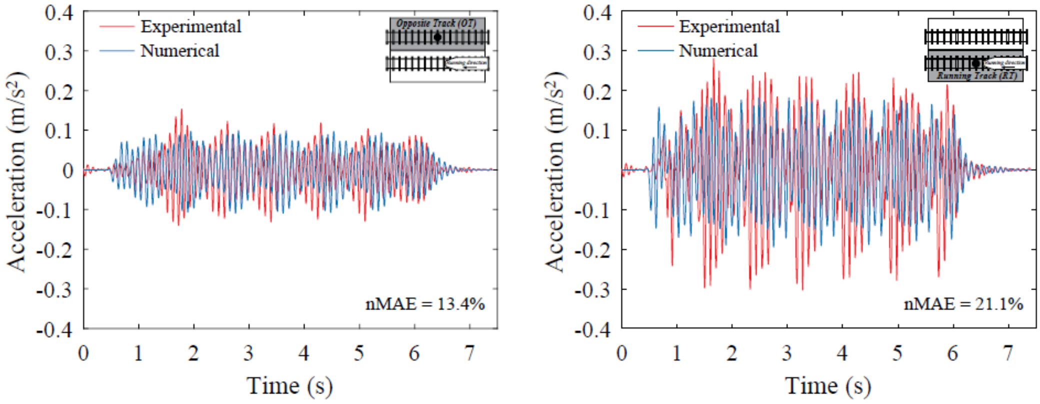
Although structurally separated, the half-decks are slightly connected through the track continuity provided by the ballast. The calibration process led to a decrease in the modulus of elasticity of the ballast over the longitudinal joint, from 145 MPa to 76.25 MPa, demonstrating a possible degradation of the material. This degradation tends to be more pronounced with the shear strain levels experienced in these joints due to the cyclic movements between adjacent half-decks caused by train passages. The numerical shear strains in the longitudinal joint (midspan) due to the passage of the Alfa Pendular and the shear modulus degradation curve [8] are plotted in Figure 6.



(a) OT (b) RT

Figure 6: Effects in the longitudinal joint due to the train passage: (a) time-history of the shear strains and (b) shear modulus degradation.

The maximum shear strain (6.04×10^{-4}) corresponds to a degradation of 16.6 % of the shear (and elasticity) modulus (76.25x0.166=12.66 MPa). An optimal value of 36 MPa has been obtained, which guaranteed a compromise in the agreement between the numerical and experimental data on both track sides. Figure 7 depicts the final results, where a clear improvement is observed (nMAE decreased to 13.4 % and 21.1 % on OT and RT sides).



(a) OT (b) RT

Figure 7: Comparison between experimental and numerical vertical accelerations time-histories after upgrading the properties of the longitudinal joint.

4 Conclusions and Contributions

This study starts with the development of the FE model of the bridge-track system, followed by its calibration with a genetic algorithm. Then, the model is validated through a comparison between the experimental responses of the bridge acquired in a test under railway traffic with those computed numerically with a train-bridge interaction tool. The following conclusions were drawn:

1) An updating procedure based on a genetic algorithm was carried out to calibrate the numerical model, leading to a reduction in the differences between numerical and experimental natural frequencies (average error decreased from 2.65% to

0.69% after calibration) and an increase in the MAC coefficients, reaching values close to 1.0 (between 0.968 and 0.997).

2) Regarding the validation process, a good agreement between experimental and numerical results was observed, in particular after the calibration. The improvement in the results was confirmed through the nMAE indicator, which reduced from 26.9 % to 22.5 % and from 43.2 % to 33.8 % regarding the responses in the OT and RT sides, respectively. However, the amplitudes of the numerical and experimental responses were still considerably different. Therefore, a modification in the FE model of the bridge, namely in the properties of the ballast over the longitudinal joint, was carried out with the objective of getting a closer match between experimental and numerical responses under railway traffic.
3) The effects of a degradation of the ballast over the longitudinal joint were evaluated based on the shear strain levels that occur in this region due to the cyclic movements between the two half-decks. After evaluating the plausible degradation of the ballast layer over the joint through a shear modulus degradation curve, a reduction in its modulus of elasticity was tested. By doing so, the numerical results improved in relation to the experimental ones, leading to lower levels of the nMAE indicator on both sides, more specifically 13.4 % to 21.1 % with respect to the OT and RT sides, respectively.

Acknowledgements

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