Experimental and numerical investigations of a segmental masonry arch subjected to horizontal settlements

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Abstract. Settlements represent one of the main causes of collapse in arched structures. The stiffness degradation and strength reduction associated with these slow long-term movements are reflected in changes of the dynamic properties of the system (frequencies, mode shapes and damping ratios). The correct dynamic characterization of the nonlinear behaviour of masonry arches undergoing supports settlements is therefore fundamental to timely detect the occurrence of serious damage mechanisms and prevent unexpected failures.
This paper aims to investigate the dynamic behaviour of a segmental masonry arch subjected to stepwise horizontal displacements of one support. To this end, output-only dynamic identification techniques are first used to estimate the experimental dynamic properties of the system under progressive damage scenarios. Then, a numerical procedure coupling linear perturbation and modal analysis is applied to evaluate frequencies and mode shapes of the arch, taking into account the actual crack distribution induced by the settlements. The combination of experimental and numerical investigations allows to explore in detail the dynamics of the settled masonry arch as well as its potential collapse mechanism.

Keywords: Masonry Arches, Support Settlements, Damage Identification, Modal Analysis, Linear Perturbation

1 Introduction

Masonry arches are among the most used structural and architectural systems in many heritage buildings and bridges, owing to their capacity to carry high values of gravity loads. Because of this wide use, over the last decades many research works have been devoted to investigating the mechanics of such a basic structural system [1-3]. Yet, not all the aspects related to its structural behavior have been completely explored. For instance, many uncertainties arise about the effects of support displacements, particularly in what concerns the dynamic behavior of masonry arches in the presence of settlement-induced cracks. Either due to movements of the underlying structures or

to subsidence of the foundation system, settlements represent one of the main causes of collapse in arched structures. Thus, the correct dynamic characterization of the nonlinear behavior of masonry arches undergoing this type of phenomena is fundamental to timely detect the stiffness degradation and consequent strength reduction resulting from the gradual progression of support displacements, thereby preventing local damages and unexpected failures.
Most of literature studies addressing this topic focus on semicircular or pointed arches subjected to displacement of both supports [4-6], but little attention is paid to segmental arches, whose shape makes them prone to horizontal settlements because of the higher thrust. Moreover, to rigorously investigate this type of problems and to obtain good predictions of the actual dynamic behavior of settled masonry arches, a numerical strategy able to take into account the non-linearity of the masonry material is needed. On the other hand, experimental methods represent a powerful technique of analysis to simulate the studied phenomena and to provide reference information for calibrating and verifying the reliability of the adopted numerical model. For all these reasons, this paper aims at exploring the three-dimensional behavior of a segmental masonry arch subjected to horizontal settlements of one support by combining experimental and numerical investigations: the former for the direct estimation of the system's dynamic properties under progressive support sliding; the latter for the correct reproduction of the observed arch dynamics taking into account the actual crack distribution and non-linear behavior of the materials.

2 Experimental test

2.1 Description of the arch mock-up and testing campaign

Built of brick units of 100x75x50 mm^3 laid with lime mortar, the segmental arch investigated spans 1900 mm, has a springing angle of 40°, a rise of 430 mm and a thickness of 75 mm. The specimen rested on two concrete abutments, whereof the left one bolted to the floor and the right one free to slide horizontally. Aiming at replicating the weight of spandrels and infill material of typical masonry arch bridges, two lime bags of 25 kg each were symmetrically placed on the arch backs. Fig. 1 shows the geometry of the arch along with the test layout.

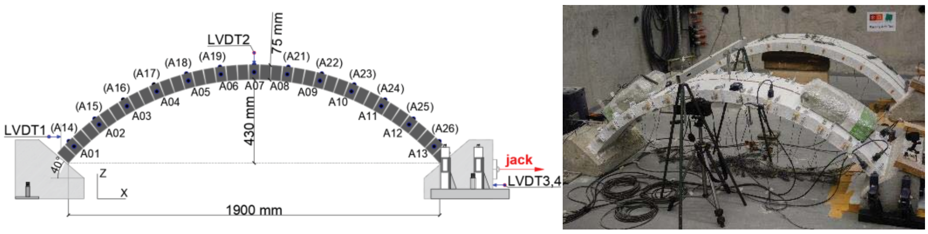

Fig. 1. Masonry arch geometry and test layout
(front side sensors: A01-A13; back side sensors: A14-A26).

To investigate the effects of support settlements on the arch dynamic behaviour, a five-stage displacement-controlled test was carried out at the IB-S laboratory of the University of Minho (Guimarães, Portugal) to induce progressive and not recoverable Damage Scenarios (DSs). Outward uniform displacement rates in horizontal direction were imposed to the right abutment of the arch by means of a hydraulic jack fixed to the concrete support. Five Linear Variable Displacement Transducers (LVDTs) were used to control the displacements throughout the test: one LVDT was employed to monitor the jack movement, two LVDTs were used to control the sliding of the support and additional two LVDTs were placed at the keystone and left springer of the arch to measure the specimen response.
The support displacement was increased stage by stage up to reaching a final value of 2.6 mm. As expected, three cracks crossing the entire arch width progressively appeared during the test: the first crack c_1 was located at the intrados, near sensors A05 (front side) and A18 (back side); while cracks c_2 and c_3 appeared at the extrados of the left (fixed support) and right (moving support) skewbacks, respectively. The final crack pattern is displayed in Fig. 2.

Settlements at the support of an arch are responsible for the formation of three crack hinges [1,6]. Their position depends on both the arch geometry and the homogeneous distribution of the mechanical properties across the structure, where the latter is largely influenced by the building process itself. Indeed, for the present case, the final asymmetrical crack configuration was visibly due to a reduced brick-mortar interface in the left region of the keystone.

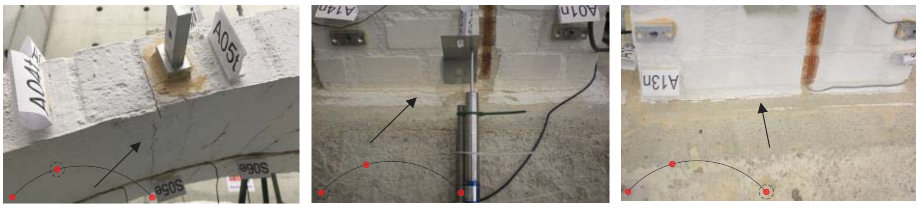

Fig. 2. Crack pattern of the arch after horizontal settlement of the right support.

2.2 Dynamic identification for the reference scenarios

Before applying the horizontal settlement at the support, the vibration response of the system was recorded with the purpose of identifying its modal properties in the absence of structural damage. Based on the results of a preliminary numerical analysis, the dynamic response of the mock-up was acquired with a sampling frequency of 400 Hz for a minimum duration of 180 seconds in both normal and tangential directions using high-sensitivity accelerometers evenly deployed in 26 points along the front and back edges of the arch (Fig. 1), for a total of 52 accelerations. Such a sensors density enabled to obtain a very good resolution in terms of mode shapes. Table 1 summarizes the results obtained for the undamaged configuration of the arch: Reference Scenario, without (RS) and with (RSW) additional weight on the backs. The estimation of the modal parameters was performed via the SSI-UPCX technique implemented in ARTeMIS Modal. Fig. 3 shows the first three modes shapes identified for the RSW.

Table 1. Frequencies *f* and damping ratios ξ of the arch for RS and RSW. MAC values are calculated between corresponding mode shapes (torsional modes are highlighted in grey).

Mode	Prevalent direction	f_{RS} [Hz]	f_{RSW} [Hz]	ξ_{RS} [%]	ξ_{RSW} [%]	MAC
1	X, asym	37.03	30.06	0.95	1.26	1.00
2	T, sym	58.64	50.95	0.88	3.06	0.70
3	Z, sym	63.56	59.44	1.02	1.22	0.93
4	Z, asym	100.76	95.23	0.97	2.12	0.90
5	T, asym	122.67	120.62	1.25	2.97	0.71
6	Z, sym	125.06	–	1.03	–	–
7	T, asym	146.21	134.02	1.46	4.19	0.67

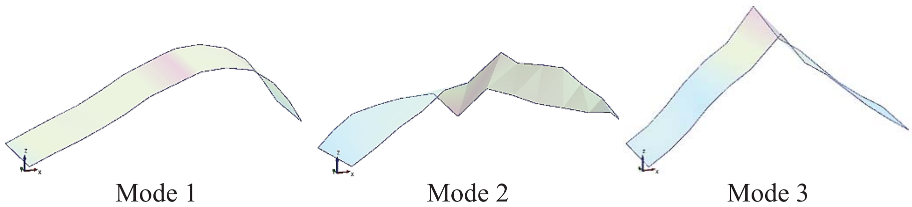

Fig. 3. First three mode shapes of the masonry arch for the reference scenario RSW.

2.3 Damage identification with increasing horizontal settlements

In order to follow the evolution of the modal properties of the arch with progressive settlements, ambient vibration tests (AVTs) were also conducted after each displacement stage. As the support slid, damage started to progress with consequent degradation of the structural stiffness. This phenomenon led to considerable changes in the arch eigenfrequencies, recording decreases up to 36% and 22% for modes 1 and 4 (Table 2). Significant frequency drops are observed for DS1, meaning that the incipient horizontal settlement (0.6 mm) soundly affected the dynamic behaviour of the arch since the early stages. The comparison between mode shapes of undamaged (RSW) and final damaged (DS5) scenarios depicted in Fig. 4 allows to visually perceive the presence of local protuberances in the modal response of the arch due to the occurred damage mechanisms. In the same figure, the MAC values between corresponding mode shapes are also reported.

Table 2. Frequency variation for the identified modes over progressive damage scenarios.

Scenario	Frequencies [Hz]					
	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6
RSW	30.06	50.95	59.44	95.23	120.62	134.02
DS1	26.31	48.04	59.04	80.47	113.16	133.45
DS3	21.44	45.14	58.27	74.97	118.71	128.84
DS5	19.16	43.19	57.54	74.09	115.78	129.00
Δf [%]	-36.26	-15.23	-3.20	-22.20	-4.01	-3.75

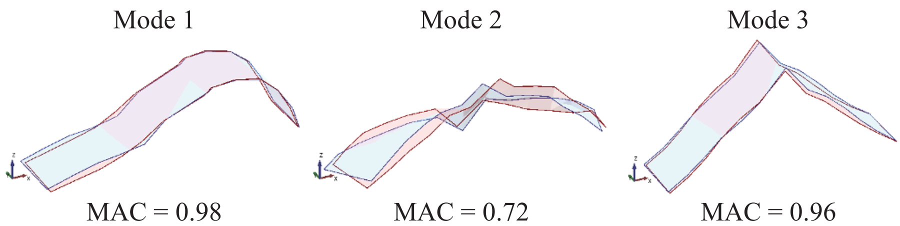

Fig. 4. Mode shapes comparison between reference (red) and last (blue) damage scenarios for the first three vibration modes of the arch.

3 Numerical analysis

3.1 Finite element modelling and assumptions

Standard model updating procedures permit to evaluate the mechanical characteristics of the materials constituting a structure. However, the numerical procedures employed in the context of linear elasticity are unsuitable to update the parameters of masonry structures, which exhibit a nonlinear behavior and show cracks due to permanent and / or accidental loads. For these reasons, their dynamic behavior should be evaluated considering the presence of the existing crack pattern. To this end, the FE NOSA-ITACA code, developed by the Mechanics of Materials and Structures Laboratory of ISTI-CNR, adopts a numerical method based on linear perturbation and modal analysis, which takes into account the influence of existing damage on the dynamic properties of masonry buildings. NOSA-ITACA adopts the constitutive equation of *masonry-like* materials and masonry is modeled as a nonlinear elastic isotropic material with zero or weak tensile strength and infinite or bounded compressive strength [7]. The procedure to evaluate the dynamic properties of masonry structures in the presence of cracks, based on linear perturbation and modal analysis, consists of two fundamental steps: 1) solution of the nonlinear equilibrium problem of the masonry structure subjected to external load, and calculation of the tangent stiffness matrix K_T to be used in the next step; 2) solution of the generalized eigenvalue problem underlying the modal analysis, where the matrix K_T is used in the place of the elastic stiffness matrix K, and estimation of the natural frequencies of the structure in the presence of cracks [8, 9].

3.2 Modal analysis with linear perturbation

The experimental specimen described in Section 2.1 has been modeled in the NOSA-ITACA code, using 53496 8-node isoparametric brick elements (element 8 of the NOSA-ITACA library, www.nosaitaca.it) and 64411 nodes for a total of 193233 degrees of freedom. Fig. 5 shows the mesh generated by the code, where the individual masonry components, bricks and mortar joints, are highlighted in cyan and red, respectively.

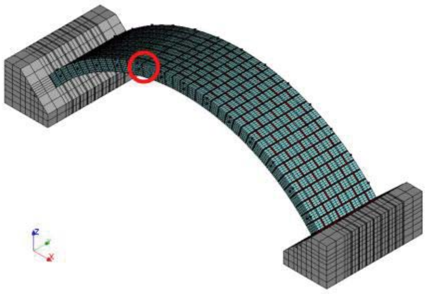

Fig. 5. Mesh model of the experimental specimen generated by NOSA-ITACA.

A preliminary standard model updating has been performed on the FE model with the aim of estimating the Young's moduli E of the two masonry components, by using the first three experimental frequencies and mode shapes of the undamaged structure, assumed as reference scenario (RS). The analysis is carried out considering the base of the concrete supports clumped and adding additional masses to the black nodes of Fig. 5 to take the weight of the testing equipment into account. The mechanical characteristics of concrete, bricks and mortar are fixed and reported in Table 3:

Table 3. Mechanical characteristics adopted for the materials in the numerical model.

	E[GPa]	ν	ρ [kg/m^3]	σt [MPa]
Brick	2.5	0.2	1653.3	0.50
Mortar	0.5	0.2	1750.0	0.37
Concrete	25.0	0.2	2500.0	--

The Young's moduli obtained via model updating and shown in Table 3 are consistent with the values reported in literature [10]. Table 4 summarizes both experimental f_{exp} and numerical frequencies f_N – the latter estimated using the parameters provided in Table 3 – as well as the MAC values between experimental and numerical mode shapes.

Table 4. Comparison between the first 3 experimental and numerical modes of vibration for the reference scenario RS.

	f_{exp} [Hz]	f_N [Hz]	$\|\Delta f\|$ [%]	MAC
Mode 1	37.03	36.55	1.30	0.94
Mode 2	58.64	55.57	5.23	0.72
Mode 3	63.56	65.87	3.63	0.81

Subsequently, a linear perturbation analysis followed by modal analysis has been performed via NOSA-ITACA, modeling concrete as a linear elastic material, bricks and mortar as *masonry-like* materials. A further model updating is conducted by keeping the values of E, ν and ρ fixed (Table 3) and varying the tensile strength σt. In order to numerically reproduce the experimental damage scenario along with its evolution, the tensile strength of the mortar joint circled in red in Fig. 5 has been reduced by about 80%. Except for RS and RSW scenarios, each analysis has been conducted by imposing the same horizontal displacements (x direction) recorded during the experimental test to all nodes belonging to the base of the right abutment.

Fig. 6 shows the trend of the first frequency vs. the right abutment settlement in the experimental and numerical cases.

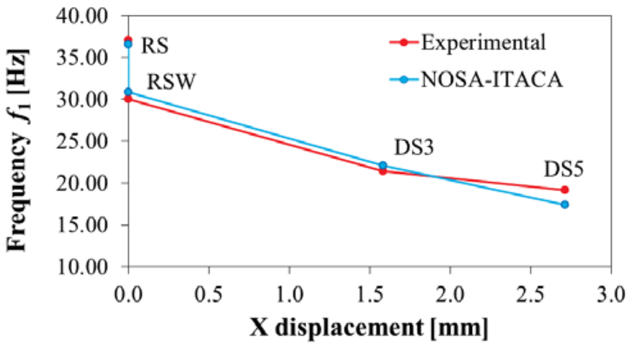

Fig. 6. First frequency versus abutment settlement.

A quick insight into the changes that a 2.6 mm horizontal settlement caused to the masonry arch in terms of frequencies and mode shapes is given in Table 5. For the sake of brevity, only the experimental and numerical frequencies and mode shapes of the first vibration mode are reported for the different scenarios. The relative frequency error is about ±2.5%, apart from damage scenario DS5 where it reaches around 9%. MAC values greater than 0.94 ensure that the adopted procedure is able to follow the damage evolution and the consequent change in the structure’s stiffness.

Fig. 7 displays the first experimental and numerical mode shape of the segmental arch corresponding to the final damage scenario (DS5), where the position of the hinges due to the increased support settlement is clearly identified.

Table 5. Comparison between experimental and numerical results of the first vibration mode for the various scenarios.

	f_{exp} [Hz]	f_N [Hz]	$\lvert\Delta f\rvert$ [%]	MAC
RS	37.03	36.55	1.30	0.94
RSW	30.06	30.84	2.59	0.95
DS3	21.44	22.13	3.21	0.97
DS5	19.16	17.44	8.98	0.98

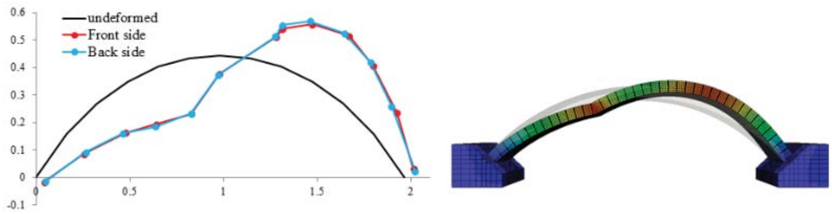

Fig. 7. First experimental (left) and numerical (right) mode shape of the arch for DS5.

4 Conclusions

This paper investigates the dynamic behavior of a segmental masonry arch subjected to horizontal settlement of one support. The problem is analyzed both experimentally and numerically with a two-fold objective: follow the evolution of the dynamic properties of the system under progressive settlement-induced damage scenarios; and obtain good simulations of the actual dynamic behavior of the settled arch taking into account the non-linearity of the constituent materials. The comparison between experimental and numerical results allows to shed light on the effects of support sliding on the vibration response of the segmental masonry arch and demonstrates the adequacy and reliability of the linear perturbation approach to estimate the modal properties of masonry structures in the presence of cracks.

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