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MODEL UPDATING OF A HISTORIC RAILWAY BRIDGE BASED ON GENETIC ALGORITHMS

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Abstract. This article describes the calibration of a finite element (FE) numerical model that simulates the dynamic behaviour of a historic metallic railway bridge that is part of Tâmega Line, which is a decommissioned line of the Portuguese Railways since 2009. The updated numerical model of the bridge reports to its present condition, which will serve as a basis for subsequent studies/projects on the strengthening of the structure, therefore promoting for the sustainability and cost-effectiveness of future interventions in terms of materials and human resources required. The geometry of the numerical model was defined based on data collected from laser geometric surveys and visual inspections, which enabled to identify the existent signs of structural damage and deterioration. With respect to the acquisition of field data concerning the bridge structural response, an ambient vibration test was carried out in order to characterize the natural frequencies and configurations of the global vibration modes. The calibration of the bridge's numerical model relies on the application of an iterative method based on a genetic algorithm. This method involves the resolution of an optimization problem, which requires the minimization of an objective function by varying a set of preselected model parameters. The results obtained with the calibration process have shown a very good agreement between numerical and experimental modal responses and an improvement of the initial numerical model. Moreover, the stability of a significant number of parameters, considering different initial populations, proved the robustness of the genetic algorithm for the optimization of the numerical models.

1 INTRODUCTION

Portuguese railway lines dating from the early 20th century typically include a significant number of bridges and viaducts whose deck consists of a steel lattice structure supported on masonry pillars and abutments [1].

Currently the need for rehabilitation and reuse of existing lines, mainly for tourism purposes, has made urgent the assessment of the structural behavior of these structures, particularly in what concerns their suitability for carrying the rolling stock planned for the intervened sections. This need is particularly relevant given that a significant number of these structures are between 60 and 100 years old and experienced a small number of maintenance/repair works during their service life [1].

Also, the current level of rail traffic, with higher axle loads and higher traffic speeds, also imposes to these structures requirements very different than those established at the time of their design and construction, in particular as regards the quantification of the dynamic effects associated to the passing of regularly spaced axle groups of the trains, whose magnitude can put at risk the structural safety, traffic safety and the passengers comfort. To this end, a reassessment of old steel bridges becomes mandatory in order to ensure that these new requirements are complied.

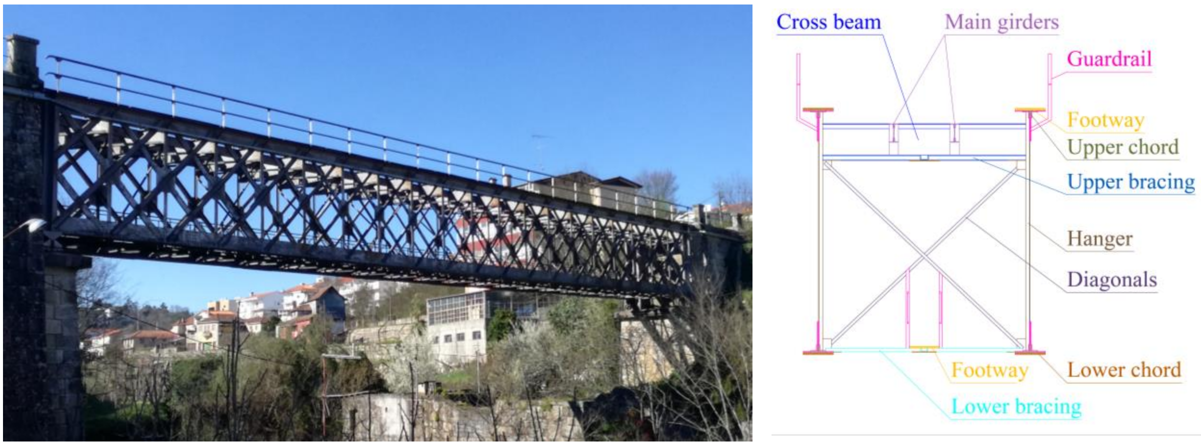
In this context, the development of numerical models based on the finite element method is of particular importance, which in the future can serve as support for more specific studies concerning the evaluation of the structural response under the action of rail traffic. These models are usually calibrated based on information provided from visual inspection campaigns, with the purpose of assessing the current condition of the structures and to identify any anomalies and deficiencies, being complemented by non-destructive tests (static load tests, ambient, free or forced vibration tests, among others) [2-3], thus allowing to reliably reproduce the real behavior of the bridges.

This article focuses on the experimental calibration of a numerical model that simulates the structural response of an old steel bridge, located on the former Tâmega railway line. For this purpose, a FE numerical model was developed to study the dynamic behavior of the bridge, complemented by an ambient vibration test, in order to identify the natural frequencies and configurations of some global vibration modes of the structure. The automatic calibration of the numerical model was performed by adopting an iterative methodology based on genetic algorithms.

2 SÃO LÁZARO RAILWAY BRIDGE

The São Lázaro bridge is located at km +12.7 of Tâmega railway line, near to the city of Amarante, which was inaugurated in 1909 and closed to operation in 2009. The Amarante Municipal Council is currently considering the possibility of rehabilitating the bridge as part of the project to requalify the Livração-Amarante section of the Tâmega railway line, with the purpose of endowing the city with a new accessibility for tourists.

The São Lázaro bridge is a single-section steel bridge with a total length of 40.6 m, which is supported on masonry abutments by means of steel bearings (Figure 1a). The deck structure consists of two truss girders, with a height of 4.0 m, which at the upper level carry a grid formed by main girders and cross beams of I shaped cross-section. The truss girders are formed by two upper and lower chords of T shaped cross-section, joined in their plane through diagonals (U shaped cross-section) and hangers (double angles). The bracing of the lattice girders is provided by means of diagonals, aligned with the truss hangers, and cross bars located in the lower and upper parts of the deck, all made of steel angles. Figure 1b shows the deck cross-section with the main structural elements indicated. The connections between different structural elements are of riveted type.



a) b)

Figure 1: São Lázaro railway bridge: a) general view, b) cross-section.

Given that no technical information about the bridge was made available, either in the form of written or drawn documents, the geometric characterization of the bridge, both in terms of its macro dimensions and sizes of its structural elements, was carried out by means of a geometric survey based on a laser scanning.

3 NUMERICAL MODEL

The three-dimensional FE numerical model was developed using the Autodesk Robot Structural Analysis software [4]. The deck was modeled using bar finite elements applied from a geometric model developed in CAD software (Figure 2a). At one abutment the bearings were modeled as pinned preventing the displacements in the x, y and z directions (Livração side), and as roller ones in the longitudinal direction at the other abutment (Amarante side). The model supports were positioned at the level of their center of rotation and their connection to the truss lower chord was made by means of a rigid bar. The positioning of the geometric center of the sections of the various structural elements was performed using offsets. The connection between the diagonals forming the bracings was executed using gusset plates and suitable internal releases to obtain the most adequate mechanical behavior. The mass of the non-structural elements, such as footways, guardrails, rivets and plates, was applied by means of mass elements, either concentrated or distributed. The total mass of these elements is 16.96 ton, representing about 28% of the total mass of the deck.

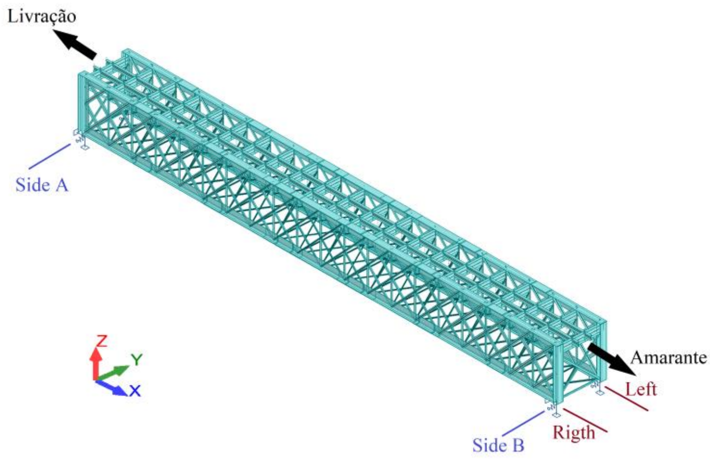


Figure 2: Three-dimensional numerical model of São Lázaro bridge.

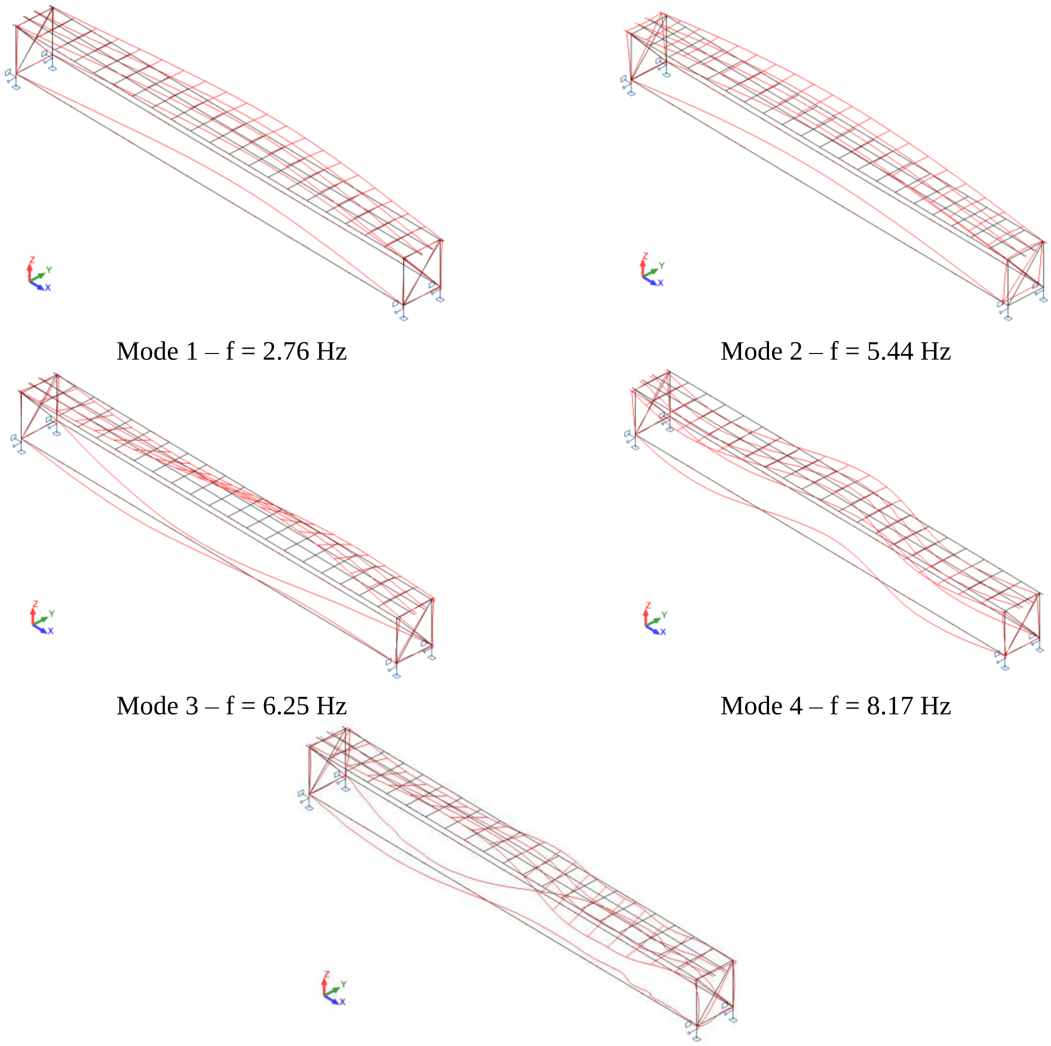
Table 1 presents the most relevant geometrical and mechanical parameters adopted in the numerical model of the bridge, including its designation, the values adopted and the corresponding units. In addition, the lower and upper limits later used in the calibration phase of the numerical model are also pointed out.

Table 1: Geometrical and mechanical parameters of the numerical model.

Parameters		Unit	Adopted value	Limits	
				Lower	Upper
E_s	Young modulus of steel	GPa	210,0	170,0	215,0
ρ_s	Density of steel	kN/m^3	77,0	69,3	92,4
υ_s	Poisson ratio of steel	--	0.3	--	--
$K_{T,1}$	Elastic coefficient in left support - side A	kN/m	20000	100	80000
$K_{T,2}$	Elastic coefficient in right support - side A				
$K_{T,3}$	Elastic coefficient in left support - side B				
$K_{T,4}$	Elastic coefficient in right support - side B				
C_1	Additional mass of the upper and lower guardrails	--	1,00	0,80	1,30
C_2	Additional mass of the footways and gusset plates				
C_3	Additional mass of the riveted elements				

The range of the Young modulus of steel is in line with the results of experimental tests performed on structural steel specimens made by Hess [5] and on specimens taken from bridges integrating the Portuguese railway network built in the 19th century studied by Patrício [1]. In what concerns the density of steel, the adopted range intends to take into account the uncertainties associated with the possibility that the structural steel used in the bridge has a lower percentage of carbon than the steel currently used, and therefore may lead to a higher density. In addition, there is some uncertainty regarding the quantity of rivets used in several structural elements. With regard to the transverse stiffness of the bearings, a very wide range has been considered in order to allow the possibility of having different degradation scenarios for these devices, as they appear to have not undergone maintenance operations since their installation. Finally, regarding non-structural masses, the values initially adopted correspond to a coefficient of variation equal to 1.0, while the range of variation reflects some uncertainties associated with their accounting.

Figure 3 shows the natural frequencies and corresponding configurations for five vibration modes calculated from the numerical model that considered the parameters values presented in Table 1. The modal configurations include essentially transverse (modes 1, 4 and 5) and vertical (modes 2 and 3) bending of the deck. In the model illustrations essentially the upper structural elements of the deck are represented in order to enable a better visual perception of the identified vibration modes.



Mode 1 – f = 2.76 Hz

Mode 2 – f = 5.44 Hz

Mode 3 – f = 6.25 Hz

Mode 4 – f = 8.17 Hz

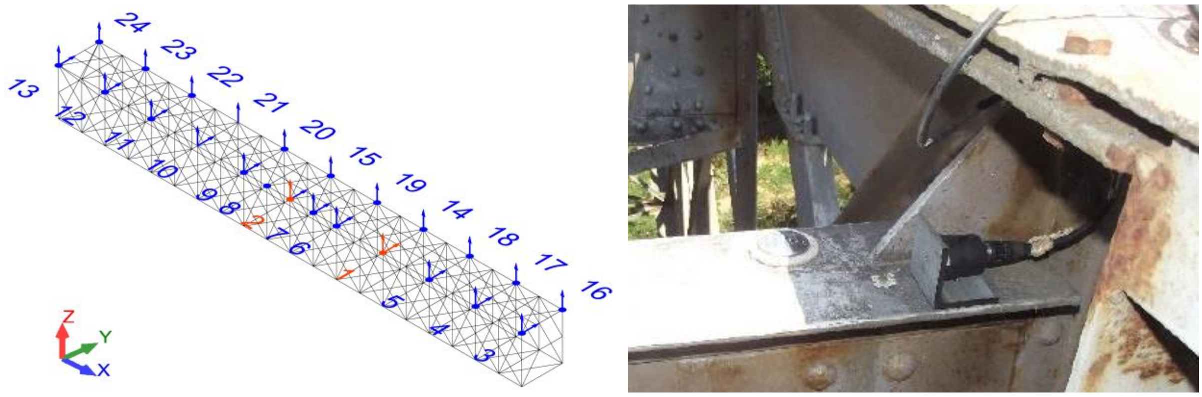
Mode 5 – f = 12.40 Hz

Figure 3: Numerical modal parameters.

4 AMBIENT VIBRATION TEST

The purpose of the ambient vibration test was to identify the bridge's modal properties, particularly the natural frequencies and the corresponding global vibration modes shapes.

The test was carried out using a technique with fixed reference points and movable measuring points, involving the use of 14 high sensitivity piezoelectric accelerometers (model PCB 393B12). The accelerations were measured in the vertical (z) and transverse (y) directions, having been instrumented a total of 24 measuring points positioned at the ends of the cross beams, close to the joints with the upper chords. The position of the accelerometers is shown in Figure 4a, while in Figure 4b a detail of their connection to the structure is presented, which was executed by means of steel angles and plates, coupled with magnets.



a) b)

Figure 4: Ambient vibration test: a) location of the accelerometers; b) detail of the connection between an accelerometer and the joint of the cross beam with the upper chord

The reference accelerometers were located at positions 1 (vertical and transverse directions) and 2 (vertical direction). Data acquisition was performed using National Instruments cDAQ-9172 system with four NI 9234 analog modules for IEPE type accelerometers. The time series were acquired with an approximate duration of 8 min and a sampling frequency of 1000 Hz, later decimated for a frequency equal to 500 Hz.

The identification of the modal parameters was performed by applying the improved version of the frequency domain decomposition method (EFDD) using the ARTeMIS software [6]. Figure 5 presents the mean and normalized singular values of the spectra matrix of all experimental configurations performed in the test, obtained by applying the EFDD method. Five vibration modes were identified in correspondence with the 5 peaks marked on the curve of the first singular value.

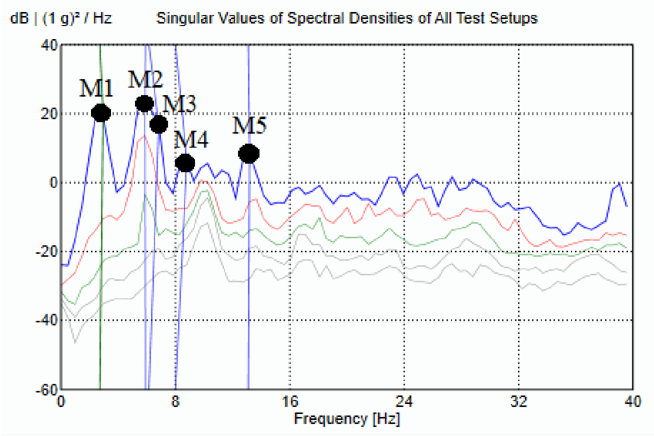
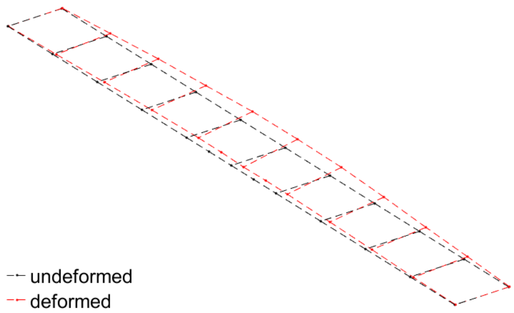
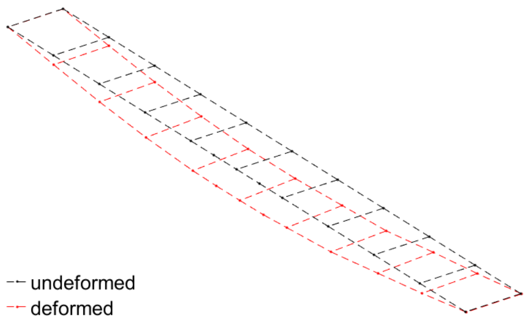


Figure 5: EFDD method: modal identification.

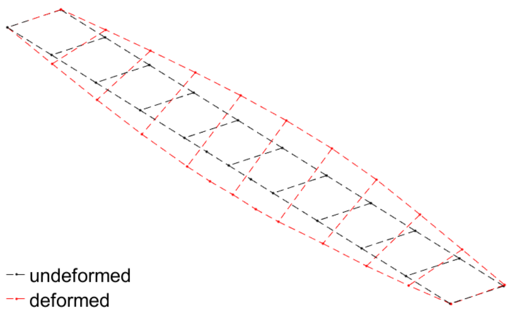
In Figure 6 the average values of the vibration frequencies and the corresponding modal configurations are shown. The analysis of these mode shapes allows identifying movements associated to transverse and vertical bending with good definition. Modes 1 and 4 mostly involve transverse bending of the deck, mode 2 essentially the vertical bending, whereas modes 3 and 5 mainly include torsion movements.



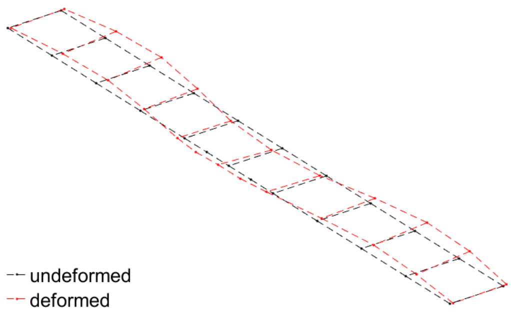
Mode 1 – f = 2.71 Hz



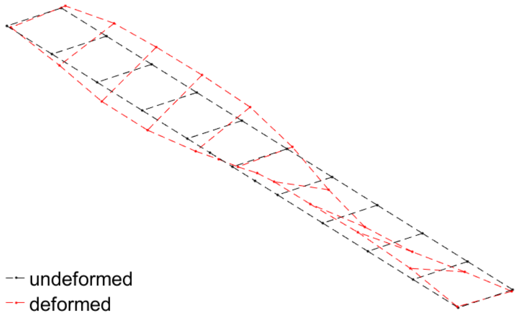
Mode 2 – f = 5.90 Hz



Mode 3 – f = 6.14 Hz



Mode 4 – f = 8.01 Hz



Mode 5 – f = 13.09 Hz

Figure 6: Experimental modal parameters.

5 CALIBRATION

The calibration of the bridge numerical model was performed based on the results of the ambient vibration test and involved the execution at first of a sensitivity analysis and afterwards of an optimization procedure. The iterative process considered in the automatic calibration of the numerical model included a genetic algorithm that is described in detail in reference [7].

5.1 Sensitivity analysis

The sensitivity analysis aimed at identifying the numerical parameters that present a higher impact on the modal responses and should be included in the succeeding optimization phase.

Figure 7 presents the results of the sensitivity analysis using a Spearman' correlation coefficients matrix [8]. The sensitivity analysis was performed using a stochastic sampling technique based on 500 samples generated by the Latin Hypercube method. The correlation coefficients in the range [-0.30; +0.30] were excluded from the graphical representation.

The correlation matrix shows that the Young modulus of the steel (E_s), its density (ρ_s), the transverse stiffness of the bridge deck supports ($K_{T,1}$, $K_{T,2}$, $K_{T,3}$, $K_{T,4}$), and the additional mass coefficient (C_2) of the lower and upper technical footways and gusset plates are the parameters that most influence the modal responses.

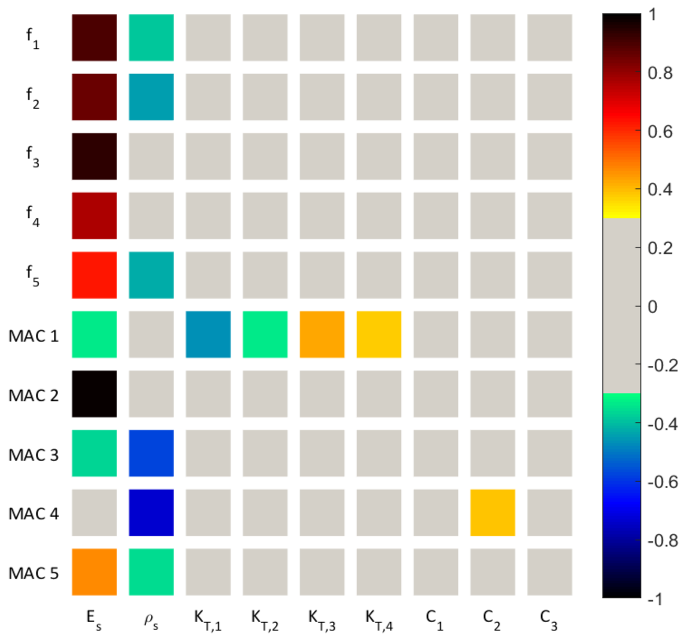


Figure 7: Spearman correlation matrix.

5.2 Optimization

The optimization phase aimed at estimating the values of the numerical parameters that minimize the differences between the numerical and experimental modal quantities, which involved the definition of an objective function and the application of an optimization technique based on a genetic algorithm.

The objective function (*f*) comprises two terms, one related with the residuals of the vibration frequencies and the other related with the residuals of MAC values:

$$f = a\sum_{i=1}^{5}\frac{\left|f_i^{exp} - f_i^{num}\right|}{f_i^{exp}} + b\sum_{i=1}^{5}\left|MAC\left(\varnothing_i^{exp}, \varnothing_i^{num}\right) - 1\right| \quad (1)$$

where f_i^{exp} and f_i^{num} are the experimental and numerical frequencies for mode *i*, ϕ_i^{exp} and ϕ_i^{num} are the vectors containing the experimental and numerical modal information regarding mode shape *i*, whereas *a* and *b* are weighting factors of the objective function terms, assumed in the present case study equal to 3.0 and 1.0, respectively.

The optimization of the model involved 7 numerical parameters and 10 modal results. The genetic algorithm took into consideration an initial population of 30 individuals for 100 generations, in a total of 3000 individuals. The initial population was randomly generated by Latin Hypercube method. In this algorithm the number of elites was defined as 1, the replacement rate set as 5% and the crossing rate as 50%.

Figure 8 presents the ratios of the values of each numerical parameter with respect to the limits given in Table 1 for the independent optimization runs GA1 to GA4. A ratio of 0% means that the parameter matches with the lower limit and a ratio of 100% means that it matches with the upper limit.

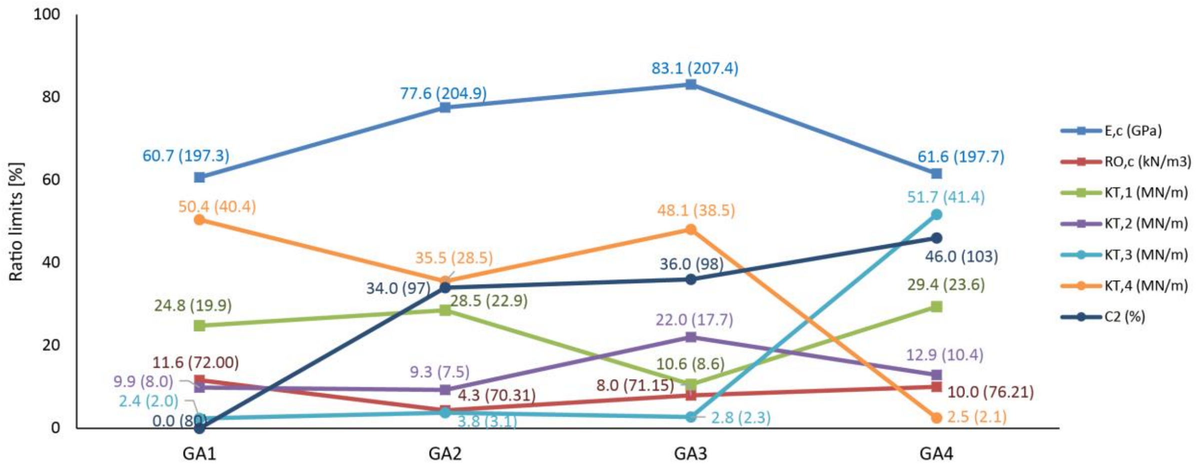


Figure 8: Values of the numerical parameters obtained for optimization cases GA1 to GA4.

The results show that the most sensitive parameters, the Young modulus (E_s) and the density (ρ_s) of steel, are those presenting estimates with lowest variability, close to 6% and 23%, respectively. The non-structural mass coefficient (C_2), which is the parameter with the lowest influence in the response, presents estimates with very significant variations, close to 45%.

With regard to the stiffness values of the side A ($K_{T,1}$ and $K_{T,2}$) and side B ($K_{T,3}$ and $K_{T,4}$) supports, the estimates show variations with opposite trend, i.e., a stiffness increase in the left bearing is usually coupled with a decrease in the stiffness of the right bearing and vice versa. This trend is more evident in bearings at side B supports compared to those at side A. This may be related to the fact that there are different combinations of these sets of parameters that lead to the same solution in terms of optimization problem.

Figure 9a shows the values of the experimental and numerical natural frequencies, before and after calibration, including the error values between the numerical and experimental values of these parameters, taking as reference the experimental ones. The numerical results after calibration concern the optimization case GA2, for which the objective function presented the smallest residue. Figure 9b shows the MAC parameter values before and after calibration.

The mean error of the frequencies decreased from 9.4% before calibration to 2.5% after calibration, whereas the mean value of the MAC parameters remained virtually the same, with a value of 0.962.

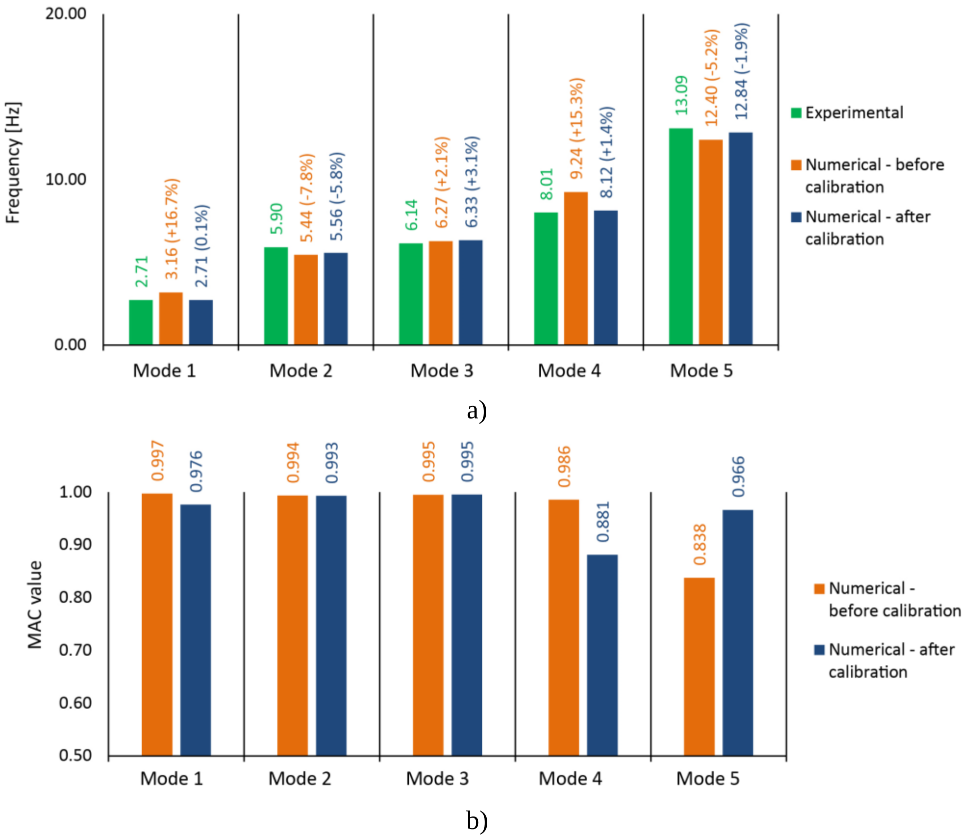


Figure 9: Correlation analysis of experimental and numerical modal parameters: a) natural frequencies. b) MAC.

6 CONCLUSIONS

This article focused on the experimental calibration of a numerical model for simulating the structural response of an old steel railway bridge. The three-dimensional FE model included the modeling of the steel deck and its support on the abutments through the bearing devices. The ambient vibration test allowed identifying the natural frequencies associated with five global vibration modes involving bending and torsion movements of the deck between 2.71 Hz and 13.09 Hz.

The results of the numerical model optimization of the bridge have presented a very good approximation with the experimental results and a significant improvement over the reference results before the calibration. On the other hand, the genetic algorithm enabled to obtain sufficiently stable estimates of a significant number of parameters, considering different initial populations, proving its efficiency and robustness. The analysis of the numerical parameters values after the calibration has allowed to verify that: i) the values of the four bearings transverse stiffness have a distinct magnitude, probably pointing different states of degradation; ii) the values of the steel Young modulus were in the range of 197.3 GPa and 207.4 GPa, which may point to a steel used in the bridge construction with improved strength characteristics compared to other steels used at the time of the bridge construction.

As future developments it is important to highlight the evaluation of the dynamic behavior of the bridge under light traffic action, based on numerical analyzes that contemplate the dynamic interaction of the bridge-train system, with the purpose of assessing structural safety, traffic safety and passenger comfort.

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