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Validation of the Nonlinear Numerical Model of a Stone Masonry Bridge Under Railway Traffic

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Abstract

This paper presents the validation of a nonlinear finite element numerical model of a multi-span stone arch railway bridge based on experimental tests and under in-service freight trains. Static loading tests allow evaluating the bridge response in terms of vertical displacements in the arches and opening/closure deformations on specific block joints of the arches while dynamic tests involved measuring accelerations in the bridge deck. The nonlinear bridge finite element model is developed by combining the potentialities of a global continuous model, based on Drucker-Prager model, and a local modelling approach based on a dedicated contact model. This contact model allows reproducing the behaviour of the joints between the masonry blocks, including the cracking patterns identified in some arches derived from an inspection campaign. The dynamic analyses are based on advanced train-bridge dynamic interaction model, including the measured track irregularities. All the numerical responses are in very good agreement with the experimental responses.

Keywords: railway masonry bridges, train-bridge dynamic interaction, nonlinear dynamic analysis, experimental testing, model validation, finite element numerical model.

1 Introduction

The longevity, robustness and low operating costs that are associated with stone masonry arch bridges allow them to be considered a good example of sustainability.

The large number of stone masonry bridges still in operation nowadays in the railway network [1], justifies the need to study this type of bridge, which is deemed essential to ensure this past knowledge is not lost but further deepened in light of current modern techniques.

Nowadays, the significant increase in freight transport needs leads to the great majority of stone masonry bridges in service being subjected to higher traffic loads and traffic speeds than those for which they were designed. These new demands require more frequent inspection and maintenance operations in addition to greater care in assessing the structural degradation due to age and intensive use [2]. The growing need for expansion, higher capacity and new requirements for people and freight mobility are issues of major importance, which justify the need for efficient condition assessment strategies aiming to extend the life cycle of these older structures so that appropriate measures can be implemented to repair, rehabilitate or consolidate these structures.

Typically, the condition assessment of railway bridges under traffic loads is performed based on dedicated dynamic analyses, which allows evaluating the structural and traffic safety and passenger comfort [3]. These dynamic studies rely on the development of advanced numerical models capable of simulating the complex dynamic behaviour of the train-track-bridge system, and its interfaces, particularly the wheel-rail and the track–bridge interfaces [4].

In this paper, a validation of a nonlinear FE numerical model of the Durrães stone arch railway bridge based on experimental tests and under in-service freight trains. The validation of the dynamic behaviour of the bridge was based on advanced vehicle-bridge dynamic interaction models, including the measured track irregularities. Both, bridge and vehicle FE models, were previously calibrated based on dynamic tests [5, 6]. The bridge model is developed by combining the potentialities of a global continuous homogeneous model, based on FEM and Drucker-Prager model, and a local nonlinear modelling approach based on a contact model. This model allows reproducing the behaviour of the joints between the masonry blocks, including the longitudinal cracking patterns identified in some arches derived from an inspection campaign.

Static loading tests allow evaluating the bridge response in terms of vertical displacements in the arches, opening/closure deformations on specific block joints of the arches and vertical compressive stress variations in the piers. Dynamic tests involved measuring accelerations in the bridge deck and piers.

2 Case-study

2.1 Durrães railway bridge

The Durrães bridge, illustrated in Figure 1, is a multi-span stone arch bridge built in 1878. The bridge extends over 178 m, with 5.3 m width and a maximum height of 22 m from the deck to the ground. It consists of 16 arches, each one with 9 m span, sup-ported by 15 piers and two abutments. The bridge deck carries a single electrified railway track on Iberian gauge and allows the circulation of freight and passenger trains with maximum speeds of 100 km/h and 120 km/h, respectively. The bridge is in an overall good condition considering its age (over 100 years old), nevertheless, there is one relevant structural damage observed in arch A15 consisting of the existence of longitudinal crack patterns with visible opening of joints (Figure 1b). This longitudinal cracking follows the interface between the stones in the first and second rows of the arches' intrados, which is predictably associated with excessive concentration of stresses due to the traffic loads derived from the increasing traffic.

a) b)

Figure 1: Durrães railway bridge: a) overview; b) lateral longitudinal cracks in arch A15.

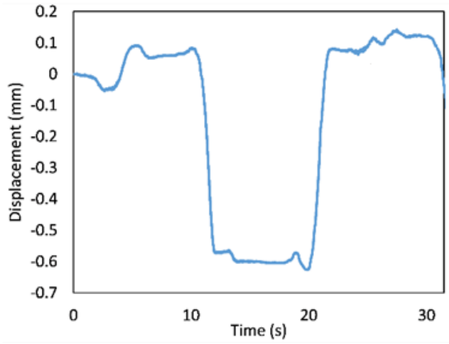
2.2 Experimental testing

An extensive experimental campaign was performed in the Durrães bridge, comprising both material testing and dedicated structural tests. The material testing consisted of laboratory tests, for the mechanical characterization of the stone and masonry joints, *in-situ* tests based on Ménard pressuremeter tests, for the characterization of the infill materials, and single/double flat-jack tests for the characterization of the masonry components [7]. The structural tests involved an ambient vibration test for modal identification [5], as well as dynamic and static tests under freight trains for the characterization of the structural responses in terms of displacements, accelerations and stresses.

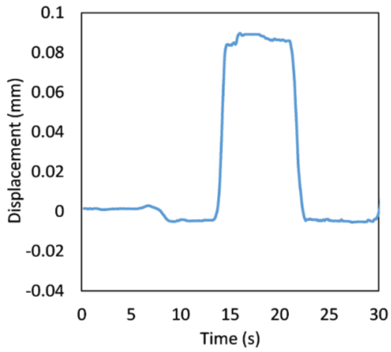
The static load tests were performed for evaluating the bridge response, in terms of deformations and stresses in the piers and arches, under in-service freight trains at specific positions [8]. The train is the Takargo freight train, and comprises a diesel locomotive Euro 4000 pulling 14 freight Sgnss type wagons. Particularly for the arches, it was installed a set of Linear Variable Differential Transformers (LVDTs) strategically positioned at 1/3 span to monitor the vertical displacements (Figure 2a) and at 1/2 span to monitor the opening/closure deformations at some joints located in the arch intrados (Figure 2b). Concerning arch A15, the graph in Figure 2c shows that the maximum downward displacement is around 0.60 mm, and in Figure 2d the measured transversal deformations reveal the existence of a quite evident crack opening, with magnitudes in the order of 0.08 mm.

a)

b)



c)

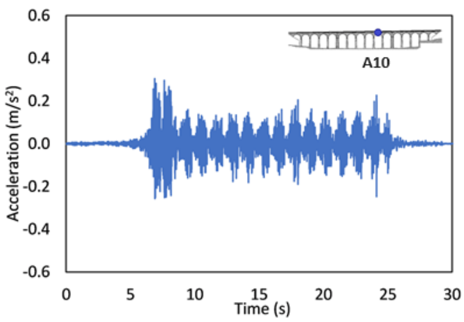


d)

Figure 2: Static load test in arch A15: a) vertical LVDT; b) transversal LVDT; c) vertical displacement measured in 1/3 span; d) relative transversal displacements in opening joint.

The dynamic test involved measuring vertical accelerations in the arches during the passage of an in-service Takargo freight train (Figure 3). The accelerations were measured in the mid-span of the arches (at the deck level). Uniaxial piezoelectric accelerometers were used, PCB model 393A03, with a sensitivity of 1V/g and a measurement range of ± 5g, accounting for the higher acceleration levels expected under traffic actions. Figure 3b shows the vertical acceleration record in one position on the deck (arch A10) for a Takargo freight train circulating at 60 km/h.

a)



b)

Figure 3: Dynamic test: a) freight train passage; b) vertical acceleration in arch A2.

3 FE Numerical Models

3.1 Global Modelling

The global numerical modelling of Durrães bridge, illustrated in Figure 4, was performed using a 3D FE model developed in ANSYS software [9]. The arches, piers, spandrel walls, infill, abutments, foundations, embankment, rails, ballast and sleepers, were modelled using volumetric finite elements (SOLID185). The rails were modelled with 2-node beam type elements (BEAM 188). The complete model consists of 88908 FE elements and 105306 nodes. The boundary conditions were established using rigid supports fixing all displacements of the mesh nodes located at the base of the foundations and abutments.

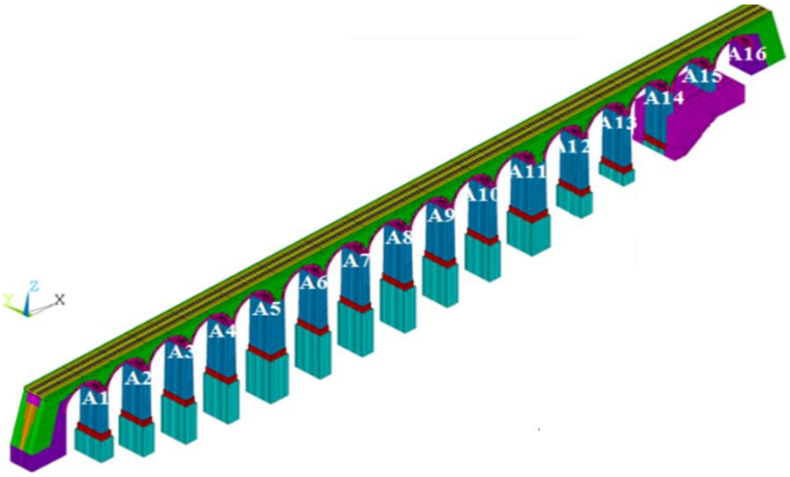


Figure 4: 3D global numerical model of Durrães bridge.

The elastic material properties of the masonry (in foundations, piers, arches and spandrel walls) and infill are characterized by the elastic modulus, Poisson's ratio and unit weight listed in Table 1. After the calibration process, the average error of the natural frequencies associated to global modes, taking as reference the corresponding experimental values, was equal to 4.0% and the average MAC value equal to 0.926 [5].

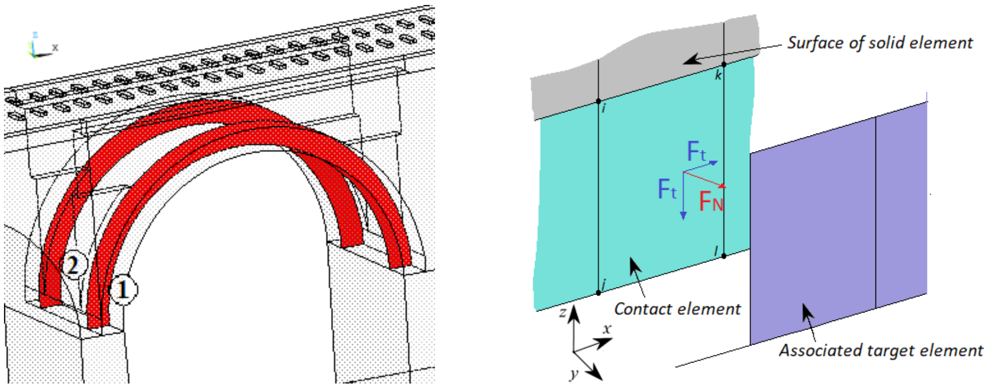
Material	Elastic Modulus (GPa)	Unit weight (kN/m^3)	Poisson's ratio
Masonry	6.6 – 10.7	24.5	0.2
Foundations	15		
Infill (upper/lower)	0.6 / 1.0	21.5	
Embankments	0.75	20.0	
Ballast	0.15		
Sleeper	36	28.9	
Rail	210	78.5	

Table 1: Elastic and mass parameters of the FE model of Durrães bridge.

The nonlinear material behaviour of the masonry (arches, spandrel walls and piers) and infill was reproduced by the Drucker-Prager model available in ANSYS software [9]. The uniaxial compressive strength and tensile strength parameters were considered to be equal to 4 MPa and 0.1 MPa, respectively. In the tension domain, an exponential softening function was adopted with a residual tensile stress equal to 10% of the tensile strength.

3.2 Local Modelling

The local modelling strategy involves a refinement of the global FE model in the region of arch A15, illustrated in Figure 5. The damage simulation involves the use of contact FE elements to simulate the nonlinear behaviour of the joints between the masonry blocks [8]. A surface-to-surface contact model (Figure 5b) including friction and cohesive behaviours is used for the block-to-block interfaces of arch A15.



a) b)

Figure 5: 3D local numerical model of Durrães bridge: a) location of contact/target elements on arch A15; b) adopted contact model

For the contact/target elements simulating the longitudinal cracks in the arch, a normal-stress stiffness value (K_n) of 0.60 MPa/mm and 0.40 MPa/mm was adopted, respectively for alignments 1 and 2, and a shear-stress stiffness value (K_s) equal to 10 MPa/mm. For the friction coefficient, a value of 0.75 was adopted. For the other interfaces, a higher value of normal stiffness was considered to prevent excessive deformation. Table 2 lists the main contact parameters assigned to all the contact/target FE elements.

Contact/target interface	K_n (MPa/mm)	K_s (MPa/mm)	Friction coefficient
Arch and Infill			
Spandrel and Infill	6		
Arch and Piers		10	0.75
Crack - alignment 1	0.4		
Crack - alignment 2	0.6		

Table 2: Contact elements' parameters.

4 Validation

The validation of the non-linear FE numerical model of Durrães bridge involved a two steps strategy. First, the results derived from a numerical static analysis under the locomotive Euro 4000 are compared with the corresponding responses obtained from the static loading tests. Second, a numerical dynamic analysis of the train-bridge system is performed for the passage of Takargo train and the corresponding responses are compared with the ones obtained from the dynamic tests under railway traffic.

4.1 Static Analysis

Figure 6 shows the values of the experimental and numerical vertical displacements at 1/3 span of arches and the relative displacements in the longitudinal cracks located on the interfaces between the spandrel wall and the arch A15. As evidenced by the figures, these numerical values are in very good agreement with the displacements measured on the experimental tests. In particular, the numerical vertical displacement at 1/3 span of arch A15 was equal to -0.58 mm.

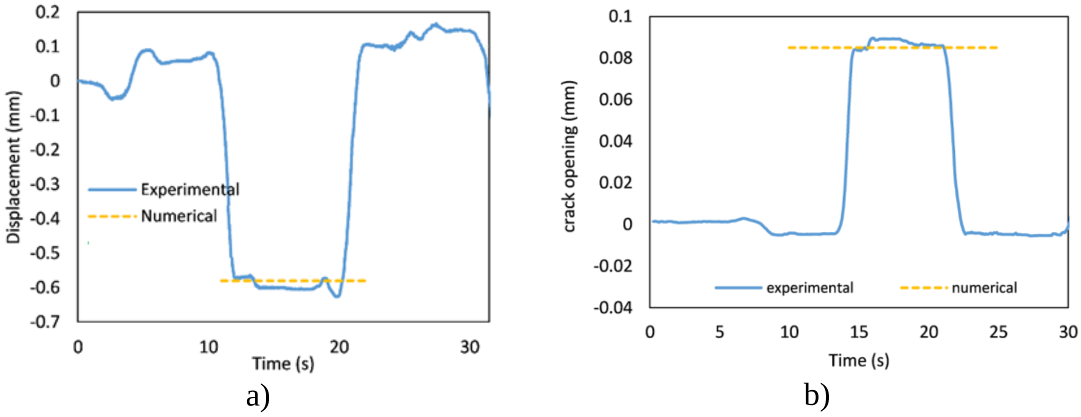
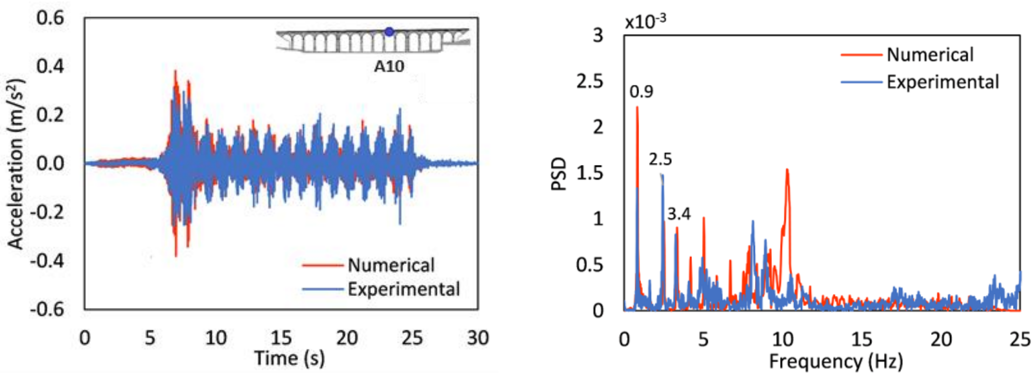


Figure 6: Experimental and numerical static displacements in arch A15: a) vertical displacements; b) crack opening values.

4.2 Dynamic Analysis

The numerical analysis of the train-bridge system was based on a dynamic analysis considering vehicle–structure interaction and including the measured track irregularities. The vehicle–bridge dynamic interaction was implemented in ANSYS software [9] considering both subsystems, train and bridge, modelled as a coupled system using advanced contact finite elements at the wheel-rail interface. [8]. The dynamic interaction only considers the vertical direction and allows the sliding and loss of contact at the wheel-rail interface. The vertical wheel-rail contact deformability was simulated based on a Hertzian spring. The Rayleigh damping coefficients for the bridge (α = 0.208 and β = 0.002) were estimated setting the damping coefficients equal to 2% for the 1st and the 10th vibration modes of the bridge. These two modes correspond to the first transversal and vertical modes, respectively.

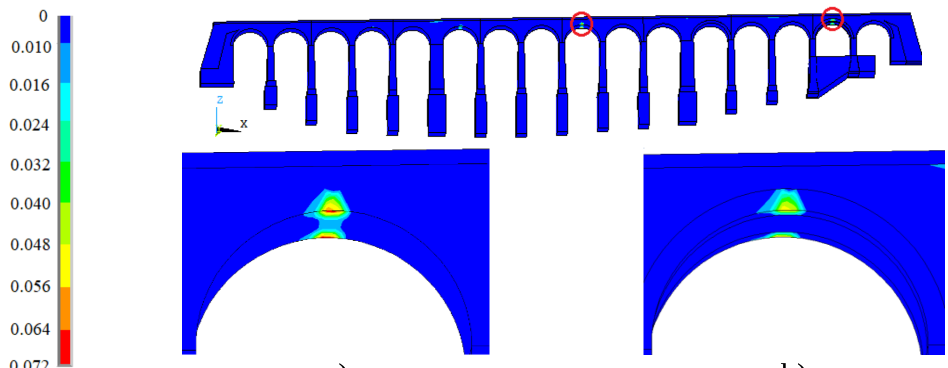
Figure 7 shows a comparison of the experimental and numerical vertical acceleration time series (filtered using a Butterworth low-pass filter with a 30 Hz cut-off frequency) in the bridge deck over arch A10 for the passage of the freight train at 60 km/h. The power spectrum density (PSD) functions of the acceleration records are also presented (Figure 7b). The accelerations records show a good agreement between numerical and experimental responses, particularly concerning the parts of the records related to the passage of the wagons. For these parts of the records, the breathing effect induced by the passage of regularly spaced groups of axles is quite well reproduced, as well as the peak acceleration values. Comparing the PSD plots, a good agreement is found in terms of amplitude peaks.



a) b)

Figure 7: Dynamic responses for the passage of the freight train at 60 km/h in arch A10: a) accelerations time-series, b) PSD amplitude.

Figure 8 show the plasticity state of the bridge structural elements in terms of maximum principal plastic strains for the passage of a freight train at 60 km/h. As observed, for this service loading, the bridge response exhibits nonlinear incursions at several arches, particularly at arches A9 and A15. The maximum values are concentrated in the arches' lateral zones, where significant tensile stresses occur. These identified critical zones are in accordance with the damages observed in the arches' intrados, where longitudinal cracks were found.



a) b)

Figure 8: Plastic strain level in terms of principal maximum strains (in ‰) for the freight train passage at 60km/h: a) arch A9; b) arch A15.

Finally, Figure 9 shows the numerical response in terms of vertical displacements at 1/3 span of arches A10 and A15, where static tests were performed. In the figure it is possible to evaluate the effect of adopting, or not, a nonlinear behaviour in the bridge numerical response. For the case of arch A15 in a more damaged condition with visible longitudinal cracks, there is a difference of around 10% in the vertical displacement when considering nonlinear behaviour. The results in arch A10 tend to show that for an arch with a less damaged state condition, the adoption of linear dynamic analysis leads to an accurate vertical response.

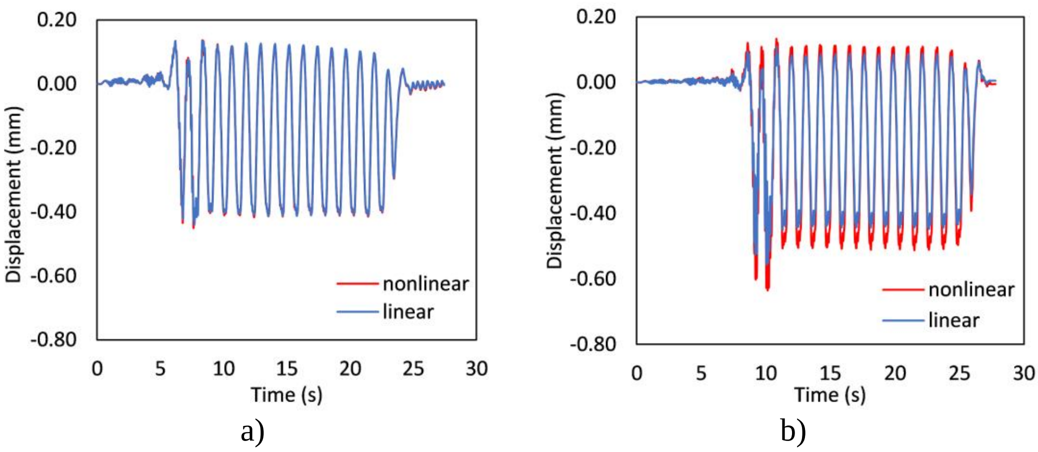


Figure 9: Vertical displacements in linear and nonlinear dynamic analysis: a) arch A10 and b) arch A15.

5 Conclusions

This paper described the validation of a FE numerical model of Durrães bridge based on experimental data obtained from static and dynamic tests under freight traffic actions. The dynamic analysis of the vehicle-bridge system was based on advanced interaction models, including the measured track irregularities.

For the bridge numerical model, a nonlinear model was adopted using a Drucker-Prager model allowing a softening representation in the tensile domain, which is particularly important for the masonry material. Additionally, the bridge numerical model was improved at a local level, in the case of arch A15, where longitudinal cracks in arch intrados were identified through visual inspections. This localized damage involved the use of dedicated contact FE elements with adjusted constitutive models. The adopted strategy allowed simulating the nonlinear behaviour of the discrete cracks within the continuous homogeneous model.

The validation of the nonlinear FE model of the bridge involved a two steps strategy. First, the results derived from a numerical static analysis are in very good agreement with the displacements measured on the experimental tests, namely, vertical deflections and opening displacements on the longitudinal cracks. Afterwards, a numerical dynamic analysis of the train-bridge system was implemented in ANSYS based on a dedicated contact formulation and the numerical responses were compared with those obtained from the dynamic tests. The numerical and experimental vertical acceleration records show good agreement, in terms of peak acceleration values as well as amplitude peaks obtained through PSD plots.

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