Chapter 6
Calibration of Dynamic Models of Railway Bridges Based on Genetic Algorithms

Diogo Ribeiro, Rui Calçada and Raimundo Delgado

Abstract This chapter presents the main experimental calibration methodologies of finite element numerical models, with particular focus on methodologies based on modal parameters. In this context, the computational implementation of an iterative method based on a genetic algorithm is described. The iterative method involves the resolution of an optimization problem, which involves the minimization of an objective function by varying a set of preselected model parameters. The objective function includes residuals associated to natural frequencies and mode shapes. The proposed methodology is applied to the calibration of the dynamic models of two railway bridges, São Lourenço bridge and Alverca viaduct, both located in the northern line of the Portuguese railways in recently upgraded track sections. The calibration results demonstrate a very good agreement between numerical and experimental modal responses and a significant improvement of the numerical models before calibration. Also the stability of a significant number of parameters, considering different initial populations, proved the robustness of the genetic algorithm in the scope of the optimization of the numerical models. The updated numerical models were validated based on dynamic tests under railway traffic. The results showed an excellent agreement between numerical and experimental responses in terms of displacements and accelerations of the bridges' decks.

Keywords Railway bridges · Numerical models · Modal parameters · Experimental calibration · Genetic algorithm · Validation

D. Ribeiro (✉)
School of Engineering, Polytechnic of Porto, Porto, Portugal
e-mail: drr@isep.ipp.pt

R. Calçada · R. Delgado
Faculty of Engineering, University of Porto, Porto, Portugal
e-mail: ruiabc@fe.up.pt

R. Delgado
e-mail: rdelgado@fe.up.pt

J. Magalhães-Mendes and D. Greiner (eds.), *Evolutionary Algorithms and Metaheuristics in Civil Engineering and Construction Management*,
Computational Methods in Applied Sciences 39, DOI 10.1007/978-3-319-20406-2_6

6.1 Introduction

Railway bridges are structures subjected to high intensity moving loads, where the dynamic effects can reach significant values. At present, these effects are being given greater importance due to the increase of the circulation speed, not only in conventional lines but also in new lines, such as the case of high speed railway lines.

In structures with complex behaviour the evaluation of these effects is performed by means of dynamic analyses using finite element (FE) models. The process of developing a FE model of a structure involves assumptions and simplifications that may cause errors. These errors are usually related to the inaccuracy in the FE model discretization, uncertainties in geometry and boundary conditions and variation in the material properties.

Therefore, the accuracy of the FE model strongly depends on the experimental validation of the numerical results that is usually performed by means of static or quasi-static measurements based on load tests [11], dynamic measurements based on ambient vibration or forced vibration tests [8, 13], or a combination of static and dynamic measurements [25]. In recent years, in situ dynamic testing has been used and reported by several authors [12, 28] in the scope of the identification of the modal parameters of structures, namely the natural frequencies and mode shapes. Experimental modal data is also perturbed by measurement errors typically related with the environmental variability (such as temperature and wind), the variability in operational conditions during the measurements (e.g. traffic) and errors with measured signals and post-processing techniques [6, 31]. Despite the presence of the referred errors it is generally assumed that the experimental data is a better representation of the structural behaviour than the initial estimations from the FE model [9].

Finite element model updating, also known as calibration of a finite element model, is a procedure to determine uncertain parameters in the initial model based on experimental results to achieve a more suitable updated model of the structure [10]. Updated models can be used for the prediction of dynamic responses under new load scenarios, for damage identification, to design health monitoring systems, as well as for improved remaining lifetime predictions [2, 12]. There are basically two distinct finite element model updating methodologies in structural dynamics: the direct [29] and the iterative methods [27, 33].

The direct methods directly update the elements of the stiffness and mass matrices in a one-step procedure. In this method the experimental modal properties can be exactly represented by the updated system matrices. Unfortunately, the updated system matrices have little physical meaning, and cannot be related to physical properties of the finite element model [10]. This approach can also lead to non-sparse and non-positive definite system matrices, where the connectivity relations between the different structural elements, in which the structure was discretized, can be disregarded.

The iterative methods are typically related to a penalty function, which is improved by a step-by-step approach. The penalty function denotes the objective function based on the discrepancy between numerically obtained and experimentally derived features, such as natural frequencies or modal deflections. This approach is more flexible in its application as the uncertain physical properties of the FE model, typically material properties or geometrical dimensions can be updated.

Brehm [2] distinguished the solving algorithms for model updating in sensitivity-based methods and optimization-based methods. The sensitivity-based methods, applied in the works developed by Friswell and Mottershead [10], Teughels [26], Jaishi and Ren [13], Huang et al. [12], among others, depends on the update of the sensitivity matrices to proceed to the next iteration. The sensitivity matrices, also referred as Jacobian matrices, contain the first derivatives of each residue of the objective function with respect to the parameters of the numerical model. Brehm [2] reports that the success of this approach requires that the initial numerical parameters values are close to the optimal solution, and also its variation during the optimization process occurs in reduced intervals. Furthermore the selection of a large number of numerical parameters, and the coexistence of parameters with high and low sensitivity in relation to the responses, can cause numerical errors due to poor conditioning of the sensitivity matrix [4, 26]. Teughels [26] also refers that in this approach the number of responses must be equal or greater than the number of numerical parameters in order to avoid numerical errors. Some of the refereed limitations may be minimized by performing modifications to the gradient-based algorithm [26, 27] or by applying regularization techniques [10].

The optimization-based methods are generally more flexible since it does not require the calculation of sensitivities and are particularly adequate to situations where there are uncertainties in the objective function, or objective functions with multiple local minima. Applications of such methods in the scope of model updating of railway bridges were referred by Chellini and Salvatore [7], Liu et al. [14] and Cantieni et al. [5]. Concerning the optimization algorithm, several methods are available to solve the optimization problem. These include gradient-based methods (quasi-Newton, sequential quadratic programming, augmented Lagrangian, etc.) [26], response surface methods [21] and nature inspired algorithms (e.g., genetic algorithm, evolutionary strategies, particle swarm optimization) [30]. The genetic algorithm, used in the present work, is not a regularly reported and referenced methodology in the scope of model updating, particularly in the field of model updating of bridges based on experimental vibration data. In this specific research topic, the research of Cantieni et al. [5] and Zabel and Brehm [31] should be emphasized. Genetic algorithms have recognized advantages such as the non-dependence of the initial starting point, capability to manage a large number of parameters and constraints, possibility to handle with discrete and binary variables, ability to find the global minimum in functions with several local minima and the possibility to accept failed designs. On the other hand, a low convergence rate in comparison to gradient-based methods is generally agreed to be its main disadvantage.

More recently Brehm [2] states that the success of the numerical parameters estimate largely depends on the reliability of the experimental data and numerical responses. This author considers that the uncertainties associated with the experimental and numerical responses estimates, can be included in the optimization problem based on statistical parameters related with coefficients of variation and associated confidence intervals. The proposed methodology, called stochastic model updating, are also referenced in the works developed by Mares et al. [16], Mottershead et al. [19] and Zabel and Brehm [32].

This chapter describes an iterative methodology for the calibration of numerical models based on genetic algorithms. This methodology is applied to the calibration of the dynamic models of two railway bridges located on the northern line of the Portuguese railways, which establishes the connection between the cities of Lisbon and Porto, in recently upgraded track sections. The calibration results of the numerical models of both bridges show a significant improvement in relation to the initial models demonstrating the efficiency and robustness of the implemented technique and particularly the genetic algorithms. The comparison of the experimental and numerical dynamic responses after calibration, in terms of displacements and accelerations on the bridges' decks for the passage of Alfa Pendular tilting train showed an excellent agreement, as well as an important improvement in relation to the initial numerical models.

6.2 Computational Implementation of an Iterative Method

The computational implementation of an iterative method based on a genetic algorithm involved the use of three software packages: Ansys [1], Matlab [17] and OptiSlang [20]. Figure 6.1 shows a flowchart that illustrates the computational implementation of the method, indicating the softwares involved in the different phases.

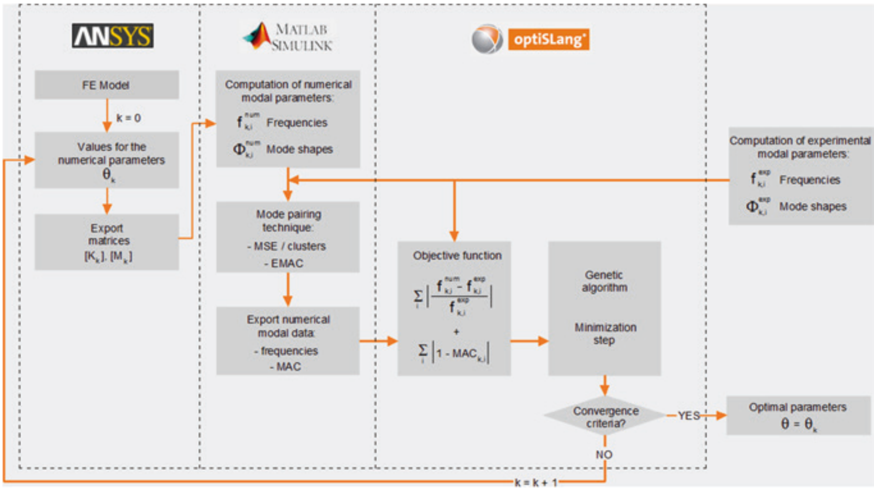

Fig. 6.1 Iterative calibration methodology based on a genetic algorithm

In ANSYS environment the FE numerical model is developed based on a set of initial parameter values θ_1, θ_2, …, θ_k, where k is the number of individuals in each generation, and the mass and stiffness matrices are extracted. The pre-selection of the calibration parameters is performed based on global sensitivity analysis [22]. The sets of parameter values of generation 1 are randomly generated in OptiSlang software by applying the Latin Hypercube method. The export of mass and stiffness matrices is performed through text files in Harwell-Boeing format, suitable for storing sparse matrices.

In Matlab software, the eigenvalues and eigenvectors problem is solved, and based on the experimental modal information, the mode pairing between numerical and experimental modes using a modal strain energy criterion based on EMAC parameter is performed [3]. The values of the natural frequencies and the corresponding MAC (Modal Assurance Criterion) values are exported in text format.

Finally, the OptiSlang software, based on an objective function and on the application of an optimization technique supported by a genetic algorithm, estimates a new set of parameters focused on the minimization of the objective function residuals. The objective function includes two terms, one relative to the residuals of the frequencies of vibration and other related to the residuals of modal configurations. This procedure is repeated iteratively until the maximum number of generations is reached.

6.3 São Lourenço Bridge

6.3.1 Description

São Lourenço railway bridge is located at km +158.662 of the northern line of the Portuguese railways. The bridge is a bowstring arch consisting of two half-decks with 42 m span, each one carrying a single track. Each deck consists of a 0.40 m thick prestressed concrete slab suspended by two longitudinal arches. The suspension is performed by means of metallic hangers and diagonals. The arches are linked in the upper part by transversal girders that assure the bracing of the arches.

The deck is supported at each abutment by two pot bearings. The distance between the supports is 38.4 m, and the extremities of the deck slab work as cantilevers with 1.8 m span. Each half-deck cross section, with a total width of 7.35 m, consists of a concrete slab laterally supported by two main girders, forming a U-section, and a side footway. In Fig. 6.2 a lateral view of São Lourenço bridge and a cross section of the deck are presented.

6.3.2 Numerical Model

The dynamic analysis of São Lourenço railway bridge was performed using a three-dimensional model, including the track, developed in Ansys software. A global view of the numerical model of the bridge is presented in Fig. 6.3.

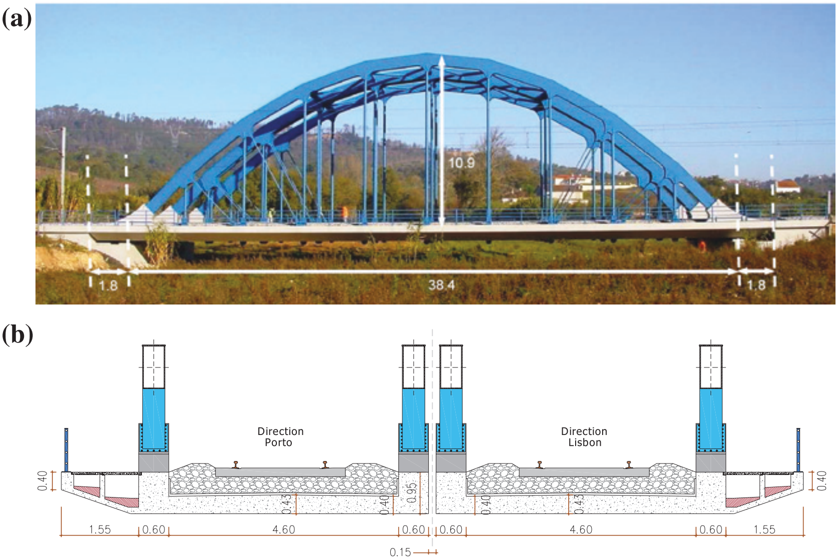

Fig. 6.2 São Lourenço bridge: **a** lateral view; **b** cross-section of the deck

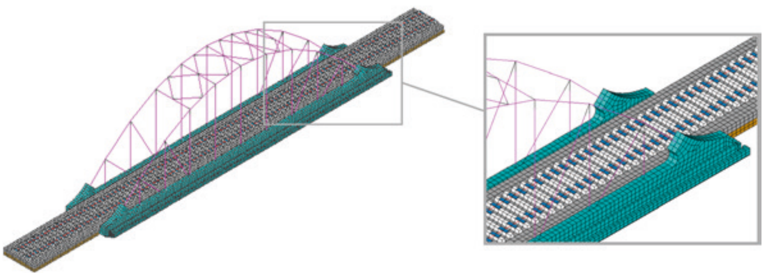

Fig. 6.3 Numerical model of São Lourenço bridge including the track

The deck slab was modelled with solid elements. The arches, hangers, diagonals and bracings were modelled with beam elements. The track was modelled in an extension corresponding to the bridge length and in a distance of about 10 m from each abutment, in order to simulate the support of the track on the adjacent embankments. The rails were modelled by beam elements levelled with the center of gravity axis, and the sleepers and the ballast layer were modelled using solid finite elements. The connections related to the support bearings were located at their centers of rotation. To correctly reproduce the deformability length of the hangers and diagonals, rigid elements were introduced in the extremities of the beam elements. The structure was divided into 26,754 nodes and 80,029 degrees-of-freedom.

Table 6.1 describes the main geometric and mechanical parameters of the numerical model of the bridge, including its designation, the adopted value and the respective unit. Additionally, the statistical properties of some of the parameters

Table 6.1 Characterization of the main parameters of the numerical model of São Lourenço bridge

Parameter	Designation	Statistical properties		Limits (lower/upper)	Adopted value	Unit
		Distribution type	Mean value/standard deviation			
E_c	Modulus deformability concrete	Normal	38.7/3.87	31.0/46.4	38.7	GPa
ρ_c	Density concrete	Normal	2446.5/97.9	2286/2607	2446.5	kg/m^3
E_{bal}	Modulus deformability ballast	–	–/–	–/–	130	MPa
ρ_{bal}	Density ballast	Uniform	1885/147.2	1630/2140	1733	kg/m^3
E_s	Modulus deformability steel	Normal	202/8.1	188.7/215.3	202	GPa
ρ_s	Density steel	–	–/–	–/–	7850	kg/m^3
K_v	Vertical stiffness of supports	Log-normal	7419/6929	–/–	3847	MN/m

that will be used later in the model calibration phase are listed. The lower and upper limits of the normal statistical distributions were obtained by subtracting or adding to the average value, a value equal to two times the standard deviation.

6.3.3 Calibration

The calibration of the bridge numerical model involved the use of 5 design variables and 24 modal responses (12 frequencies and 12 MAC values) related to global vibration modes of the deck and arches. The experimental modal parameters were obtained from an ambient vibration test described in Ribeiro et al. [23].

The genetic algorithm was based on an initial population consisting of 30 individuals and 150 generations, for a total of 4500 individuals. The initial population was randomly generated by Latin Hypercube method. In this algorithm the number of elites was equal to 1 and the number of substitute individuals was also defined equal to 1. The crossing rate was considered equal to 50 % and the mutation rate was set equal to 10 % with a standard deviation, variable along the optimization, between 0.10 and 0.01.

The optimal values of the parameters were obtained based on the results of 4 independent optimization runs (GA1 to GA4) with different initial populations. In Fig. 6.4 are represented the ratios of the values of the main numerical parameter relative to the limits indicated in Table 6.1 for optimization runs GA1 to GA4. A ratio of 0 % means that the parameter coincides with the lower limit. A ratio of

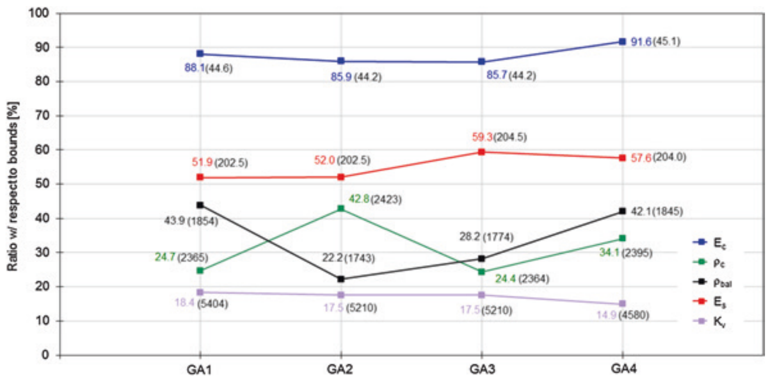

Fig. 6.4 Values of the numerical parameters for the optimization runs GA1 to GA4

100 % means that it coincides with the upper limit. The values of the numerical parameters are indicated in brackets.

The optimum value of the module of deformability of concrete situated in the range between 44 and 45 GPa, the value of the module of deformability of steel was set between 202.5 and 204.5 GPa and the value of the vertical stiffness of the supports was in the range of 4600 and 5600 MN/m. These parameters, which influence more the numeric responses, show variations always below 10 %. For the densities of the concrete and ballast, the estimates show slightly higher variations, close to 20 %. This should be related to the fact that these parameters contribute similarly to the mass of the deck and therefore it exist different combinations of these parameters that lead to the same solution in terms of the optimization problem.

Figure 6.5 presents a comparison between the experimental and numerical after calibration modal configurations of the bridge. To simplify the graphical

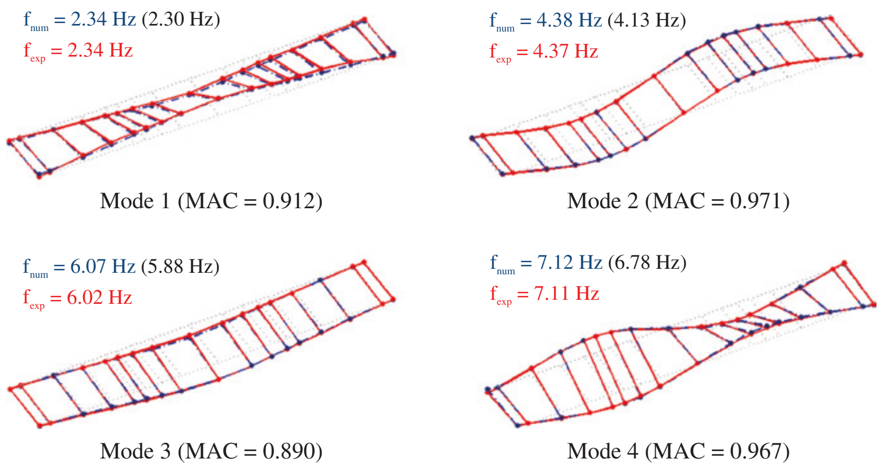

Fig. 6.5 Comparison between the experimental and numerical modal parameters

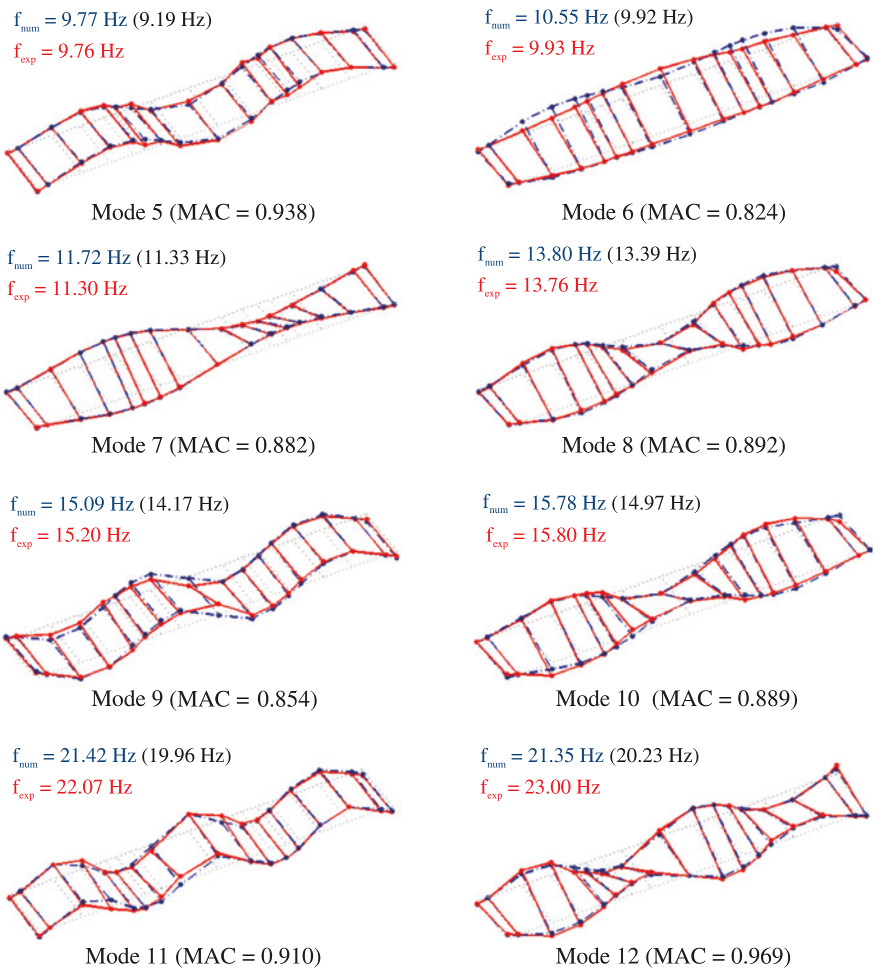

Fig. 6.5 (continued)

representation only points belonging to the deck are presented. The modal configurations are related to transversal bending modes of the arches (1, 4, and 8) and bending modes (2, 3, 5, 9 and 11) and torsion modes of the deck (6, 7, 10 and 12). The results after calibration refer to the optimization run GA1, which is associated with the lowest residual of the objective function, equal to 1.327. In the same Figure the values of the numerical frequencies after calibration (f_{num}), experimental frequencies (f_{exp}) and MAC values, are also shown. In brackets the values of the numerical frequencies before calibration, which resulted from the modal problem resolution based on the adopted values of parameters listed in Table 6.1, are also indicated.

The calibration results showed a very good agreement between numerical and experimental modal responses and a significant improvement in relation to the numerical model before calibration. The average error of the frequencies decreased from 4.7 %, before calibration, to 1.9 % after calibration. The average MAC value increased from 0.880, before calibration, to 0.908 after calibration.

6.3.4 Validation

The validation of the numerical model was performed based on a dynamic test under railway traffic which allowed evaluating the dynamic response in terms of displacements and accelerations at several locations of the bridge deck [23].

Figure 6.6 presents some details of the instrumentation used, consisting of LVDTs for measuring the displacement in one of the supports (Fig. 6.6a) and in a section between 1/3 and 1/4 span of the deck (Fig. 6.6b), and a piezoelectric accelerometer (Fig. 6.6c) positioned in the same section of the deck.

The numerical responses were obtained based on a dynamic analysis considering train-bridge interaction, including measured track irregularities, performed by TBI software [22]. In problems with train-bridge interaction, TBI software uses the modal superposition method for solving the dynamic problem of the bridge, and the Newmark method, for solving the dynamic problem of the train. The numerical model of Alfa Pendular train was calibrated based on experimental modal parameters, as described in detail in Ribeiro et al. [24]. The contribution of 85 vibration modes for the response of the bridge, with frequencies between 2.34 and 30 Hz, was considered. The time step of the analysis was equal to 0.001 s. The adopted values of the damping coefficients were equal to the average values of the coefficients obtained from an ambient vibration test [23].

Figure 6.7 compares the dynamic responses of the bridge obtained by experimental and numerical calibration, before and after updating, for the passage of

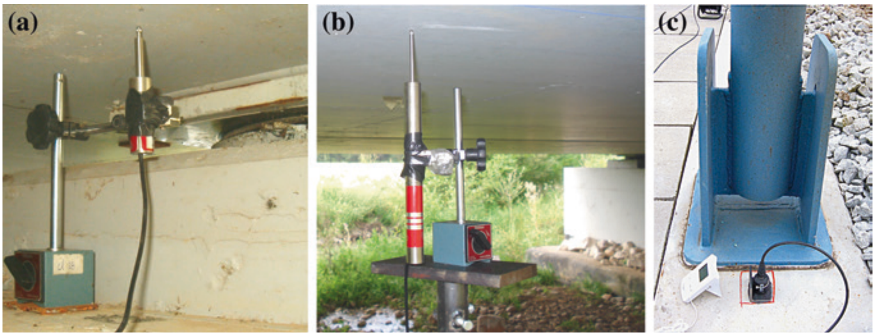

Fig. 6.6 Dynamic test under railway traffic: **a** LVDT nearby the support; **b** LVDT and **c** accelerometer, both on the main girder of the deck

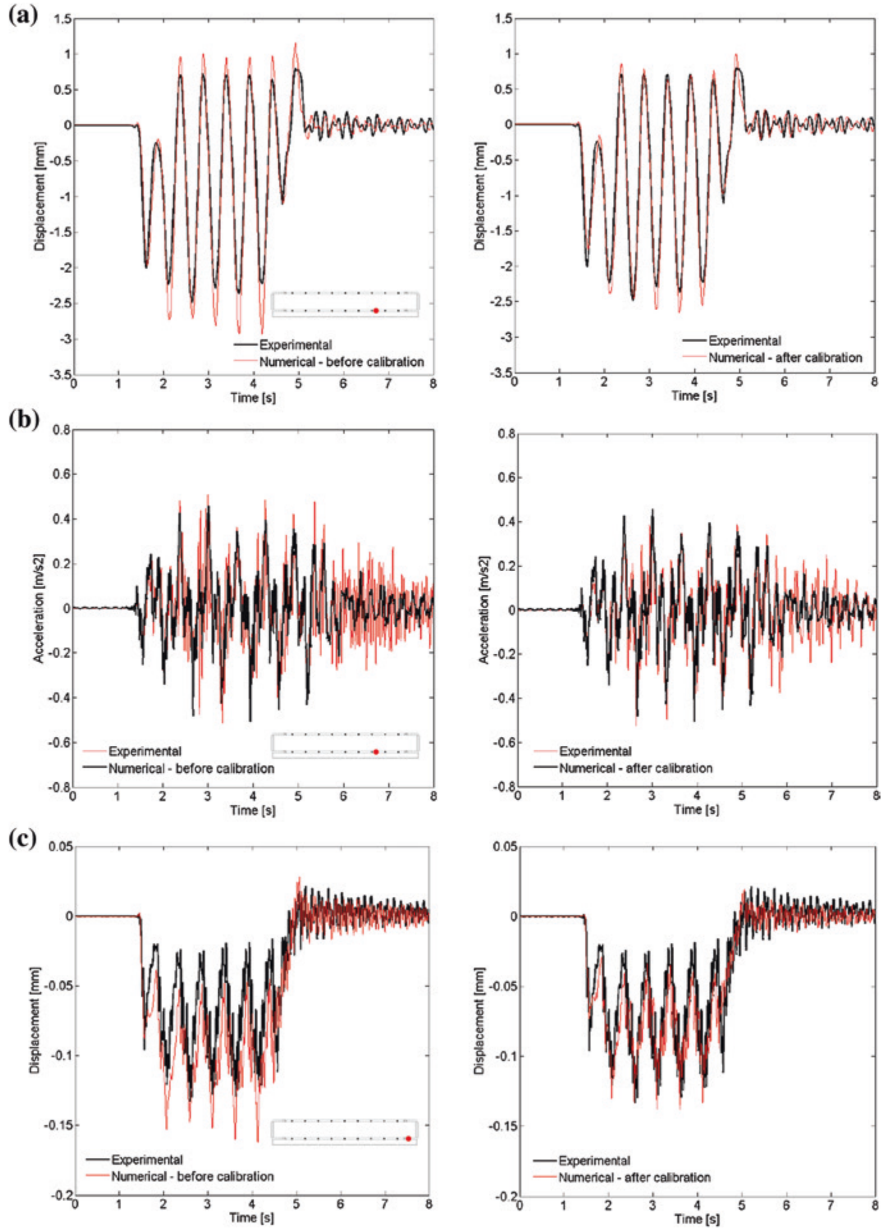

Fig. 6.7 Comparison of the experimental and numerical, before and after updating, dynamic responses of the bridge for the passage of Alfa Pendular train at a speed of 180 km/h: **a** displacements and **b** accelerations in a section between 1/3 and 1/4 span of the deck; **c** displacements at the support

Alfa Pendular train at a speed of 180 km/h. The experimental acceleration records were filtered based on a low-pass digital filter with a cut-off frequency equal to 30 Hz.

The numerical results after updating revealed a better approximation to the experimental results, in comparison with the results before updating. In this context it should be highlighted the significant improvement of the correlation between the records of displacements at deck and supports, once the model calibration process led to an overall increase of the stiffness of the structure and also the vertical stiffness of the supports. In terms of accelerations, the inclusion of the track irregularities was crucial to obtain a better agreement with the experimental results, especially for higher frequencies [22, 23].

6.4 Alverca Viaduct

6.4.1 Description

Alverca railway viaduct is a flyover structure located at km +18.676 of the northern line of the Portuguese railways. Its construction allowed to separate the rail traffic flowing in the downstream and upstream directions and also to maintain the maximum speed of trains at 200 km/h. Figure 6.8 presents a side view of the current zone of the viaduct (Fig. 6.8a) and a cross-section of the deck (Fig. 6.8b).

The viaduct has a total length of 1091 m divided into 47 simply supported spans with the following spans: 9 × 16.5 m + 9 × 17.5 m + 29 × 21.0 m. Each span supports one single railway track and is composed of a prefabricated and prestressed U shape beam on which pre-slabs serving as formwork to the concrete upper slab cast in situ were placed, forming a single-cell box-girder deck. The ballast retaining walls are monolithically connected to the upper slab of the deck.

(a) (b)

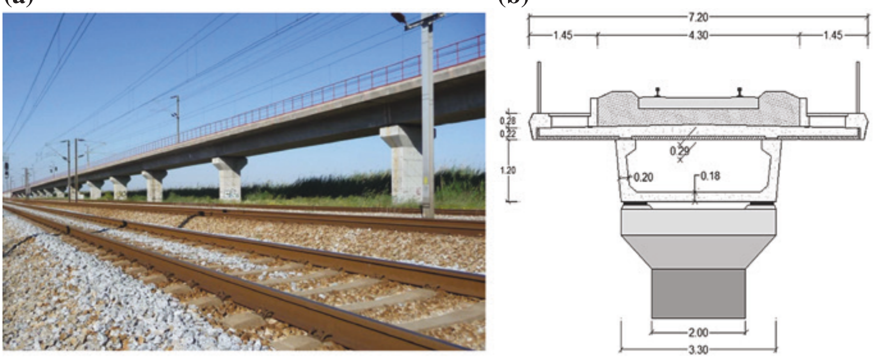

Fig. 6.8 Alverca viaduct: **a** perspective view; **b** cross-section of the deck

The deck is directly supported in the piers and in the abutments by elastomeric reinforced bearings. In each span the supports are fixed in one extremity and longitudinally guided in the other extremity. The track consists of UIC60 continuously welded rails, elastomeric rubber pads, prestressed concrete monoblock sleepers and a 30 cm ballast layer under sleepers.

6.4.2 Numerical Model

The dynamic analysis of the Alverca railway viaduct was carried out using a three-dimensional numerical model, including the track, developed in Ansys software. The analysis focused on the three spans adjacent to the north abutment: one 16.5 m long span (Span 1) and two 21 m long spans (Spans 2 and 3). Additionally, an extra extension of the track, with a length of 6 m, apart from the abutment, was modelled in order to simulate the effect of the track over the adjacent embankment. Figure 6.9 shows an overview of the numerical model with a detail of the track components.

The prefabricated beam, the upper slab and the ballast retaining walls were modelled by shell finite elements. The sleepers, the rail pads and the ballast layer were modelled by volume finite elements. The compatibility of displacements and rotations between the nodes of the precast beam and the nodes of the upper slab as well as the compatibility of displacements between the nodes of the upper slab of the deck and the lower nodes of the ballast layer were accomplished by rigid finite elements. Each support was regarded as a single point and modelled by a spring element. The rails were modelled as beam elements, positioned at their center of gravity. The non-structural elements such as safeguards and edge beams were considered as additional masses and applied to the nodes of the finite element mesh according to the real location of those elements. The numerical model of the viaduct includes 19,018 nodes and 20,906 elements. The level of refinement of the finite element mesh was optimized in order to reduce the time of modal analyses, which will have to be performed during the automatic calibration process of the numerical model.

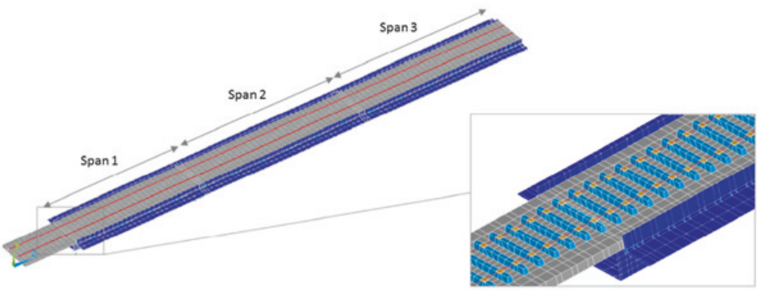

Fig. 6.9 Numerical model of Alverca viaduct including the track

Table 6.2 Characterization of the main parameters of the numerical model of Alverca viaduct

Parameter	Designation	Statistical properties		Limits (lower/upper)	Adopted value	Unit
		Distribution type	Mean value/standard deviation			
E_{c1} E_{c2} E_{c3}	Modulus of elasticity of concrete of the upper slab (Span 1/Span 2/Span 3)	Normal	35.4/4.3	28.4/42.4	35.4	GPa
E_c	Modulus of elasticity of concrete of the prefabricated beam	Normal	40.9/4.9	32.9/49.0	40.9	GPa
ρ_c	Density of the concrete	Normal	2446.5/122.3	2245.9/2647.1	2469.8	kg/m^3
K_v	Vertical stiffness of supports	Uniform	5400/2020.7	1900/8900	5200	MN/m
K_{h1} K_{h2} K_{h3}	Longitudinal stiffness of the supports (Span 1/Span 2/Span 3)	Uniform	3.35/0.89	1.8/4.9	3.6	MN/m
e_{bal}	Thickness of the ballast layer	Normal	0.25/0.013	0.23/0.27	0.25	m
E_{bal}	Modulus of elasticity of the ballast	Uniform	140/34.6	80/200	145	MPa
ρ_{bal}	Density of the ballast	Uniform	1875/129.9	1650/2100	2039	kg/m^3

Table 6.2 presents the main geometrical and mechanical parameters taken under consideration in the numerical model of the viaduct, including its designation, the statistical properties and the adopted value. The lower and upper limits of each parameter are also defined, and will be taken into account during the calibration process of the numerical model.

6.4.3 *Calibration*

The calibration of the model involved the use of 11 numerical parameters and 12 modal responses (6 frequencies and 6 MAC values) regarding the global vibration modes of the structure and the local vibration modes of the upper slab of the deck of span 2. The experimental modal parameters were obtained from an ambient vibration test described in Malveiro et al. [15].

The genetic algorithm was based on an initial population consisting of 30 individuals and 200 generations, for a total of 6000 individuals. In this algorithm the number of elites, as well as the number of substitute individuals, was equal to 1. The crossing rate was equal to 50 % and the mutation rate was equal to 15 % with a standard deviation varying throughout the optimization process and ranging between 0.10 and 0.01.

Figure 6.10 shows the ratios of the values of the main numerical parameter relative to the limits indicated in Table 6.2, obtained from three independent optimization runs (GA1 to GA3) based on different initial populations. The values of the numerical parameters are indicated in brackets.

The parameters that demonstrate to have more influence over modal responses, e.g., the modulus of elasticity of concrete of the precast beam and upper slab of the decks, the horizontal stiffness of supports and the density of concrete, provide estimates with lower variability. Furthermore, the parameters that less influence the modal responses, such as density and modulus of elasticity of ballast tend to present greater variation for the different optimization runs [15].

Figure 6.11 shows a comparison of the experimental and numerical after calibration modal configurations of the viaduct. The configurations presented are related to global modes of vibration of the structure (1G to 3G) and local modes of vibration of the upper slab of the deck (1L to 3L). The graphical representation of the global mods include the 3 spans of the viaduct, while for local modes only span 2 is presented. The results after calibration concern the case of optimization GA2, which was the case with lower residual of the objective function, equal to 0.556. In the same Figure the values of the numerical frequencies after calibration (f_{num}), experimental frequencies (f_{exp}) and MAC values, are also shown. In brackets the values of the numerical frequencies before calibration, which resulted from

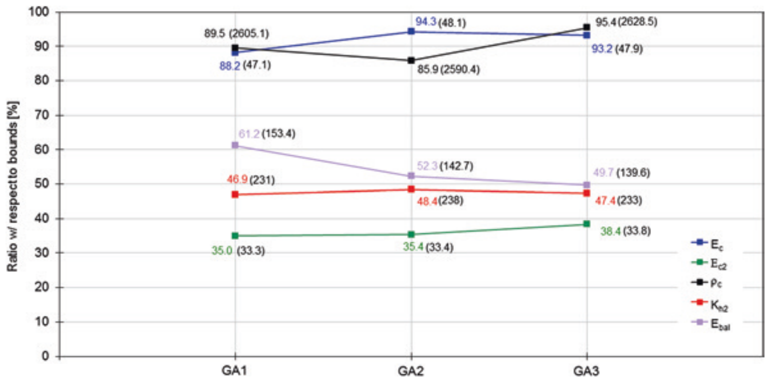

Fig. 6.10 Values of the numerical parameters for the optimization runs GA1 to GA3

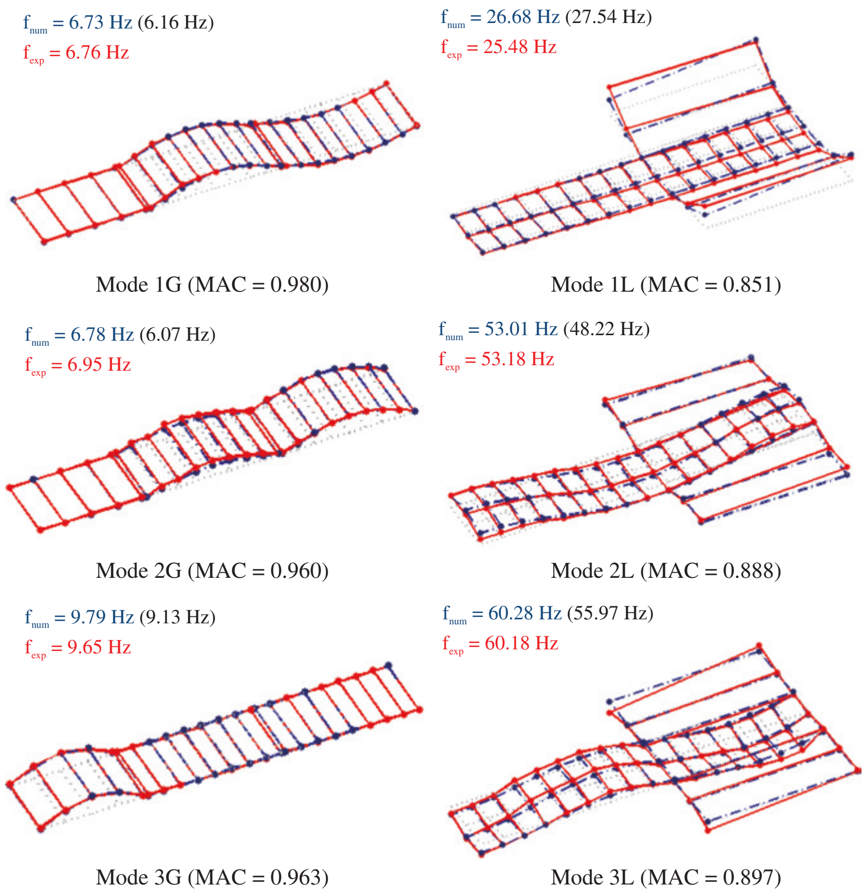

Fig. 6.11 Comparison between the experimental and numerical modal parameters

the modal problem resolution based on the adopted values of the parameters listed in Table 6.2, are also indicated.

There is a good match between the numerical and experimental modal configurations, particularly in global vibration modes. The average error of global modes frequencies decreased from 9.0 % before calibration to 1.5 % after calibration. The average value of the MAC parameter remains the same, before and after calibration, and equal to 0.968. For local modes, the average error of the frequencies went from 8.1 %, before calibration, to 1.7 % after calibration, while the average value of the MAC parameter increased from 0.784, before calibration, to 0.879 after calibration.

6.4.4 Validation

The validation of the numerical model was based on the results of a dynamic test under traffic actions. This test allowed the evaluation of the dynamic response in terms of displacements and accelerations at the mid-span section of the deck slab of span 2, for the passage of Alfa Pendular train at 185 km/h [15, 18].

The vertical displacement was measured on the lower slab of the deck by a LVDT positioned by means of a metallic tripod fixed on the ground (Fig. 6.12a). The vertical acceleration was measured on the upper slab (Fig. 6.12b) and lower slab (Fig. 6.12c) of the deck using two piezoelectric accelerometers. The accelerometer located on the upper slab involved the installation of a metallic protection tube in the interior of the ballast layer. The accelerometer installed on the lower slab was fixed by means of metallic plates bonded to the surface of the concrete.

The numerical responses were obtained based on a dynamic analysis considering train viaduct interaction, including measured track irregularities, performed by TBI software. The numerical model of the viaduct used for the dynamic analysis involved a reduction of the longitudinal stiffness of the guided supports in order to correctly reproduce the mobility of the supports under traffic actions [15]. The numerical model of Alfa Pendular train used in the analyzes is described by Meixedo et al. [18]. The contribution of 33 vibration modes for the response of the viaduct, with frequencies between 6.73 and 30 Hz, was considered. The time step of the analysis was equal to 0.001 s. The adopted values of the damping coefficients were equal to the average values obtained from an ambient vibration test [15].

Figure 6.13 presents a comparison between the numerical and experimental time records of the vertical displacement of the lower slab and vertical accelerations of

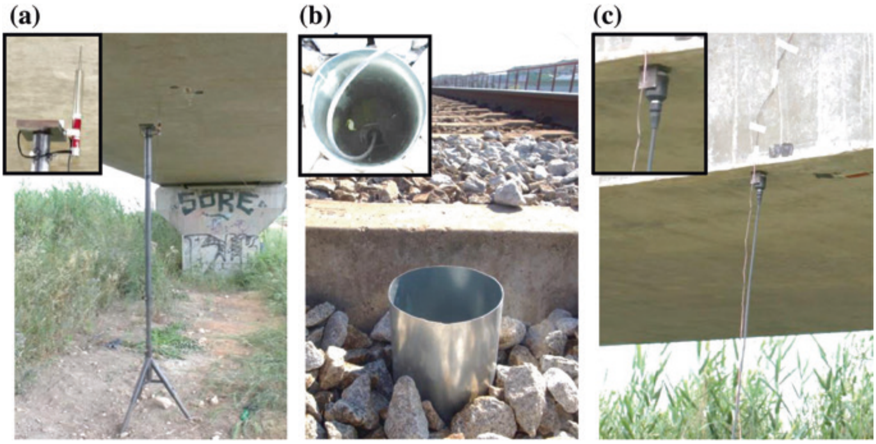

Fig. 6.12 Dynamic test under railway traffic: **a** LVDT on the deck; **b** accelerometer on the upper slab of the deck; **c** accelerometer on the lower slab of the deck

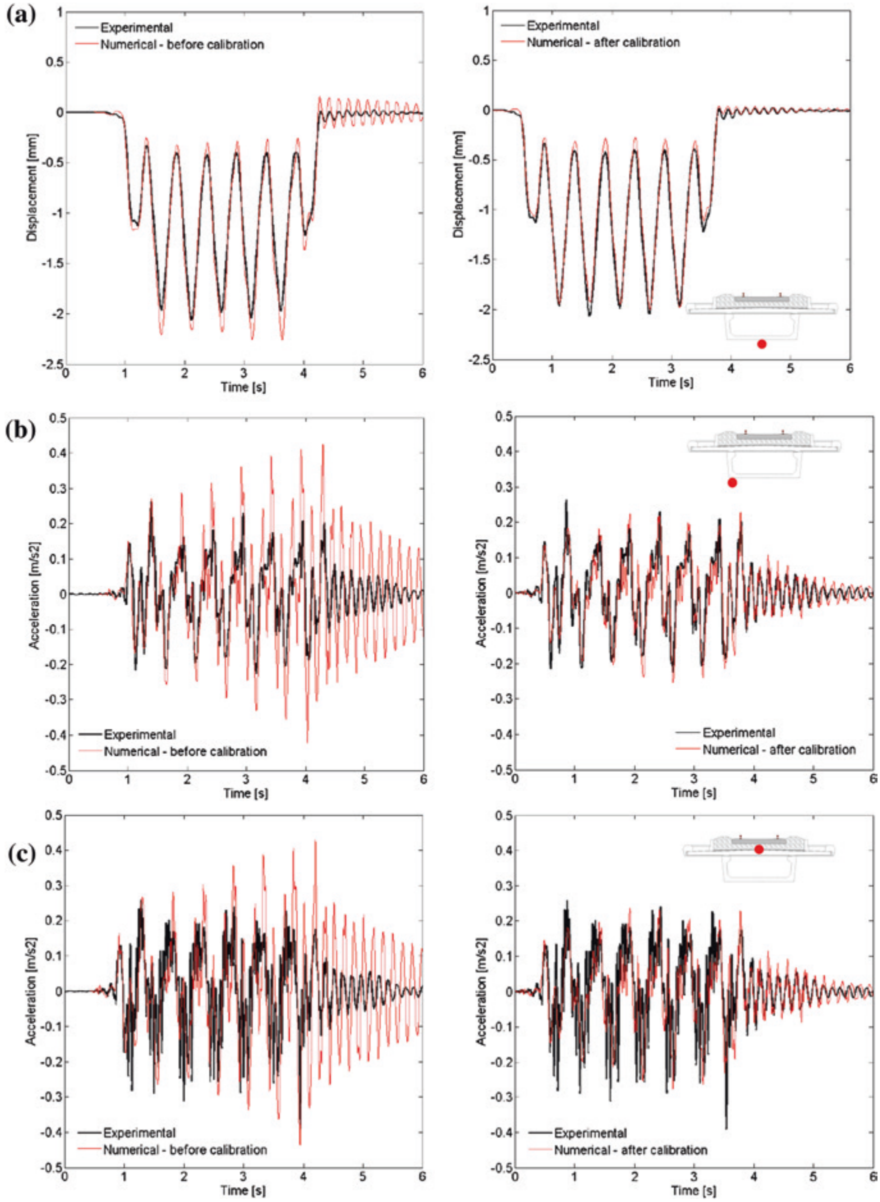

Fig. 6.13 Comparison of the experimental and numerical, before and after updating, dynamic responses of the viaduct for the passage of Alfa Pendular train at a speed of 185 km/h, at the midspan section of the deck: **a** displacements; **b** accelerations in the lower slab; **c** accelerations in the upper slab

the upper and lower slabs, in the mid-span section of span 2, for the passage of Alfa Pendular train at 185 km/h. The experimental records were filtered based on a low-pass digital filter with a cut-off frequency equal to 30 Hz.

The results show a very good agreement between experimental and numerical records after calibration, as well as a significant improvement in relation to the numerical records before calibration. The dynamic responses, in particular in terms of displacement of the deck, are clearly dominated by the frequency associated with the passage of the regularly spaced groups of axles (f) with a spacing (d) of 25.9 m (f = v/d = 185/3.6/25.9 = 1.98 Hz).

Regarding the response in terms of acceleration, besides the contribution of the frequency related to the train action, it should be noted the important contribution of the vibration mode 1G at a frequency of 6.47 Hz. This contribution is particularly evident in the record before calibration where inclusively is visible an amplification of the dynamic response. The response is also influenced by the contribution of higher frequencies, above 15 Hz, mainly related to the track irregularities [15, 18].

6.5 Conclusions

In this chapter the computational implementation of an iterative method for calibration of FE numerical models based on genetic algorithms, and its application to the dynamic models of two railway bridges, São Lourenço bridge and Alverca viaduct, was described.

The developed computational implementation allows performing an automatic and efficient calibration of numerical models based on experimental data, particularly modal parameters, and based on the interoperability between three software packages: Ansys, Matlab and OptiSlang.

The calibration results of the numerical models of both bridges conducted to stable estimates of the main numerical parameters and modal responses, considering different initial populations, and therefore demonstrating the robustness and efficiency of genetic algorithms in complex optimization problems with a large number of variables. The results also showed a significant improvement of the numerical models before calibration, as attested by the important reduction in the mean value of the errors associated with the frequencies of vibration and MAC parameters.

The validation of the numerical models involved the comparison of the measured responses from dynamic tests under railway traffic, with the numerical responses obtained through TBI software, based on train-bridge dynamic interaction models including track irregularities. The experimental and numerical records of displacements and accelerations obtained on the decks of both bridges showed a very good agreement, especially for the models after calibration.

In future works, the calibrated numerical models of São Lourenço bridge and Alverca viaduct may serve as basis for the implementation of advanced damage detection techniques. These techniques, based on numerical simulations, will rely on dynamic performance indicators associated to the train-track-bridge system, namely those associated to the traffic stability, track stability and passengers comfort. Based on these studies some recommendations that can support the decisions of infrastructure managers, with impact on reducing the costs of inspection and maintenance of bridges and on increased safety in operation, will be established.

Acknowledgments The authors would also like to thank to REFER, the company responsible for the management of the Portuguese railway network, for all the information provided about São Lourenço bridge and Alverca viaduct and for all the support given during the experimental tests, as well as the support provided by CSF, Centre of Competence in Railways of the Faculty of Engineering of the University of Porto.

References

1. ANSYS (2007) Structural analysis guide—release 11.0. In: ANSYS Incorporate, editor
2. Brehm M (2011) Vibration-based model updating: reduction and quantification of uncertainties. Ph.D. thesis, Bauhaus-Universitat Weimar, Weimar, Germany
3. Brehm M, Zabel V, Bucher C (2010) An automatic mode pairing strategy using an enhanced modal assurance criterion based on modal strain energies. J Sound Vib 329:5375–5392
4. Brownjohn J, Xia P, Hao H, Xia Y (2001) Civil structure condition assessment by FE model updating: methodology and case studies. Finite Elem Anal Des 37:761–775
5. Cantieni R, Brehm M, Zabel V, Rauert T, Hoffmeister B (2008) Ambient modal analysis and model updating of a twin composite filler beam railway bridge for high-speed trains with continuous ballast. In: IMAC-XXVI conference on structural dynamics. Orlando, USA
6. Chan THT, Li ZX, Yu Y, Sun ZH (2009) Concurrent multi-scale modeling of civil infrastructures for analyses on structural deteriorating—Part II: model updating and verification. Finite Elem Anal Des 45:795–805
7. Chellini G, Salvatore W (2007) Updated models for steel-concrete composite HS railway bridges. In: Conference experimental vibration analysis for civil engineering structures (EVACES). Porto, Portugal
8. Cunha Á, Caetano E, Calçada R, Roeck GD, Peeters B (2003) Dynamic measurements on bridges: design, rehabilitation and monitoring. Bridge Eng 156:135–148
9. Ewins DJ (1984) Modal testing: theory and practice. Research Studies Press, London
10. Friswell MI, Mottershead JE (1995) Finite element model updating in structural dynamics. Kluwer Academic Publishers, Boston
11. Frýba L, Pirner M (2001) Load tests and modal analysis of bridges. Eng Struct 23:102–109
12. Huang M, Guo W, Zhu H, Li L (2008) Dynamic test and finite element model updating of bridge structures based on ambient vibration. Front Archit Civ Eng China 2:139–144
13. Jaishi B, Ren W-X (2005) Structural finite element model updating using ambient vibration test results. J Struct Eng 45:617–628
14. Liu W, Gao W, Sun Y, Xu M (2008) Optimal sensor placement for spatial lattice structure based on genetic algorithms. J Sound Vib 317:175–189
15. Malveiro J, Ribeiro D, Calçada R, Delgado R (2014) Updating and validation of the numerical model of a railway viaduct with precast deck based on dynamic tests. Struct Infrastruct Eng 10(11):1484–1509
16. Mares C, Friswell M, Mottershead J (2002) Model updating using robust estimation. Mech Syst Signal Process 16(1):169–183
17. Mathworks (2011) MATLAB—Getting started guide. Natick, USA

18. Meixedo A, Ribeiro D, Calçada R, Delgado R (2014) Global and local dynamic effects on a railway viaduct with precast deck. In: Railways 2014—the second international conference on railway technology: research, development and maintenance. Ajaccio, France
19. Mottershead J, Mares C, James S, Friswell M (2006) Stochastic model updating: Part 2—application to a set of physical structures. Mech Syst Signal Process 20(8):2171–2185
20. OptiSLang (2008) OptiSLang—the optimizing Structural Language, 3.0 edn. In: GmbH D (ed) Weimar, Germany, 2008
21. Ren W-X, Chen H-B (2010) Finite element model updating in structural dynamics by using the response surface method. Eng Struct 32:2455–2465
22. Ribeiro D (2012) Efeitos Dinâmicos Induzidos por Tráfego em Pontes Ferroviárias: Modelação Numérica, Calibração e Validação Experimental. Ph.D. thesis, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal [in Portuguese]
23. Ribeiro D, Calçada R, Delgado R, Brehm M, Zabel V (2012) Finite element model updating of a bowstring-arch railway bridge based on experimental modal parameters. Eng Struct 40:413–435
24. Ribeiro D, Calçada R, Delgado R, Brehm M, Zabel V (2013) Finite element model updating of a railway vehicle based on experimental modal parameters. Veh Syst Dyn 51(6):821–856
25. Schlune H, Plos M, Gylltoft K (2009) Dynamic finite element model updating of prestressed concrete continuous box-girder bridge. Earthq Eng Eng Vib 31:1477–1485
26. Teughels A (2003) Inverse modelling of civil engineering structures based on operational modal data. Ph.D. thesis, Katolic University of Leuven, Leuven, Belgium
27. Teughels A, Roeck GD, Suykens J (2003) Global optimization by coupled local minimizers and its application to FE model updating. Comput Struct 81:2337–2351
28. Wenzel H, Pichler D (2005) Ambient vibration monitoring. Wiley, Chichester
29. Yang YB, Chen YJ (2009) A new direct method for updating structural models based on measured modal data. Eng Struct 31:32–42
30. Yu X, Gen M (2010) Introduction to evolutionary algorithms. Springer, Heidelberg
31. Zabel V, Brehm M (2009a) System identification of high-speed railway bridges. In: Gmbh D (ed) Weimar optimization and stochastic days 50, Weimar
32. Zabel V, Brehm M (2009b) Stochastic model updating methods. In: IMAC XXVI conference on structural dynamics, Orlando, USA
33. Zong Z-H, Jaishi B, Ge J-P, Ren W-X (2005) Dynamic analysis of a half-through concrete-filled steel tubular arch bridge. Eng Struct 27:3–15