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ANALYSIS OF STRUCTURAL BEHAVIOUR OF HISTORICAL STONE ARCHES AND VAULTS: EXPERIMENTAL TESTS AND NUMERICAL ANALYSES

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ABSTRACT

Recent seismic events showed the dramatic need, especially in case of historical and existing buildings, of important strengthening activities to be carried out. In order to properly design them, a careful assessment of real structural behaviour and load-carrying capacity of these buildings is strongly required. This is particularly important when dealing with constructions made of heterogeneous materials like masonry or stonework, where often conventional analysis techniques do not behave satisfactorily. The behaviour of historic stonework, in fact, depends from geometry of structure, geometry and strength of stone, boundary conditions and interface law between mortar and stone. In this perspective, experimental activities are also necessary.

This paper presents the first results of an extensive experimental and numerical investigation on historical stone arches and vaults. A series of in-situ tests were carried out on different types of stone arches belonging to a large building of the XIX century, with the purpose of investigating their mechanical response and obtaining the structural behaviour of stonework under different types of in-plane loads. The arches were tested with symmetric static and dynamic load and then with asymmetric static load, in order to evaluate the stiffness and frequencies of vibration of the structure. A number of reconstructed stonework walls were also tested under compression in order to define the stone masonry compressive strength. The experimental structural results were compared with the numerical solutions obtained by a detailed finite element model. Numerical linear and non-linear FE analyses were conducted in order to reproduce the experimental tests and analyse the interaction between series of arches that are linked by cross vault or tunnel vault. Finally, nonlinear analyses with vertical and horizontal loads were carried out with the scope of simulating the seismic effect and to verify the behaviour of this type of vaulted structures under one direction in-plane loads. In these numerical simulations the modelling of the non-linear material behaviour is carried out on macro-level.

Keywords: *Arch, Historic stonework, Non-linear analysis, Experimental test, Cross vault*

1. INTRODUCTION

Masonry is the most popular building material used in Italy; as a matter of fact, the majority of Italian building heritage is made of it. The percentage significantly increases only considering historical buildings built several centuries ago. The fundamental characteristic of this material mainly lies in its handmade construction process, which allows its extreme versatility and its good operating speed. As historical buildings still standing show, masonry turns out to be also a very flexible and adaptable system, which is composed of a rigid skeleton, the blocks, and of a deformable binder, the mortar. This characteristic is very important, especially if the ductility capacity, which a seismic event requires, is considered [1]. Therefore, it is important to know the real building capacities, both concerning its resistance and ductility, in order to suggest an effective strengthening and improving

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intervention. Starting from the results of experimental tests, carried out both in situ and in laboratory, this research aims to assess and analyze the structural behaviour of a series of stone arches, connected by means of brickwork vaults and forming an extended complex, located in Trieste (Italy) and known as “ex-Silos storehouses". This complex, built in the XIX century and used to collect and store wheat in the past, is in an awful state of preservation and in an advanced state of deterioration. Series of experimental load tests were conducted in order to assess the real situation of the complex and to judge its actual safety, thus allowing for a fine tuning of the strengthening and rehabilitation intervention. Specimens of masonry were collected in-situ in order to test their mechanical properties. Finite elements models were developed for the purpose of interpreting the experimental tests and of studying the arches interaction. Finally, by means of these models, non-linear analyses were conducted in order to estimate the real ductility capacity of arches because of horizontal loads.

2. THE HISTORICAL EVENTS AND GEOMETRY ACQUISITION

The topic of this study is a group of buildings known as “ex-Silos storehouses”, whose history is strictly linked to the laying of Southern Railway of the Hapsburg Empire. On 27 July 1857, in the presence of the emperor Francesco Giuseppe there was the opening ceremony of the last stretch of the railway, linking Ljubljana to Trieste and achieving the whole Southern Railway. After few years, the new station turned out to be insufficient and unsuitable in developing Trieste’s trades. So, eight years since the opening ceremony, the station showed all its faults, which could be corrected, lowering the square from 10.12 metres altitude to seashores level of 3.16 metres. The new station was inaugurated and opened on 19 June 1878. In the meanwhile, the square station was lowered again. In 1883, the new station was made. Tax free port, which was in force since 1719 by permission of the emperor Charles VI, was however abolished in 1891. In the meanwhile, with the construction of the buildings of the new harbour, the “Silos” of the old railway station were progressively abandoned too. The station storehouses were scarcely used and quickly abandoned and forgot. Two fires damage the head ”Silos” building respectively in 1970 and 1971, and in 1975, in occasion of a national announcement of competition, the same part, come out in a new requalification project of the area, for a possible transformation in an auditorium.
A survey of the entire storehouses shows that the structural and geometric characteristics of the two buildings, making up the complex, are similar; as a matter of fact, their plant dimensions are 250 × 27-28 metres, arranged in parallel, with a courtyard of similar length and whose width is approximately 27 metres [2]. The buildings have two floors with a total height of 21 metres. The structure bearing the first floor was built with bearing perimeter walls and interior intermediate pillars arranged on a regular spacing of about 6.50 metres with above a longitudinal arch. The vertical load-bearing walls are made of squared limestone with joints of poor hydrated lime mortar; moreover, on the façade, there are pilasters of stone both externally and in the inner courtyard. The thickness of the perimeter walls on the ground floor is approximately 1.60 metres. On both buildings at the first floor, about 8 metres from ground level, there is a vaulted floor. This is surrounded by the transverse arches, by the central longitudinal arch and by the side walls of the façade. The central arches have a free span of about 5 metres, a width of about 2 metres like that of bearing columns. On the contrary, the transverse arches of the buildings have a spacing of 6.5 metres, a free span of about 10-11 metres, key brick height between 6 and 6.5 metres, starting between 2 and 2.5 metres from the ground.

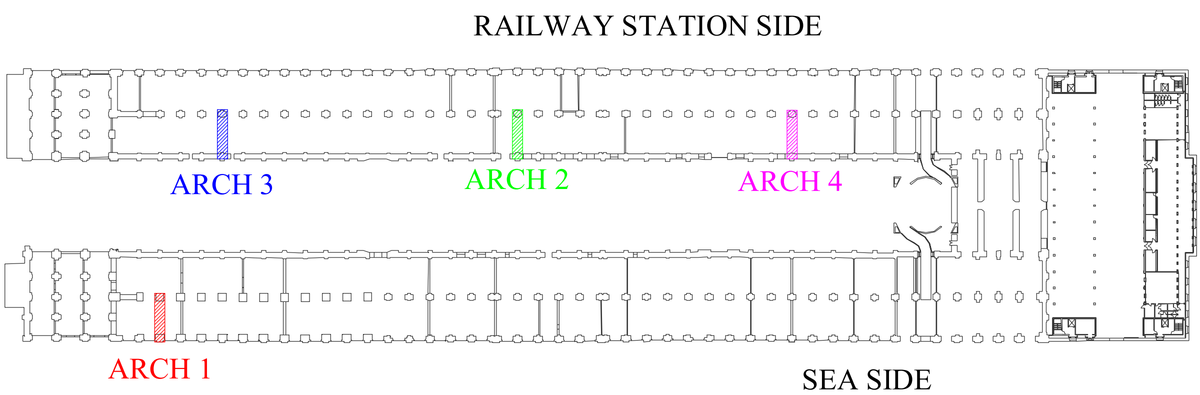

Fig. 1 Plant of the ground floor of the storehouses with tested arches individualizes

Arches introduce considerable rigidity to the structure, connecting the central columns with the perimeter walls and bearing both vaults and the upper floors. The brickwork vaults have a constant thickness of about 40-45 centimetres and are plastered both in the intrados and the extrados. They are tunnel vaults, except for the last three bays per building, where there are cross vaults. The bricks used in their construction have high mechanical properties and are interposed by lime mortar joints of good texture. On the first floor, vaults bear the load of the wooden floor, made with beams which have spacing of about 1 metre, a wooden plank and sometimes an overlying slab in concrete.

3. THE EXPERIMENTAL TESTS

Due to the large number of arches of the buildings, four of them were chosen as representatives of all existing types. Fig. 1 shows a plant of the complex with the arches tested highlighted. In order to determine the mechanical characteristics of the perimeter walls, a series of compression tests on portions of stonework, have been performed by the Laboratory LISG of University of Bologna, on specimens with dimensions of 60 × 60 × 130 cm (width × thickness × height). Four were extracted from existing perimeter walls, while two further specimens have been reconstructed in the laboratory

Table 1 Identification code, type, dimensions, specific gravity, ultimate strength, ultimate compression stress and elastic modulus of stonework specimens tested

Panel ID	Type of specimen	L [mm]	B [mm]	H [mm]	γ [kN/m^3]	P_u [kN]	f_{cb} [MPa]	E [MPa]
1	Extracted	400	460	820	–	494	2.68	273
2	Extracted	623	641	1105	24.18	1613	4.04	320
3	Extracted	657	634	1330	24.12	1554	3.73	488
4	Extracted	655	620	1225	25.33	1044	2.57	255
5	Reconstructed	629	588	1165	26.49	1875	5.08	262
6	Reconstructed	595	604	1209	25.41	1662	4.63	449

Table 2 Identification code, diameter, height, specific gravity, ultimate compression stress of stone core specimens tested

Core sample ID	∅ [mm]	H [mm]	γ [kN/m^3]	f_{cp} [MPa]	$f_{cp,average}$ [MPa]
1 - 1	54.16	54.27	25.86	161.4	158.1
1 - 2	54.06	54.30	26.01	154.6	
1 - 3	54.40	54.18	25.68	158.3	
2 - 1	54.46	54.10	25.54	156.2	160.2
2 - 2	54.34	53.81	25.75	166.2	
2 - 3	54.19	54.19	25.67	158.2	
3 - 1	54.32	54.25	24.81	102.0	94.4
3 - 2	54.13	54.10	24.76	91.6	
3 - 3	54.12	53.88	24.74	89.5	
4 - 1	54.22	54.27	25.64	134.2	137.2
4 - 2	54.19	53.25	25.69	130.1	
4 - 3	54.24	53.63	25.46	147.3	
5 - 1	54.51	54.55	25.11	133.6	152.8
5 - 2	54.28	54.44	25.56	172.0	

[3, 4]. Then, additional compression tests on five cylindrical core samples obtained by drilling of stone blocks in-situ, have been conducted. These tests allowed estimating the mechanical properties of stonework and stone blocks, which are summarized below (Tab. 1 and Tab. 2). Poor performances of stonework are mainly due to large thickness of mortar layers and to its limited mechanical strength.
The load tests, carried out in-situ on four representative transverse arches, were basically of three types: static test with symmetric in plane load, static test with asymmetric in plane load and dynamic test. The symmetric static tests consist in applying two increasing forces to the quarters of each arch, while in asymmetric tests the load was applied to one quarter of free span only. A properly designed steel reaction system allowed performing static load tests by pulling the arches from below. During the load tests, vertical and horizontal displacements in representative points were recorded. The instruments position is shown in (Fig. 2). A Wild optical level with sensitivity of 0.01 millimetres was used in order to measure the arches vertical deflection in position indicated by L1 – L5 of (Fig. 2). A wire displacement transducer was used in order to confirm the outcomes of mid-span deflection from optical level readings.

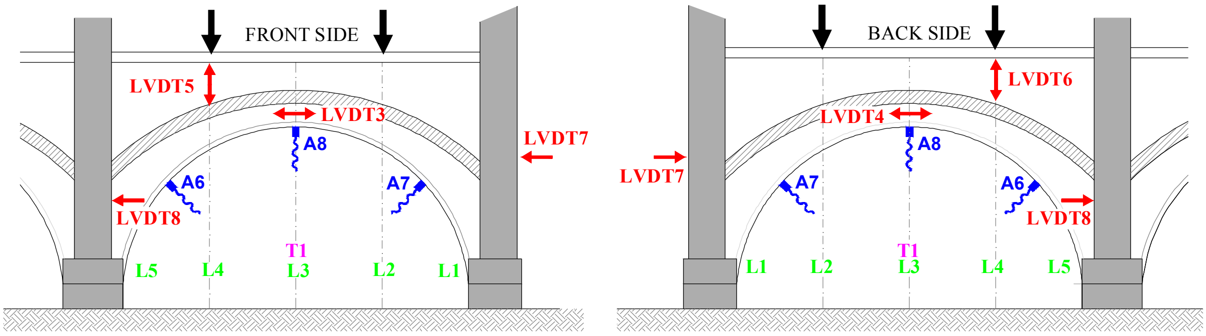

Fig. 2 Configuration of instruments utilized during tests

Other six displacement inductive transducers (LVDT – Linear Variable Differential Transformer) with sensitivity of 0.001 millimetres were used for measuring the horizontal displacements of wall-arch connection line (LVDT7 and LVDT8) and obtain mean strains key position (LVDT3 and LVDT4). In (Fig. 2) inductive transducers can be distinguished in red (LVDT 3 – LVDT 8), the graduated rod in green (L1-L5) and the wire transducer in magenta (T1). Dynamic load tests were also carried out, in order to evaluate the natural frequencies of arches. For these tests, a weight was hung on the arch intrados by means of a harmonic metal wire. Then, the weight was instantaneously released. The accelerograms shown by the building immediately after the impulse were recorded using three piezoelectric accelerometers, set on the arch intrados. The post-processing of these three accelerograms allows calculating the power spectral density of the signal, and at the end, to obtain an estimation of natural frequencies. In (Fig. 2) the accelerometers can be distinguished in blue (A6-A8). Tab. 3 reports a list of the tests carried out for each arch, indicating the maximum load achieved during the static tests, the weight released during the dynamic tests and the first two natural frequencies resulting from the post-processing of the recorded accelerograms.

Table 3 Summary of tests carried out for each one of selected arch

Arch ID	Maximum static load in symmetric test [kN]	Maximum static load in asymmetric test [kN]	Weight released in dynamic test [kN]	f_1 [Hz]	f_2 [Hz]
1	250	–	2,5	9.0	15.8
2	250	250	2,5	15.6	19.5
3	350	350	2,5	12.1	17.9
4	350	350	2,5	15.0	18.6

The results of the tests carried out on arch 4 are extensively discuss here as an example, considering that trends for the other arches are quite similar. Fig. 3 shows the vertical displacement of the arch for three symmetrical loads values (150 kN, 250 kN and 350 kN). As expected a symmetrical behaviour is observed with vertical displacements regularly increasing towards mid-span. Fig. 4 shows horizontal average strains evolution with applied symmetric load, calculated by LVDT3 and LVDT4.

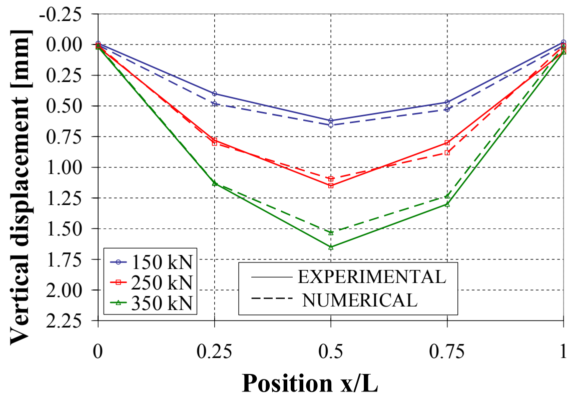

Fig. 3 Vertical displacement of arch 4 for static symmetric load test obtained by optical level

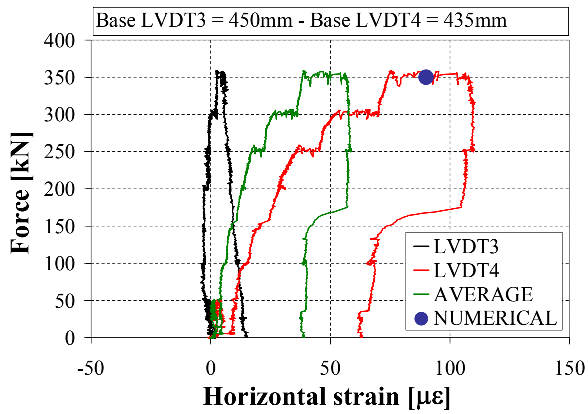

Fig. 4 Horizontal strain at key brick position calculated during static symmetric load test of arch 4

As regards the asymmetric static load tests, the (Fig. 5) shows how the asymmetric load shifts the maximum value of the vertical displacement in correspondence of loaded quarter. In term of vertical displacements, the results obtained by means of the optical instrument match those obtained by the wire transducer (Fig. 6). Fig. 7 shows horizontal average strains evolution with applied asymmetric load, calculated by LVDT3 and LVDT4. Finally, in (Fig. 8) horizontal displacements of column and lateral wall are reported for asymmetric load case; both LVDT7 and LVDT8 registered a negative displacement; this means that arch toes drifted towards the inner part of the building of about 0.1 mm. On the contrary, in previous case with symmetric load, the arch toes drifted away, as expected.

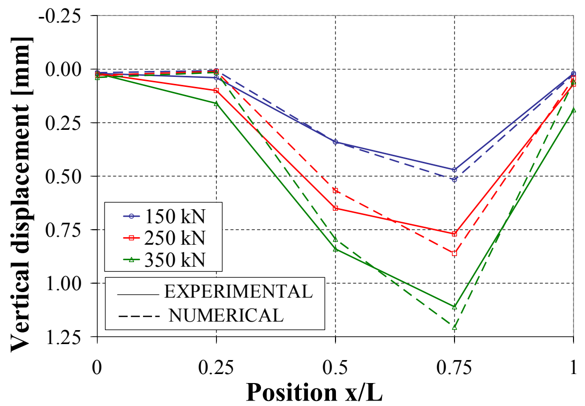

Fig. 5 Vertical displacement of arch 4 for static asymmetric load test obtained by optical level

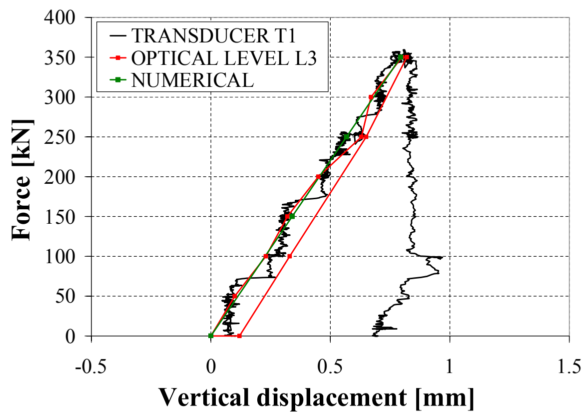

Fig. 6 Comparison of results between optical level and wire transducer for mid-span of arch 4

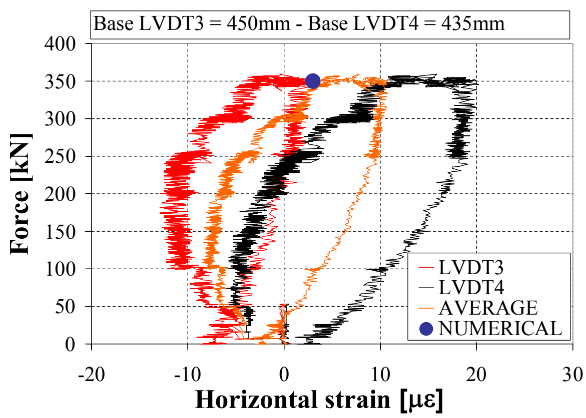

Fig. 7 Horizontal strain at key brick position calculated during static asymmetric load test of arch 4

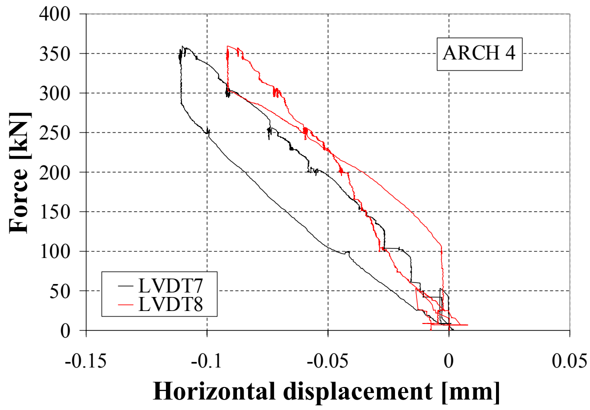

Fig. 8 Horizontal displacement of perimeter wall and intermediate column in static asymmetric load test

As regards the dynamic tests interpretation, the three graphs of power spectral density of the signal, obtained by the post-processing of the three recorded accelerograms are reported in (Fig. 9). The first two natural frequencies of the arch 4 can be identified as 15.0 Hz and 18.6 Hz.
In order to compare results of the other tested arches, in the following, the vertical displacements which were measured for each arch both for the symmetric (Fig. 10, Fig. 12, Fig. 14) and asymmetric (Fig. 11, Fig. 13, Fig. 15) static load tests for the three load values of 100 kN, 250 kN and 350 kN are reported.

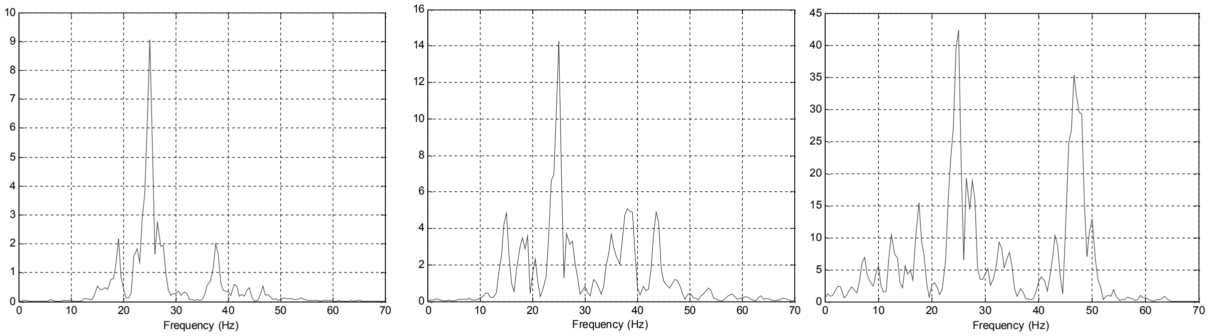

Fig. 9 PSD of accelerograms measured by A6, A7 and A8 accelerometers during dynamic tests of arch 4

Different arches show similar and corresponding results under symmetric loading; as a matter of facts, their lowering are comparable and very similar. Under asymmetric load they have similar trends but they showed significantly different values. These results were expected since each arch of the four tested, shows different and quite heterogeneous deterioration conditions and around there are many different situations especially as regards the type of floor, perimeter walls and internal partitions affecting the overall mechanical behaviour.

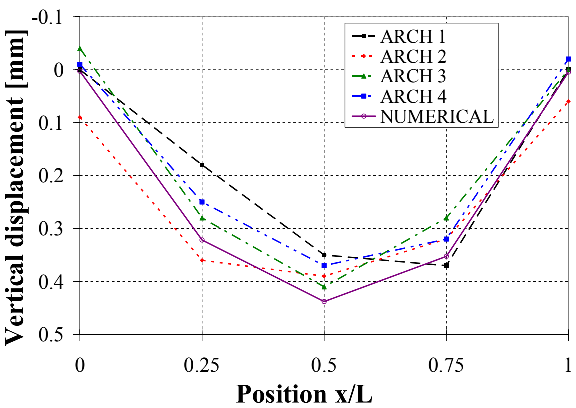

Fig. 10 Comparison between vertical displacements of tested arches for 100 kN (symmetric load)

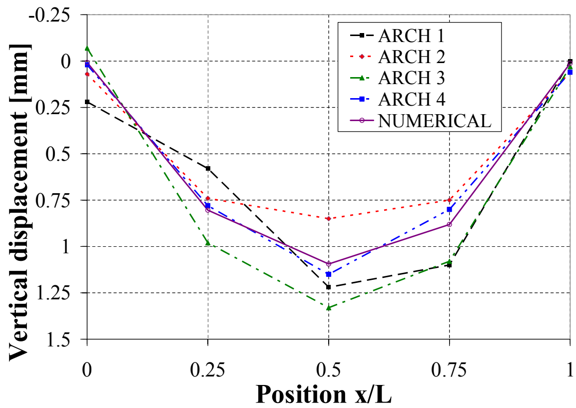

Fig. 11 Comparison between vertical displacements of tested arches for 250 kN (symmetric load)

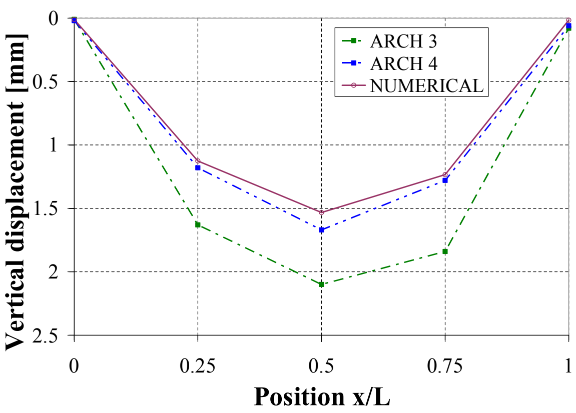

Fig. 12 Comparison between vertical displacements of tested arches for 350 kN (symmetric load)

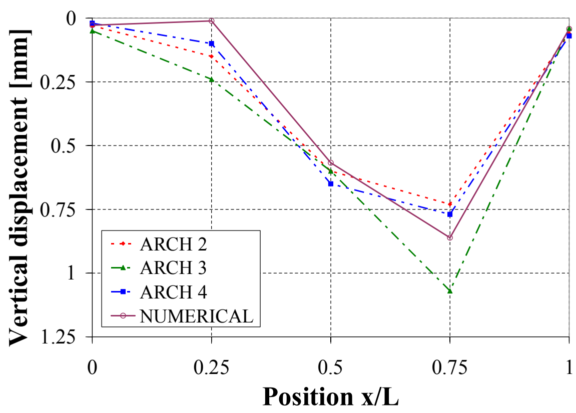

Fig. 13 Comparison between vertical displacements of tested arches for 250 kN (asymmetric load)

4. THE NUMERICAL RESULTS

In order to validate and better understand the experimental results, a finite elements model reproducing the geometry of arch 4, extensively discussed above, was performed. In more details, the entire cross-section of the building (on station side) containing the arch 4 was modelled. In order to find the interactions with the contiguous arches, three of them and the connecting cross vaults were considered (Fig. 16). This model is made of 312833 "brick" finite elements with 6 and 8 nodes. Since the describes tests considered service loads, analyses carried out to reproduce experimental results were characterized by the adoption of a linear elastic material with E = 1800 MPa and ν = 0.20 for all the structural elements. The adopted value of elastic modulus is larger than the value obtained from tests but it was obtained following a simple optimization technique comparing numerical and experimental structural results. In this perspective it is useful to remark that real walls (1.6 metres thick) are badly represented by a drilled core of 15 cm in diameter. Moreover, the cross vaults are made of brickwork and the adopted elastic modulus is typical for this type of material. For reason of brevity, numerical results (both for symmetric and asymmetric load case) are over imposed to experimental ones from (Fig. 3) to (Fig. 7) and from (Fig. 10) to (Fig. 15).

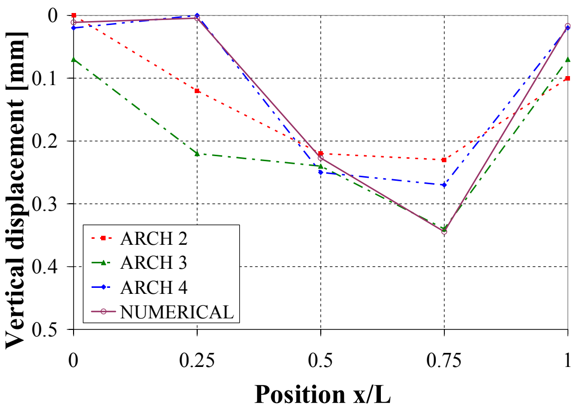

Fig. 14 Comparison between vertical displacements of tested arches for 100 kN (asymmetric load)

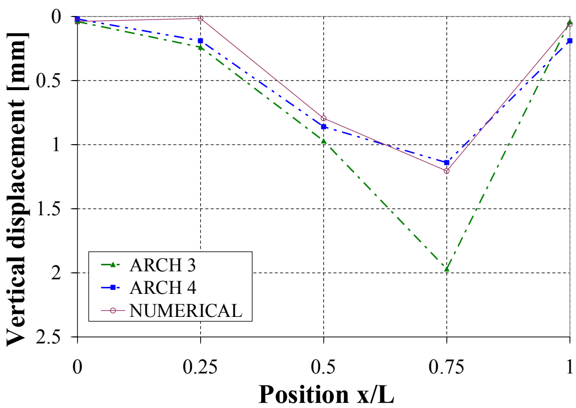

Fig. 15 Comparison between vertical displacements of tested arches for 350 kN (asymmetric load)

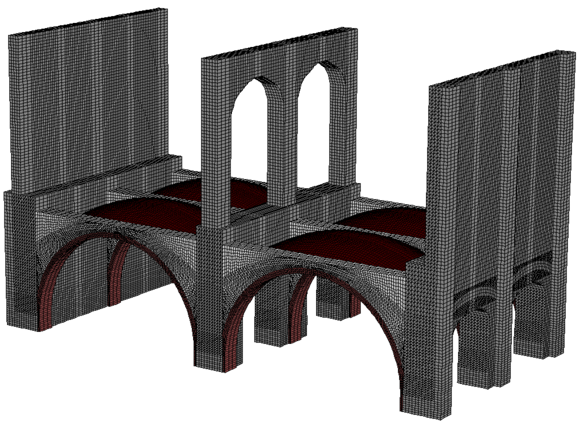

Fig. 16 Finite elements model used for the study of behaviour of arches and vaults

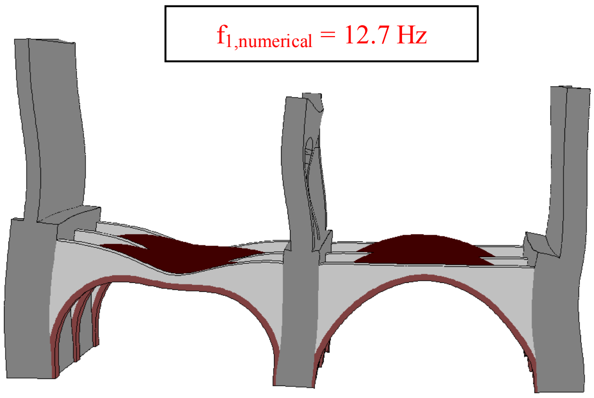

Fig. 17 Modal shape of the first eigen mode that excites vertically the structure

They match satisfactorily the experimental tests results both as regards displacements and strains, and thereby demonstrating that the finite elements model is able to grasps the structural behaviour of the real building with good approximation. In particular, vertical displacements properly describe the mean experimental behaviour while local strains at least are identified as order of magnitude as show (Fig. 4) and (Fig. 7).

A modal analysis was also conducted in order to identify the natural frequencies of the system (Fig. 17). Although this is only a local model, and, as a consequence, it is limited to a transversal portion of the building, the first calculated frequency, exciting a considerable mass along the vertical direction, is $f_{1,\ numerical}$ = 12.7 Hz quite close to $f_{1,\ experimental}$ = 15 Hz of arch 4.

Moreover, considering that the average value of the first frequencies experimentally measured for the four arches $f_{1,\ average}$ = 12.9 Hz, the model properly matches also the dynamic behaviour under service conditions.

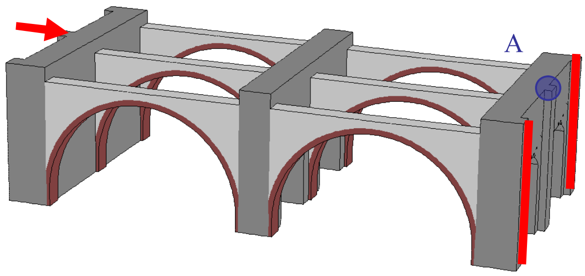

Fig. 18 Model without vaults used for the study of transverse translational stiffness

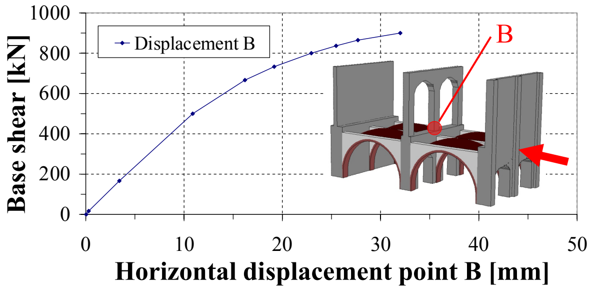

Fig. 19 Capacity curve from pushover

In order to assess, albeit approximately, the vaulted floor stiffness with respect to horizontal displacements, a linear static analysis was conducted imposing a horizontal force on central arches and constraining the model to the horizontal translations only at extremity arches (Fig. 18). This analysis was carried out with and without the cross vaults. The values of the displacements at the control point A are summarized below (Tab. 4). By processing these displacements, it could be inferred that the percentage of transverse translational stiffness of the brickwork vaults is about 79% of the whole structure. Therefore, these elements play a fundamental role in the structure, spreading horizontal loads on more arches.

Table 4 Contribution of vaults at transverse stiffness

Point	Displacement with the vaults δ_A' [mm]	Displacement without the vaults δ_A'' [mm]	Contribution of vaults (δ_A'' - δ_A') / δ_A''
A	0.12	0.56	78.5 %

In order to understand the ductility of the arches-vaults system, a non-linear static pushover analysis was conducted. In this analysis, the non-linear behaviour of materials was modelled using a smeared model with homogenized properties for isotropic continuum [5]. This model is computationally not so expensive and allows analysis of large structures. The yielding criterion adopted is that of classical Mohr-Coulomb theory with the choice of 0.1 MPa for cohesion and 21° for friction angle. The global failure collapse criterion was defined as the reaching of maximum compressive stress (4 MPa). From the curve of (Fig. 19) it is possible to estimate the ductility of the structure as regards transversal horizontal load applied at vaulted floor level.

5. CONCLUSIONS

The interpretation of experimental tests, carried out on four stone arches, allowed to estimate and to evaluate the mechanical and structural properties of the studied building, as regards its arches and vaults bearing system. By means of these static and dynamic tests, it was possible to calibrate a finite elements model in order to reproduce the experimental tests. The good concordance between experimental behaviour and numerical analyses confirms the reliability of the results obtained. Linear and non-linear analyses were carried out as regards horizontal loads, obtaining that vaults play an important role in spreading loads and that the system bearing horizontal loads shows good structural ductility.

ACKNOWLEDMENTS

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