# Numerical modelling of a load test on a masonry arch bridge

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ABSTRACT: This paper deals with the numerical analysis of a masonry arch bridge. The bridge, dated from early 20th century, is made of limestone masonry and is located in the city centre of Leiria, Portugal. In the framework of a governmental urban rehabilitation programme, the part of the city involving the bridge is under rehabilitation works. Therefore, a detailed characterization of the bridge state was needed. The purpose of the modelling described in this paper is related to the simulation of three load tests carried out as well the assessment of the load carrying capacity of the bridge.

## 1 INTRODUCTION

The assessment of the condition of bridges is a major issue nowadays. This task is particularly difficult when dealing with historical masonry arch bridges due to their specificities as use of natural materials, lack of knowledge related to mechanical properties of materials and their large variability, existing damage caused by increasing traffic loads, aging and environmental factors and lack of maintenance.

A very important approach is the use of numerical models able to reproduce the structural response, both at serviceability and ultimate limit states. For that, several methods and computational tools are available. Different types of constitutive models originate a sequence of models, which allow the analysis to include more complex response effects. The most common idealizations of material behaviour are elastic behaviour, plastic behaviour and nonlinear behaviour.

In general, linear elastic analyses might not appropriate for masonry constructions, namely masonry arch bridges, however, in a first stage of analysis, the hypothesis of linear elastic behaviour can be of great help. A linear analysis requires few input data, being less demanding, in terms of computer resources and engineering time used, when compared with nonlinear methods. Moreover, for materials with low tensile strength, linear analysis can provide a reasonable description of the process leading to the crack pattern.

Plastic analysis, or limit analysis, is concerned with the evaluation of the maximum load that a structure can sustain. The assumption of plastic behaviour implies that, on one hand, the maximum load is obtained at failure and, on the other hand, the material should possess a ductile behaviour. Apparently, this last requirement seems to be unrealizable since the plastic deformations may exceed the ductility of the masonry. However, the limited ductility in compression does not play a relevant role as collapses are generally related to the low tensile strength (Croci, 1998). Thus, the assumption of a zero tensile strength renders the method of plastic analysis adequate for the analysis of masonry arch bridges.

Finally, nonlinear analysis is the most powerful method of analysis, the only one able to trace the complete structural response of a structure from the elastic range, through cracking and crushing, up to failure. Therefore, nonlinear analysis appears as the most adequate approach to be used in numerical simulations of masonry structures. However, its accuracy depends to a great extend on the availability of data required to define the advanced constitutive material laws. Furthermore, its use depends on which objectives are required from the analysis. If the sought information can be attained using a simpler method, which turns out to be less expensive or more in agreement with available data, then its use is advised.

This paper deals with the numerical analysis of a masonry arch bridge. Aiming at assessing its general condition, a set of structural and non-structural tests was performed, including load tests. The numerical modelling of the bridge was carried out in order to simulate the load tests carried out as well as to assess its load carrying capacity.

## 2 BRIDGE UNDER STUDY

### *2.1 General description*

The bridge under study is located in the city centre of Leiria across the Lis River, in Portugal, and was built in the year of 1904 resorting to locally available limestone, see Figure 1 and Figure 2(a). The bridge has a flat roadway, whereas the single arch is segmental-shaped and reaches a span of about 16 m, as schematically represented in Figure 2(b). Some decades after the construction, the local authorities carried out a roadway widening by means of a reinforced concrete slab supported directly on the infill. Currently this slab is in a very poor condition, where reinforcing steel corrosion and consequent concrete spalling can be clearly observed.

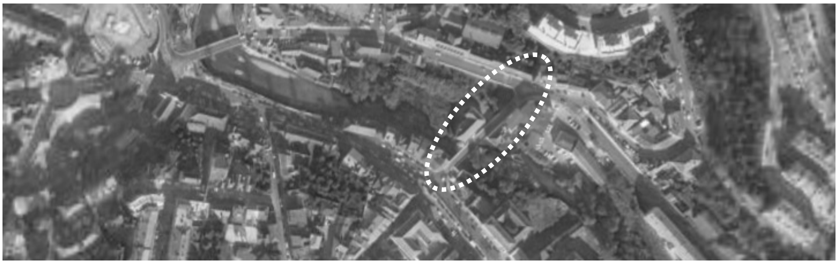
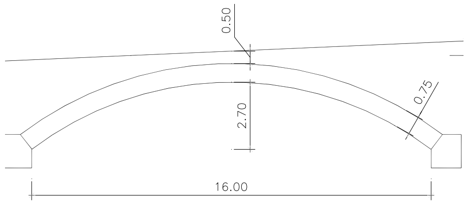


Figure 1 : Aerial view of the city centre, including the bridge.

(a)




(b)

Figure 2 : Elevation view (downstream side): (a) photo; (b) model.

In the framework of a governmental urban rehabilitation programme, the part of the city involving the bridge is under rehabilitation works. Therefore, a detailed characterization of the bridge condition was needed. To achieve such purpose, a set of tests was performed, being the most important a detailed visual survey, geotechnical probing, laboratory tests on limestone specimens, load tests on the bridge and numerical modelling aiming at both simulate the load tests and the evaluation of the bridge load carrying capacity.

### *2.2 Laboratory tests*

Eight limestone specimens were colleted from the bottom part of the bridge and tested under monotonic compressive loading. The results allowed to obtain an average Young's modulus of 57 GPa and an average compressive strength of about 60 MPa. These values allow asserting that the material is a hard limestone ($f_c > 30$ MPa), however they cannot be directly used in the numerical models because they are referred to limestone specimens only, whereas masonry behaves in a different way.

### *2.3 Load tests performed on the bridge*

In order to characterize the experimental static response of the bridge different load tests were performed. For that, two different trucks were used to generate the most unfavourable structural effects. The two trucks are schematically represented in Figure 3 and are indicated as truck 1 (398 kN) and truck 2 (426 kN), respectively.

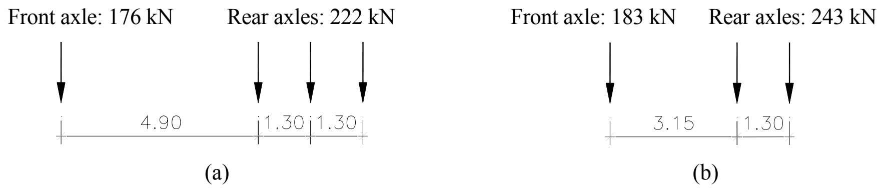


Figure 3 : Schematic representation of the two trucks used in the static tests: (a) truck 1; (b) truck 2.

The experimental testing of the bridge under static loading consisted of three different load arrangements, described as follows:

- *Test 1:* the two trucks were set facing the back parts (the distance between the rear axles of the two tracks was about 2,70 m) and centred in the middle of the bridge span, see Figure 4(a);
- *Test 2:* both trucks were set facing the back parts (again the distance between the rear axles of the two tracks was around 2,70 m) and now centred in the quarter span of the bridge close to the left shore, as illustrated in Figure 4(b);
- *Test 3:* the truck 1 was placed with the central rear axle centred in the quarter span of the bridge close to the right shore, see Figure 4(c).

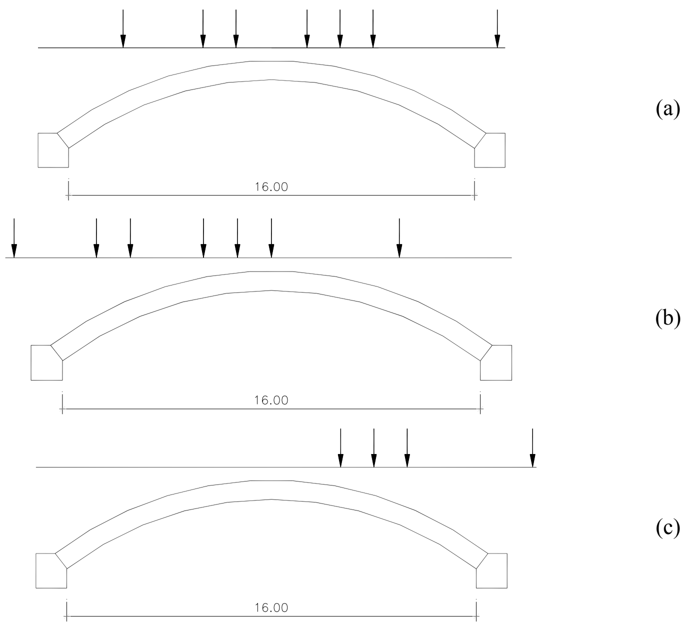


Figure 4 : Schematic representation of each static load test performed (upstream view): (a) test 1; (b) test 2; (c) test 3.

For the three tests several displacements were registered, being the most relevant the following:

- Displacement $d_1$: vertical arch displacement, measured at the quarter span close to the left shore;
- Displacement $d_2$: vertical arch displacement, measured at the crown. Two different vertical displacements were experimentally measured in this position (one at each side of the bridge) in order to assess the existence of torsion, if any. Here, the average experimental displacement was used to be compared against the numerical values;
- Displacement $d_3$: vertical arch displacement, measured at the quarter span (right shore).

Due to problems related to the scaffold installed on the bridge for the detailed survey, see also Figure 2(a), arch displacements were measured by means of LVDT's positioned in the ground. For that, suspended weights by means of steel wires fixed at the arch intrados and touching the LVDT's at the ground level were used. This procedure implied an error in the range of 0.03 to 0.05 mm. A higher precision would imply the use of a more complex and expensive setup and would be very difficult to reach in practice. The experimental displacements measured during testing of the bridge will be presented and compared against the numerical ones in the next section.

## 3 NUMERICAL ANALYSIS

### *3.1 Modelling strategy and properties*

The numerical modelling described in this paper has a double purpose. The simulation of the three load tests is sought but also the assessment of the bridge carrying capacity is intended. To reach these two objectives, two distinct models are proposed, namely a model based on the elastic behaviour to simulate the load tests and a plastic-based model to assess the load capacity. The main advantage of these two models over a unique model resorting to a nonlinear incremental analysis is the few input data required. This issue will be further discussed in the next sections.

### *3.2 Modelling of the load tests*

Aiming at reproducing the experimental results of the load tests, a finite element model was developed (DIANA, 2005) assuming the hypothesis of linear elastic behaviour of the materials. Despite the masonry, in general, exhibits a nonlinear behaviour for moderate load levels, the likely low stress and deformation levels induced to the bridge during the tests might allow to accept the assumption of elastic behaviour of the materials.

The bridge was modelled using eight-node continuum plane stress elements with Gauss integration. The structural elements considered are both the masonry arch and the spandrel walls, as shown in Figure 5. For low load levels, the spandrel can have an important influence on the structural behaviour of the bridge, while for higher load levels these effects are reduced due to the accumulated damage, which generates a lost of connection between the spandrel walls and the arch.

Based on holes performed in the infill material during the survey, it is believed that the slab, visible bellow the parapets, does not cover the entire width of the bridge. Therefore, and considering that no drawings were available, any possible structural effects related to the existence of the reinforced concrete slab were neglected.

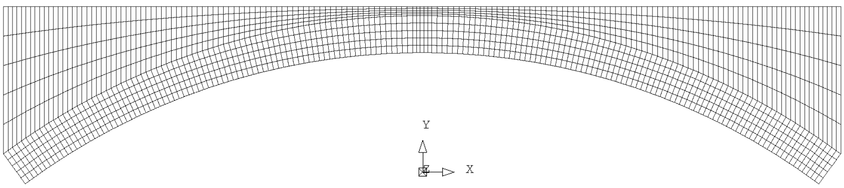



Figure 5 : Finite element mesh used to model the arch and spandrel walls.

As aforementioned, the experimental results on the limestone specimens cannot be directly used in the modeling of the masonry. In the absence of experimental results, the definition of the elastic properties was based on available values from similar masonry assemblages. In this way, the Young's modulus of the arch and the spandrel wall were assumed equal to 15 GPa and 3 GPa, respectively.

For the limestone masonry it was used a density of 25 kN/m$^3$ whereas for the infill material it was used a value of 20 kN/m$^3$. Since the infill was not explicitly modelled and considering that in masonry arch bridges the infill causes a redistribution of concentrated live loads applied on the top surface, it was assumed here that these loads are dispersed through the soil according the Boussinesq theory, for an angle of 30º.

The experimental displacements due to the three load tests are illustrated in Table 1. The analysis of figures shows that the maximum displacements are due to the load test 1, while for the load test 3 the displacements are quite close to the error range, see also Figure 4. In general, these displacements are relatively small, due to the high stiffness of the bridge.

Table 1 also includes the comparison between the numerical and experimental displacements for each load test performed on the bridge.

Table 1 : Experimental and numerical displacements for the three load tests.

| Test | Experimental [mm] | | | Numerical [mm] | | |
|---|---|---|---|---|---|---|
| | $d_1$ | $d_2$ | $d_3$ | $d_1$ | $d_2$ | $d_3$ |
| Test 1 | –0.14 | –0.27 | –0.18 | –0.162 | –0.201 | –0.134 |
| Test 2 | –0.10 | –0.12 | –0.10 | –0.155 | –0.148 | –0.075 |
| Test 3 | +0.03 | –0.03 | –0.04 | +0.104 | –0.046 | –0.214 |

(The sign "+" indicates the upward vertical displacement)

The maximum differences between experimental and numerical results obtained for the load tests 1 and 2, (respectively 0.07 mm and 0.06 mm) are very similar to the error associated to measurements and, therefore, can be considered acceptable. Moreover, the maximum difference observed for the load test 3 (0.17 mm) may not be considered representative because the experimental displacements for this load test are extremely low and most probably lower that the error involved in its measurement.

Figure 6 shows the minimum compressive stresses due to the dead weight of the bridge including also the weight of the infill, pavement, slab and parapets. With exception of the elastic peak stresses near the abutments, the maximum compressive strength is around 1.2 MPa, which can hence be accepted for this type of structures.

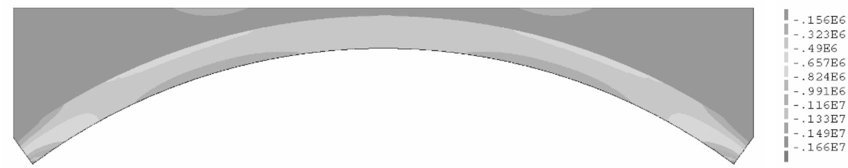



Figure 6 : Minimum compressive stresses depicted on the undeformed mesh for the dead weight of the bridge (values in [Pa]).

Considering the load tests, the minimum compressive stresses due to the dead weight of the bridge and each one of the load cases are represented in Figure 7. From these figures it is possible to visualize that the maximum compressive stresses have a magnitude of around 1.5 MPa (excluding the elastic peaks), which might originate nonlinear behaviour locally in a few sections. On the other hand, the maximum tensile stress is around 50 kPa (from load test 3), which makes the hypothesis of material elastic behaviour reasonable to be used within this numerical analysis.

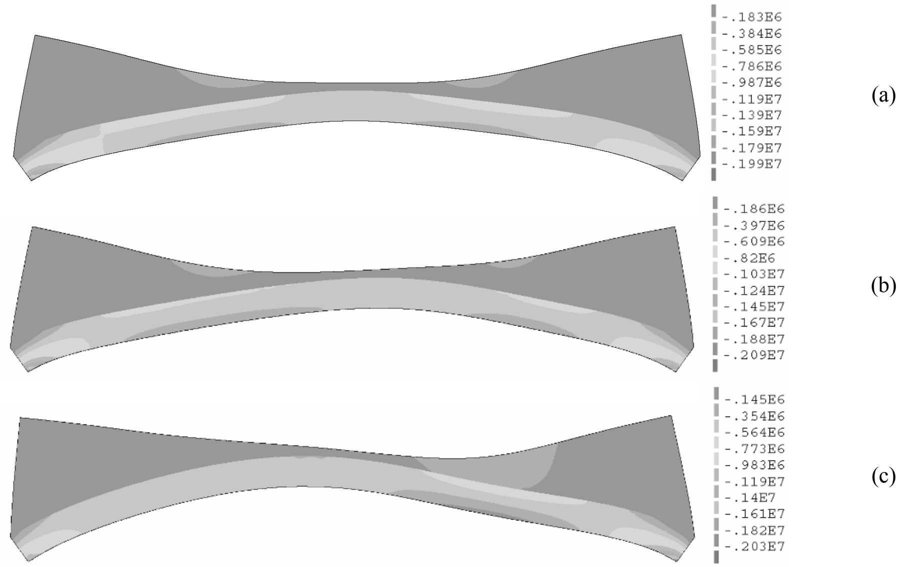



Figure 7 : Minimum compressive stresses (values in [Pa]) depicted on the incremental deformed mesh for the dead weight of the bridge and the three load tests: (a) test 1; (b) test 2; (c) test 3.

In global terms, the results achieved show that the numerical model provides displacements in agreement with those from experiments. To improve the agreement between numerical and experimental results, it would be necessary to resort to a more complex numerical model able to include the possible influence of the reinforced concrete slab and the effects of the infill. In addition, such level of accuracy could only be achieved by assessing the real mechanical properties of the materials involved (masonry, infill and slab).

### *3.3 Load carrying capacity*

Besides the simulation of the load tests, also a numerical assessment in terms of carrying capacity was required in order to assess the safety conditions of the bridge in order to be used by vehicles. The objective here is to provide a good estimation of the maximum load that the bridge can sustain prior to failure.

Among the available computational methods proposed in literature to compute the carrying capacity of masonry arch bridges, from hand-based methods to advanced nonlinear tools, the limit analysis method is the most generally applicable, see Livesley (1978) and Gilbert and Melbourne (1994) for further details. Within the limit analysis method, the load distribution is known but the load magnitude that the bridge can carry is unknown, but it can be computed. Therefore, limit analysis is a very practical tool since it only requires a reduced number of material parameters and it can provide a good insight into the failure pattern and limit load.

Here, the bridge was modelled as an in-plane single span segmental arch, see Figure 8. In the absence of in-situ test results, the material properties were considered to assume typical values found in similar structures (Oliveira and Lourenço, 2005). In particular, a value of 4 MPa was adopted for the masonry compressive strength (PIET, 1970), whereas for the horizontal passive pressure a conservative value equal to half of the classical value given by Rankine theory was used (Smith et al., 2004). The value selected for the compressive strength of masonry take into account the type of material (hard limestone) and the existence of joints between stones.

Besides the self-weight of the materials (masonry and fill), a rolling load composed by the Portuguese standard vehicle was considered (RSA, 1983). This standard vehicle is composed by three axles equally spaced by 1.50 m and with a 200 kN load per axle.

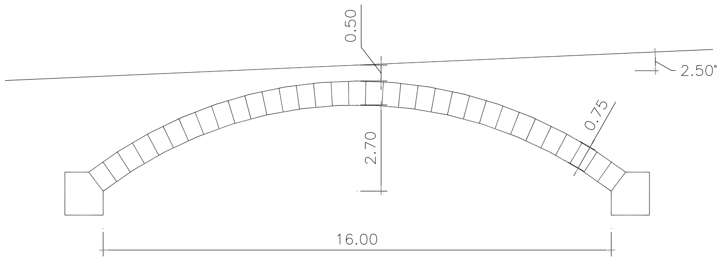



Figure 8 : Structural arch model (upstream view).

Using the computer program RING, developed within the rigid block limit analysis method (Gilbert, 2005), the minimum failure load factor was found to be equal to 4.47. This load factor was found for the vehicle central axle positioned at 31.9 % of the free span. Figure 9 illustrates the associated four hinges failure mechanism found, where both the dead and live load pressures applied to the arch, the hinges and the thrust-line are shown. The load factor obtained seems to indicate that the bridge can be safely crossed by traffic.

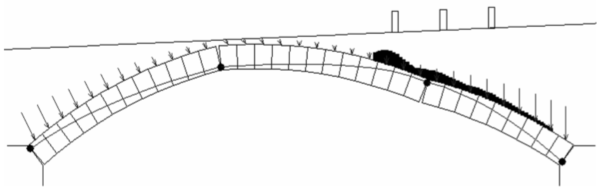


Figure 9 : Collapse mechanism for the minimum failure load.

## 4 CONCLUDING REMARKS

The numerical modelling of historical masonry arch bridges is a particularly difficult task, mainly due to lack of knowledge related to mechanical properties of materials, which renders very difficult the use of advanced nonlinear constitutive models, unless an experimental program is carried out.

The bridge numerical modelling described in this paper aims at a double purpose: the simulation of load tests and the assessment of its carrying capacity. To accomplish both of the objectives, two different models were used, namely a model based on an elastic analysis to simulate the load tests and a plastic-based model to assess the ultimate load capacity. The main advantages of these two models over a unique model based on a nonlinear analysis are the few input data required and the good insight they can provide.

Concerning the simulation of the load tests, the comparison in terms of displacements at three control sections shows an agreement between experimental and numerical results. Furthermore, the stress levels reached show a low to moderate minimum compressive stress (1.5 MPa) and a very low maximum tensile stress (50 kPa), which makes the hypothesis of material elastic behaviour reasonable to be used within this numerical analysis.

The appraisal of the bridge carrying capacity for the Portuguese standard vehicle using a computer program based on the limit analysis allowed reaching a load factor equal to 4.47. This higher load factor seems to indicate that the bridge can be safely crossed by traffic.

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