NUMERICAL MODELLING OF THE STRUCTURAL BEHAVIOUR OF STONE ARCH BRIDGES UNDER RAILWAY TRAFFIC LOADING

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SUMMARY

This paper is part of the studies of the StoneArcRail project and focuses on the numerical simulation of the structural response of stone arch railway bridges under traffic loading. The study comprehended three case studies: two multi-arch bridges, namely the Côa and Durrães bridges with 238 and 178m long, respectively, and a small one, PK124 with a single arch and 11m long. The two long bridges are modelled by a 3D finite element model using equivalent homogeneous elements and elastic materials, while the small bridge is modelled also resorting to FE micro modelling strategies. Dynamic analyses including train-bridge interaction and track irregularities were performed and nonlinear response analyses using detailed bridge modelling under incremental static loading allow to identify the load-carrying of the bridge models.

Keywords: *Stone masonry, railway bridges, numerical modelling, train-bridge interaction, load-carrying capacity.*

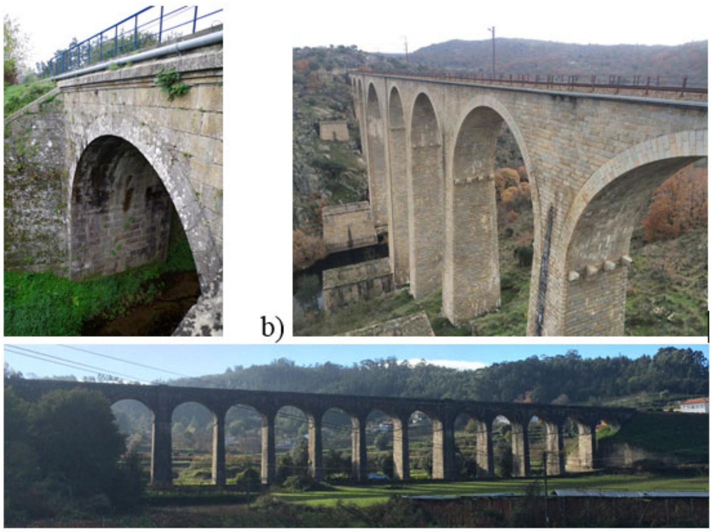
1. INTRODUCTION

This article reports on the work of the StoneArcRail project that seeks the experimental and numerical characterization of the structural behaviour of existing stone arch bridges in Portugal under rail traffic loading. This structural system has been widely used as shown in data reported of UIC [1], which several European have contributed to and concluded that about 60 % of railway bridges are arched ones or culverts. In the particular case of Portugal, 90% of the total existing railway bridges use this structural systems and 80 % of these bridges have spans lower than 5m and 70 % are aged between 100 and 150 years.

The evaluation of the structural response of stone arch bridges by means of suitable numerical modelling strategies with realistic loading conditions, settlements, material and structural composition can provide a better understanding of the structural behaviour and contributing to help on implementing suitable management plans for this type of bridges. The StoneArcRail project attempts to contribute to, yet not solve, some of the open issues found by the Portuguese infrastructures' network (IP-Infrastructures of

Portugal; ex-REFER) like the identification of load and train speed limits for in-service masonry arch bridges, establish safety and comfort criteria, specifically adapted for this type of bridges, define measures to mitigate the effects caused by train-induced vibrations in the masonry arches.

This project focused on the experimental and numerical study of a few bridge cases which can be considered representative of the most common and important typologies of masonry arch bridges existing (and in service) in the Portuguese railway network (Fig. 1). This paper focuses essentially in the work developed on the numerical part of the project that the specific objectives were set as follows: analysis of the vibration effects caused by railway traffic; evaluation of the influence of traffic loading parameters (speed and type of trains) and of the structural components on the whole bridge behaviour; assessment of structural and track safety and users comfort.



a) b) c)

Fig. 1. *Case studies: a) PK 124 bridge; b) Côa bridge; c) Durrães bridge.*

The study comprehended three case studies: two multi-arch bridges, namely the Côa bridge (Fig. 1b) in the Beira Alta Line towards the Vilar Formoso East Portugal-Spain border and Durrães bridge (Fig. 1c) in the Minho Line with 238 and 178 m long, respectively, and a small one located at PK 124 of the Minho line, near São Pedro da Torre with a single arch and 11m long (Fig. 1a). The two long bridges are modelled by a 3D Finite Element model using equivalent homogeneous elements and elastic materials, while the small bridge is modelled also resorting to FE micro modelling strategies. All the three cases were aim of an extensive experimental campaign including topography levelling, ambient vibration tests, GPR (Ground Penetrating Radar) and DPSH tests, Pressuremeter tests (PMT), in situ flat-jack testing and some Laboratory tests to the mechanical characterization of the stone. Some of the results of this experimental campaign are shown in other paper of the conference [2].

2. NUMERICAL MODELLING

The numerical study was focused on definition and calibration of the numerical models for the three case study bridges and two trains, one freight train and another passenger train. Both, finite element method (FEM) and discrete element method (DEM) were used to perform the bridges' structural analysis resorting to usual commercial computer codes. Besides conventional CAD and mesh pre-processing packages, four structural analysis computer codes were used, namely the FEM based CAST3M [3] and ANSYS [4], and the DEM based UDEC [5] and 3DEC [6]. FEM was used to perform structural analysis based on continuous global elastic models. More detailed models based on non-linear behaviour of material constituents as discrete definition of structural masonry were defined based on DEM and FEM.

Durrães and PK124 bridges were developed initially in 2D using information from the topography levelling conducted, project data and some data collected *in-situ* or from the experimental campaign. The finite element (FE) mesh was generated in CAST3M initially in 2D and then given thickness to the model. Mesh information, such us nodes and elements are extracted and convert into a specific language readable by the ANSYS program through a function created in Matlab [7]. For Coa bridge and both trains the adopted modelling strategy was performed based on 3D initial geometry. The same mesh was available on CAST3M and ANSYS, thus allowing running similar analyses in both programs. The boundary conditions are applied in each by blocking the same nodes and displacement directions.

The FE model in ANSYS was mainly used to perform the calibration (requiring intensive and iterative calculations for material parameter characterization) and the simulation of traffic circulation by means of dynamic time history analysis. The FE detailed model in CAST3M and DE models in UDEC and 3DEC were used to evaluate the bridge load carrying capacity.

The strategies used to generate the FE meshes of the solid elements of the PK124 detailed model (stone blocks and infill) are similar to does for the PK124 global FE model although the generation of joint elements between the stone blocks is required in the detailed FE models. For this purpose the strategies to generate the FE detailed models of stone arch bridges used in previous studies [8, 9] were also adopted in the PK124 bridge case study.

2.1. FEM based continuous models

FEM based continuous models (see Fig. 2) were developed for the three bridges and both trains considering linear elastic material behaviour. Non-homogeneous materials such masonry and infills were simulated by equivalent homogeneous and continuous materials with linear elastic mechanical properties duly calibrated based on the results of experimental assessment [2]. Based on first estimates of material parameters bounded by experimental data, numerical modal data including frequencies and mode shapes were obtained and compared with experimentally obtained ones, applying the Modal Assurance Criteria (MAC) introduced by Allemang [10]. In order to reduce the differences between the numerical and experimental modal results, a calibration procedure was adopted, based on the ambient vibration test results and involving two stages: a sensitivity analysis (to select the most influencing parameters) and an optimization process (involving the parameters selected in the sensitivity analysis, the optimizing variables). The calibration methodology consisted on an iterative method based on genetic algorithms, originally developed by Ribeiro [11]. Both trains were also calibrated and Alfa Pendular train FEM model used was from [12]. These numerical models were then used for the bridges' dynamic analyses, wherein train-bridge interaction was also considered.

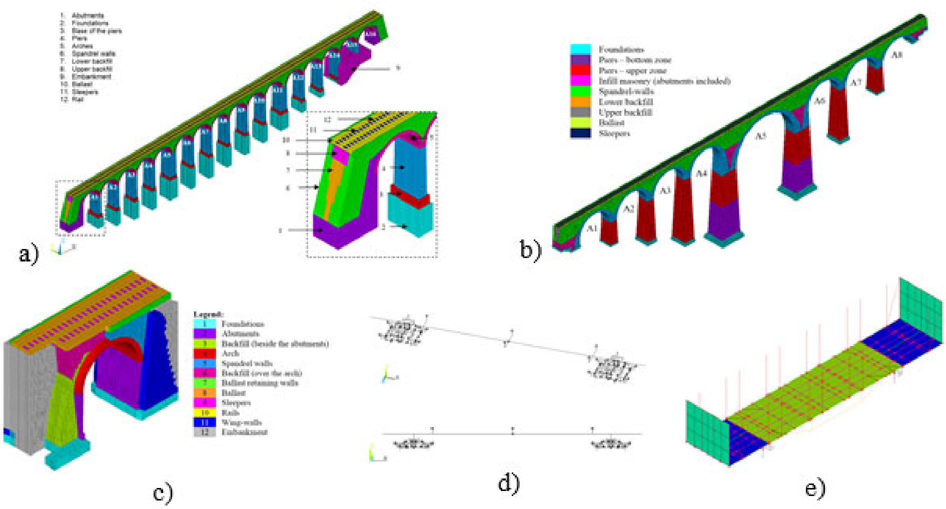


Fig. 2. *FE global models: a) Durrães bridge; b) Côa bridge; c) PK 124 bridge; d) Alfa Pendular train; e) Freight train.*

2.2. Detailed FEM and DEM based discrete models

Detailed FEM and DEM based discrete models were developed for PK124 bridge to allow performing more refined analyses wherein the non-linear behaviour can be considered for assessing load carrying capacity of the bridge structure under traffic loading. In these FEM models, the masonry bridge components (arches, spandrels, abutments and backfill behind abutments) are represented by FE micro modelling strategies using solid elements to define the individualized blocks and zero thickness joint elements at their interfaces (stone-to-stone joint type). The backfill is also modelled with solid elements connected to zero thickness joint elements in the interfaces between the infill and blocks of the arches and pavement, with different characteristics for the infill-to-stone joint type (Fig. 3a). The stone blocks are considered with linear elastic behaviour characterized by the elastic modulus (E) and Poisson's ratio (ν) and its specific weight (γ), while the values of initial normal (kn) and shear (ks) stiffness of the joint elements, as well as the material parameters of the infill material, were defined based on laboratory tests and modal identification results; material parameters estimated in experimental characterization of Durrães bridge were also considered to adjust the mechanical properties assigned to the PK124 bridge model.

For comparative purposes, and to achieve better confidence on the results of load carrying capacity, two other types of numerical modelling strategies were adopted for the PK124 bridge, namely one based on the Discrete Element Method (DEM) considering 2D and 3D models (Fig. 3b) and another based on the Rigid Block (RB) limit analysis method. For the DEM models the material behaviour was also tuned so as to agree with the FEM modelling, assuming the initial material parameters' values and constitutive models to control the nonlinear evolution similar to those used in the FEM model. Thus, the contact elements are controlled by a nonlinear Mohr-Coulomb friction model without dilatancy and the Drucker-Prager model is used to represent the infill material behaviour, both constitutive models available in the computer codes UDEC and 3DEC used for these analysis. For the RB analysis, performed resorting to the RING software, identical characteristics were considered for the geometry, materials and loading as for those used in the detailed FE and DE 2D models. The material parameters have also been defined in view of allowing the comparison of results of the rigid block model with those obtained from the FE and DE models.

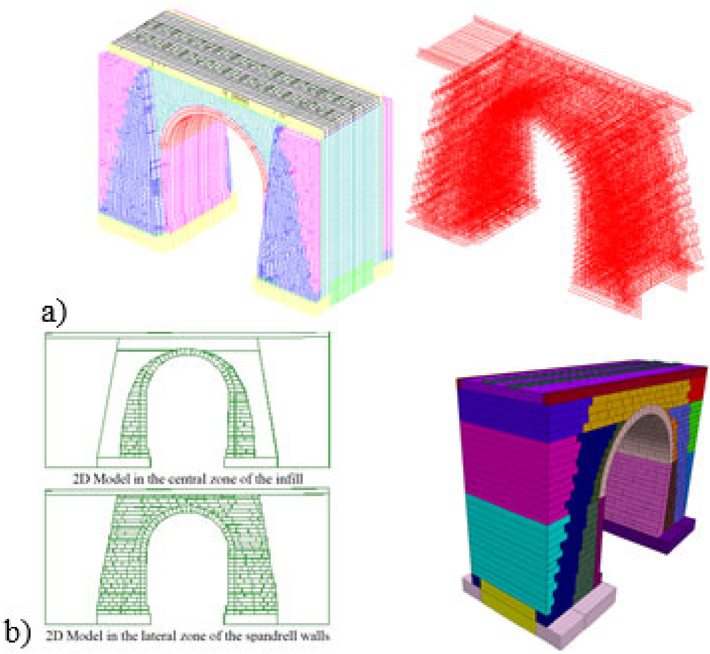


Fig. 3. FE and DE detailed models of PK 124 bridge: a) FEM based CAST3M model (solid and joint elements); b) DEM based 2D and 3D models.

3. NUMERICAL ASSESSMENT

One of the main objectives of the project is to evaluate the behaviour of both the bridges and the trains, due to the dynamic effects caused by interaction between them and by the present of track irregularities. The dynamic responses were carried out by TBI (Train-Bridge Interaction) software, developed in Matlab by Ribeiro [13]. The software uses the modal superposition method for solving the dynamic equilibrium equations of the bridge, and a direct integration method (Newmark method) for solving the dynamic equilibrium equations of the train.

Another goal of the project consisted in evaluating the limit load which requires numerical modelling beyond the linear elastic behaviour as well as the assessment of the limit loading applied statically on the bridge models.

3.1. Dynamic effects

The dynamic analyses made in TBI software allowed to obtain bridges' accelerations, displacements and strains, as well as accelerations in train vehicles, in order to assess passenger comfort or stability of the carried load (in freight trains). Train speed ranges were assumed as 100 to 400 km/h and 80 to 220 km/h, respectively for the passenger and freight trains. The track irregularities were obtained based on records provided by the track inspection vehicle EM 120 from REFER. Fig. 4a illustrates the longitudinal levelling profiles of the left and right rail of the Côa bridge, in a track section between km +238.286 and km +238.253. These records consider the contributions related to wavelengths between 3 m and 70 m. The maximum amplitude equal to 12.7 mm appears essentially at the abutments of the bridge, especially in the Guarda's side. Fig. 4b illustrates two responses for the main arch of Côa bridge in terms of accelerations and displacements for the range of speed considered in the dynamic analysis due to the passing of Alfa Pendular train. The code-standard limit of 3.5 m/s^2, presented in Annex A2 of Eurocode 1990 [14] for the acceleration is exceeded in several locations, which means that is necessary to stablish a speed limit for train circulation. For the Durrães and PK124 bridges, considering the freight train action, there were no vertical accelerations exceeding the code-standard limit. The number of vibration modes change from bridge to bridge, considering modes up to a frequency of 30Hz. The time step of the analysis was equal to 0.001 s and the adopted values of the damping coefficients were equal to the average values of those obtained from an ambient vibration test [15].

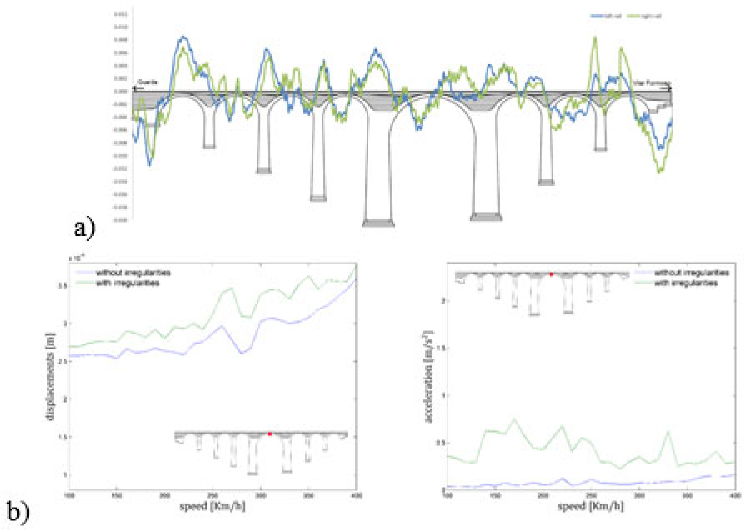


Fig. 4. *Numerical results: a) Longitudinal levelling profiles of the left and right rail of the Côa bridge; b) Vertical response at the centre of principal arch A5 in terms of displacements and accelerations.*

Regarding the effects on the studied vehicles, the freight one reached high acceleration values; however, since no information is available on code-standard limit acceleration values for freight vehicles to be compared with the obtained ones, no explicit conclusion can be drawn thereof. As for the passenger train in the Côa bridge, high accelerations are also obtained as shown in Fig. 5 such that, according to applicable code-standard limits of vertical acceleration in the passenger car body, for very good passenger comfort level the speed limit at which the train can run is 120 km/h, while for good and satisfactory comfort levels the speed limit is 160 and 240 km/h, respectively.

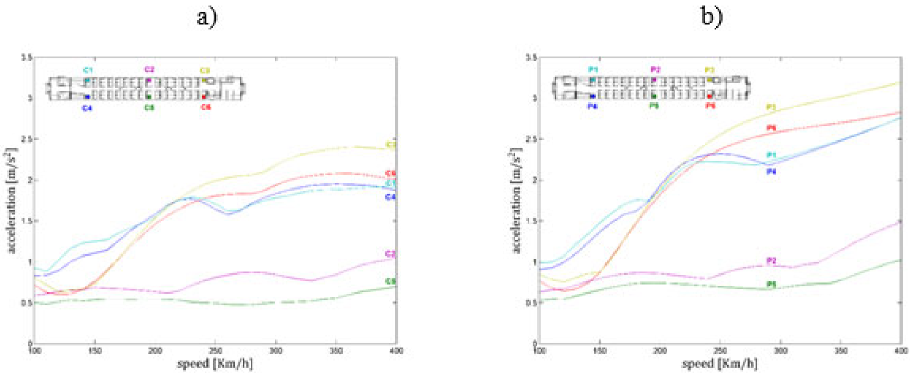
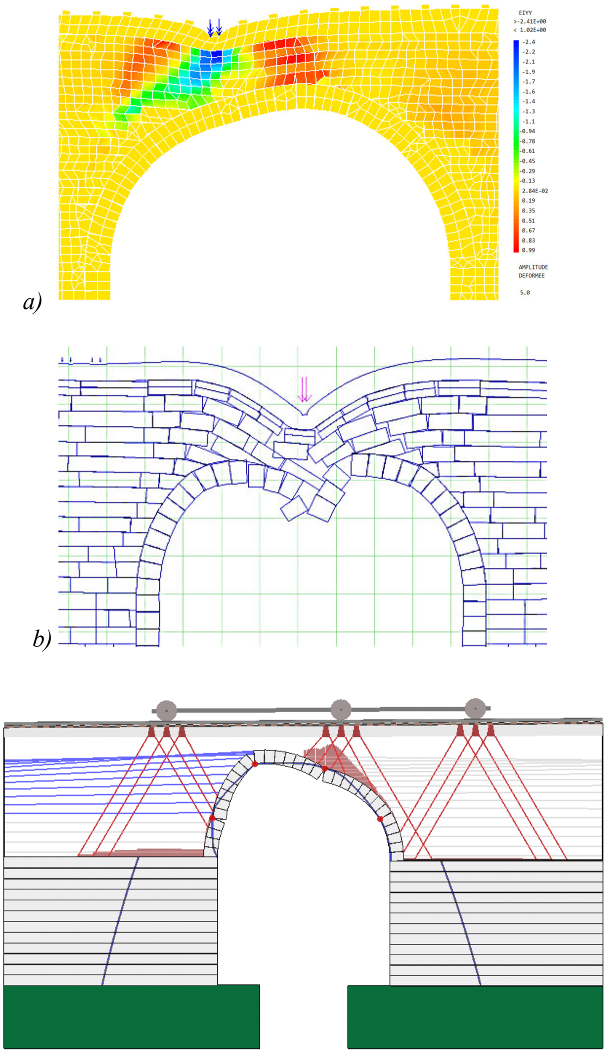


Fig. 5. *Maximum acceleration values, as function of speed, at 1st vehicle, with tack irregularities in: a) carbody; b) passenger*

3.2. Load carrying capacity

Another goal of the project consisted in evaluating the limit load which requires numerical modelling beyond the linear elastic behaviour. Therefore, due to computational and time limitations, this would not be feasible for the larger, Durrães and Côa, bridges and the natural option was to perform that evaluation only for the PK124 bridge. Even so, the model complexity was such that, non-linear numerical analysis modelling had to be done considering incremental static analysis using 2D or 3D DEM models and 2D FEM models. For this purpose, the most unfavourable train (passenger and freight) positions were obtained so as to induce the arch failure associated with the formation of hinge mechanisms, which was evaluated on the basis of global FEM model analysis under train moving loads. Then, for the same train positions, increasing load levels were considered in order to obtain the final collapse load. With the same purpose, RB models were analysed considering several load cases corresponding to different train positions along the bridge. The analyses of such models' response, throughout the incremental load history, allowed identifying the damage evolution in bridge models associated with masonry joint opening and sliding as well as infill material yielding.



a)

b)

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Fig. 6. *FE and DE detailed models of PK 124 bridge: a) FEM based CAST3M model; b) DEM based UDEC model; c) RB based RING model.*

It was found that very high values are required for the load factor of the nominal train loading in order to develop a collapse mechanism in the bridge. The analysis of the 3D models allowed evaluating the bridge response under the action of the freight train loading until the intensity level of 10 without the formation of any hinge in the arch. For the 2D DE models the maximum multiplier applied with the Alfa-pendular loading was 70 and for the freight train loading the value of the multiplier was 10.

4. CONCLUSIONS

This article focused on the numerical study aiming at assessing the structural response of stone arch bridges under traffic loads and based on realistic knowledge of the constituent material properties. For these basic purposes, a comprehensive experimental campaign, including laboratory and in-situ tests, has been performed on three bridges still operating in Portugal. FEM based continuous models were developed for the three bridges and both trains, one freight train and another passenger train considering linear elastic material behaviour. Detailed FEM and DEM modelling strategies are adopted for simulating the structural response of the smallest bridge of the three. Dynamic analyses were performed to evaluate the behaviour of both the bridges and the trains, due to the dynamic effects caused by interaction between them and by the present of track irregularities. To evaluate the limit loading applied on the PK124 bridge, a numerical modelling beyond the linear elastic behaviour was required. The dynamic analyses made in TBI software allowed to obtain bridges' accelerations, displacements and strains, as well as accelerations in train vehicles, in order to assess passenger comfort or stability of the carried load (in freight trains).

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