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NUMERICAL PRE-DICTION OF THE SEISMIC BEHAVIOUR OF A MASONRY VAULT MOCK-UP USING THE NOSA-ITACA CODE

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Abstract

Masonry vaults are widely employed in ancient constructions and play a crucial role in their static and dynamic response. Even if they are designed to withstand gravity and dead loads, these structural elements must also resist dynamic excitations caused by traffic and earthquakes; hence, the knowledge of their behaviour still requires in-depth analyses from experimental and numerical points of view. Within the framework of the SERA.TA project (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe), a blind pre-diction contest has been organized to assess the numerical analyses capability to predict the seismic response of a 1:1 scale model of masonry cross vault, realized and tested at LNEC laboratory (Portugal). This paper describes the analyses conducted on a numerical model of the vault created by NOSA-ITACA, a code developed in-house by ISTI-CNR for the analysis and calibration of masonry structures. The experimental accelerations, displacements, and crack patterns have been compared with the predicted numerical ones achieved in the blind pre-diction phase, by performing a nonlinear dynamic analysis of the unstrengthened finite element model of the vault.

Keywords: no tension material, masonry vault, groin vault, nonlinear elasticity, nonlinear dynamic analysis, seismic behavior, masonry material.

1 INTRODUCTION

Masonry vaults are widely employed in ancient constructions and play a crucial role in their static and dynamic behaviour. Even if they are specifically designed to endure heavy vertical loads, they must usually also withstand dynamic excitations caused by traffic and earthquakes; hence, the knowledge of their behaviour still requires in-depth analyses from experimental and numerical points of view.

Focusing on groin vaults, often employed to cover churches' naves and palaces' rooms, the most frequent cause of their failure is represented by the support movements resulting in a shearing action in the horizontal plane of the structure.

From an experimental point of view, few researchers conducted full experimental campaigns on vault shear behaviour as reported and described in detail in [1], while from a numerical perspective, plenty of approaches were developed to examine the structural behaviour of these structures ranging from limit analysis to Finite Element Method (FEM) and Discrete Element Method (DEM) [2, 3].

Nowadays, FEM is one of the most adopted procedures for structural analysis, with many constitutive equations capable of analysing the masonry response, modelled as an equivalent continuum [4-7], or as an assemblage of macro elements with few degrees of freedom and a pre-established behaviour [3, 8, 9].

Within the framework of the SERA.TA project (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe), a blind pre-diction contest has been organised to assess the numerical analyses approaches' capability to predict the seismic response of a full-scale masonry cross vault tested at LNEC laboratory (Portugal), considering its un-strengthened and strengthened configuration [10].

This paper describes the analyses conducted on a numerical model of the vault created by NOSA-ITACA (www.nosaitaca.it/software/), a finite element code developed in-house by ISTI-CNR, for the analysis and calibration of masonry structures.

The numerical simulations were performed before the experimental tests, modelling the masonry vault by the constitutive equation of no-tension (or *masonry-like*) materials implemented in NOSA-ITACA [4].

The numerical results obtained for the vault un-strengthened configuration have been compared to the experimental one in terms of frequencies, mode shapes, accelerations, displacements, and crack patterns [11].

2 MASONRY VAULT FINITE ELEMENT MODEL

The groin vault investigated in this paper has been built and tested at the National Laboratory for Civil Engineering (LNEC) in Portugal.

The geometry of the mock-up is about 3.5x3.5 m in plan and includes: two semi-circular barrel vaults with a net span of 2.9 m, a rise of 0.80 m and a constant thickness of 0.12 m; two masonry piers clamped at the base representing the fixed vault’s support; two 0.84x0.84m steel blocks representing the movable supports; three couples of steel rods linking the four abutments [10].

The specimen numerical model has been created by importing the vault CAD model in the NOSA-ITACA code.

NOSA-ITACA is free software developed in-house by ISTI-CNR to disseminate the use of mathematical models and numerical tools in the field of Cultural Heritage. The code has been developed to study the static and dynamic behaviour of masonry structures. In recent years, it has been updated by adding several features that enable modal analysis, linear perturbation analysis [12], and model updating [13].

A refined finite-element model of the vault has been built, assuming a macro-modelling approach. The mesh, shown in Figure 1, consists of 83664 4-node isoparametric tetrahedrons, 7612 8-node isoparametric hexahedrons and six 2-node isoparametric truss elements (element n.25, n.8 and n.35 of the NOSA-ITACA library) with 29353 nodes, for a total of 88056 degrees of freedom. Truss elements are used to model the tie-rods, assuming a cross-section of $8.04 \cdot 10^{-4}$ m^2 (corresponding to steel bars of 32 mm diameter). The material forming the vaults has been modelled by the constitutive equation of *masonry-like* (or no-tension) materials that models masonry as an isotropic homogeneous nonlinear elastic material with zero or low tensile strength σ_t and infinite or bounded compressive strength σ_c [4]. This equation can consider some of the masonry's peculiarities, particularly its inability to withstand significant tensile stresses.

The other parts of the mesh (steel blocks, beams, tie-rods, pillars and infill material) are modelled assuming a linear elastic behaviour. The materials' mechanical properties used in the blind pre-diction analysis are summarized in Table 1; the masonry's elastic modulus E and Poisson ratio ν properties used are based on preliminary experimental tests provided to the participants beforehand [10]. Tensile and compressive strengths σ_t and σ_c are taken equal to 0.0 and 4.55 MPa (the latter obtained as a ratio between the experimental compressive strength [10] and a partial factor for material equal to 2 as suggested by Italian regulations NTC2018 [14]). Regarding the boundary conditions, the piers are assumed to be clamped at the base (red dots in Figure 1). At the same time, the steel masses and IPE beams have the vertical direction fixed, allowing movement in the longitudinal and transverse directions (blue dots in Figure 1).

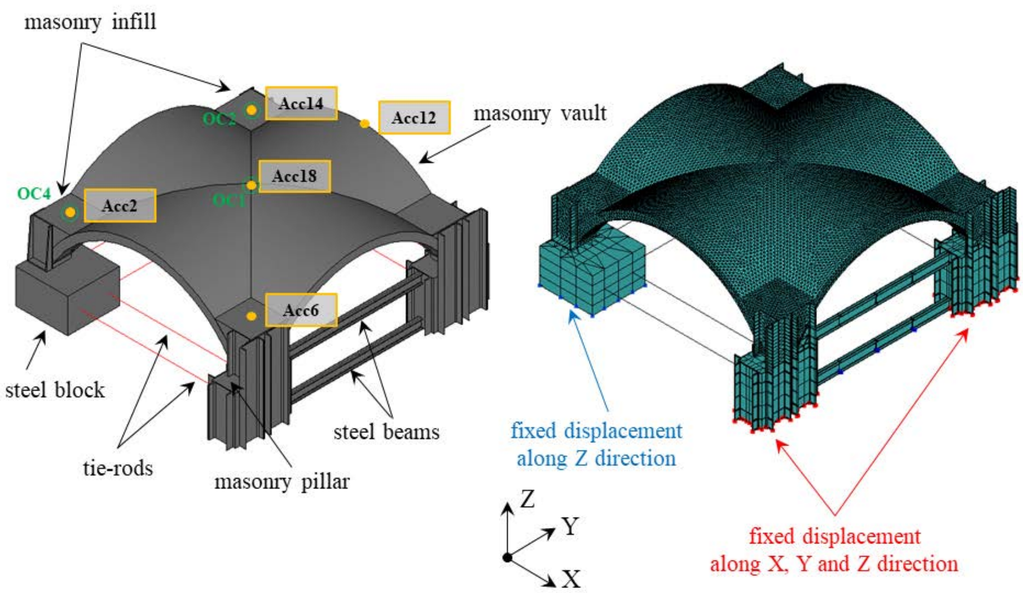

Figure 1: Vault geometry and finite element model

	Masonry vault	Infill and pillars	Steel blocks and beams
E [GPa]	2.223	2.223	210.0
ρ [kg/m^3]	2255.2	2255.2	7880.0
ν	0.2	0.2	0.3

σ_t [MPa]	0.00	--	--
σ_c [MPa]	4.55	--	--

Table 1: Mechanical materials properties. *E*, Young's modulus; ρ, mass density; ν, Poisson ratio; σt, tensile strength; σc, compressive strength

3 DYNAMIC BEHAVIOUR OF THE UN-STRENGTHENED VAULT

In the blind pre-diction phase, a preliminary modal analysis has been performed to estimate the numerical model's dynamic behaviour and check the correspondence between experimental and numerical mode shapes.

Table 2 reports the experimental f_{exp} [11] and numerical frequencies f_{num}, the absolute relative error and modal participation factors calculated by NOSA-ITACA. The table shows that the frequencies of the un-strengthened model are higher than the measured one, probably due to an unsuitable masonry's elastic modulus provided by the blind pre-diction organizers. At the same time, there is a good matching between the first, second and fourth numerical mode shapes and the first three experimental ones reported in [11] that are, respectively, a shear mechanism along the Y direction, a bending mode in the X direction and a vertical mode shape along Z, as sketched in Figure 2 [11].

	f_{exp} [Hz]	f_{num} [Hz]	Relative error [%]	Mx [%]	My [%]	Mz [%]
Mode 1	6.15	9.33	51.50	0.00	73.30	0.00
Mode 2	11.62	24.85	113.86	63.53	0.00	0.04
Mode 3	--	29.59	--	0.00	0.00	0.00
Mode 4	19.39	30.88	59.26	0.06	0.00	0.08

Table 2: Experimental vs numerical frequencies and modal participation factors

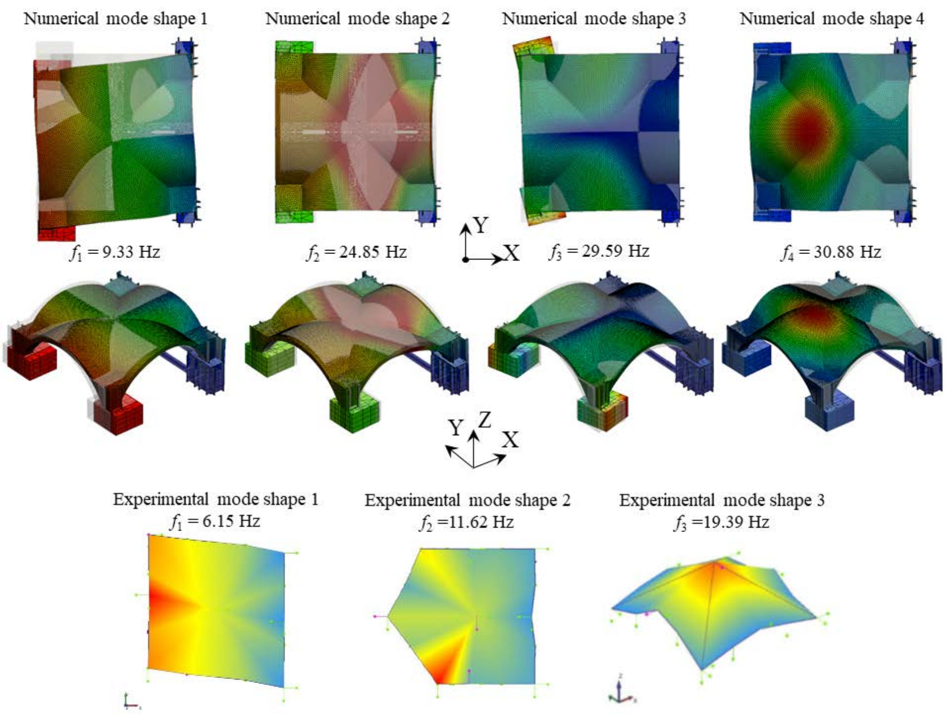

Figure 2: First four numerical mode shapes and the three experimental mode shapes

Subsequently, a nonlinear dynamic analysis has been performed considering the self-weight of the vault and applying at the base, in the Y direction, the accelerogram recorded during the L'Aquila earthquake (Figure 3). The analysis has been carried out assuming a time step of 0.0025 sec and scaling the accelerogram magnitude by 25%, as indicated by the organizers of the contest. The damping matrix has been calculated according to the Rayleigh hypothesis with a damping ratio equal to 2.5%.

In particular, the two Rayleigh coefficients are estimated, assuming the first and the twenty-second frequency as the reference, which involves 86% of the mass in the horizontal directions.

The failure mechanism obtained from the nonlinear dynamic analysis is shown in Figure 3. As seen in the picture, it is predominantly an in-plane shear mechanism accompanied, albeit to a lesser extent, by an in-plane bending of the south and north faces arches; the deformation follows the second mode shape of the vault that looks like a typical asymmetrical mode shape of an arch with lateral displacement according to one direction. The deformed shape, sketched in Figure 3 with a deformation scaling factor equal to 20, occurs at the time t = 4.25 seconds; the contour band refers to the value of the norm of the displacements expressed in meters.

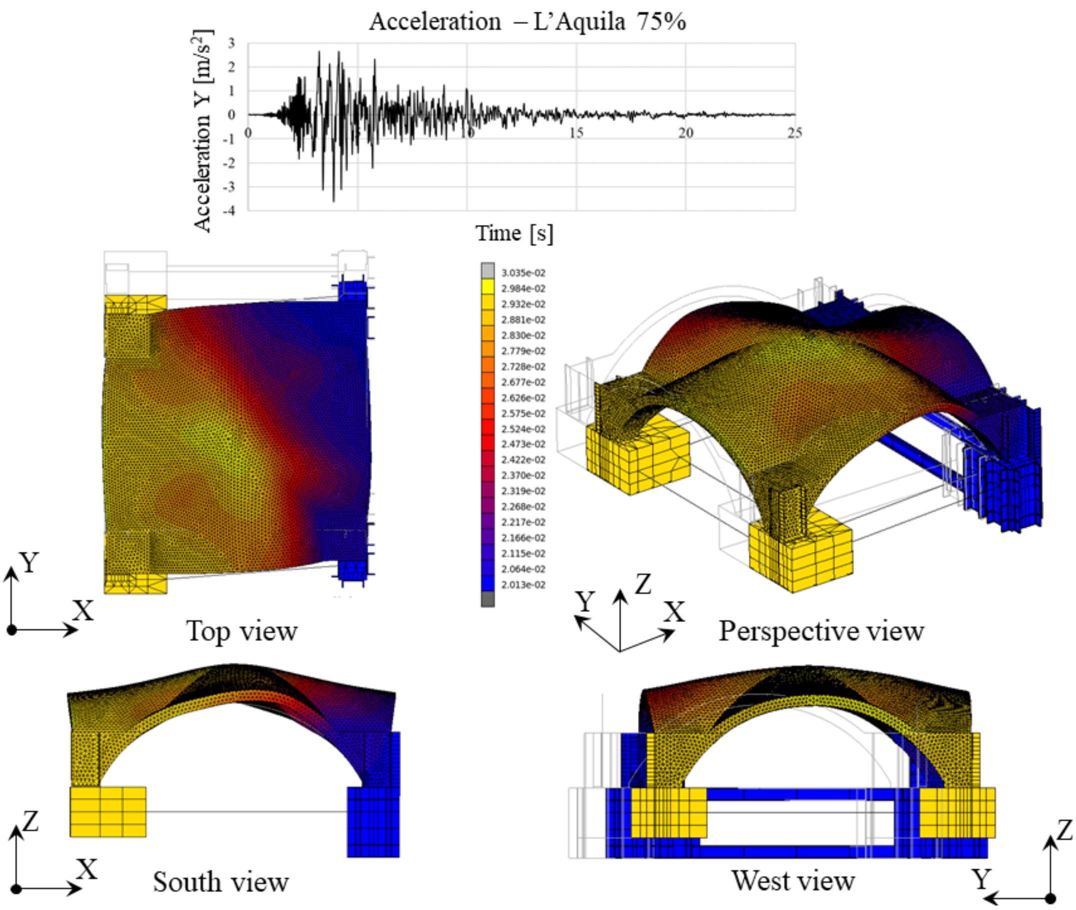

Figure 3: Acceleration time history and vault's deformation shape at failure

Figure 4 shows the crack pattern in terms of EFEQV (the norm of the fracture strain tensor); it is characterized by diagonal cracks, with a high concentration of fracture strain at the top of the vault extrados and near the infill. The numerical fracture distribution matches the experimental damage (shown in the middle of Figure 4), except for the cracks on the webs that are not visible in the numerical solution. Furthermore, the numerical results show a limited concentration of crushing cracks in small areas near the filling material.

Tables 3 and 4 compare the maximum experimental and numerical total displacements and accelerations of the selected points shown in Figure 1. In terms of displacement, the discrepancy between the results is greater than 90% in the X direction, while in the Y direction it is about 13% except for OC_{2-y}. As concerns the acceleration, the numerical model overestimates, in general, its value in the X and Y directions.

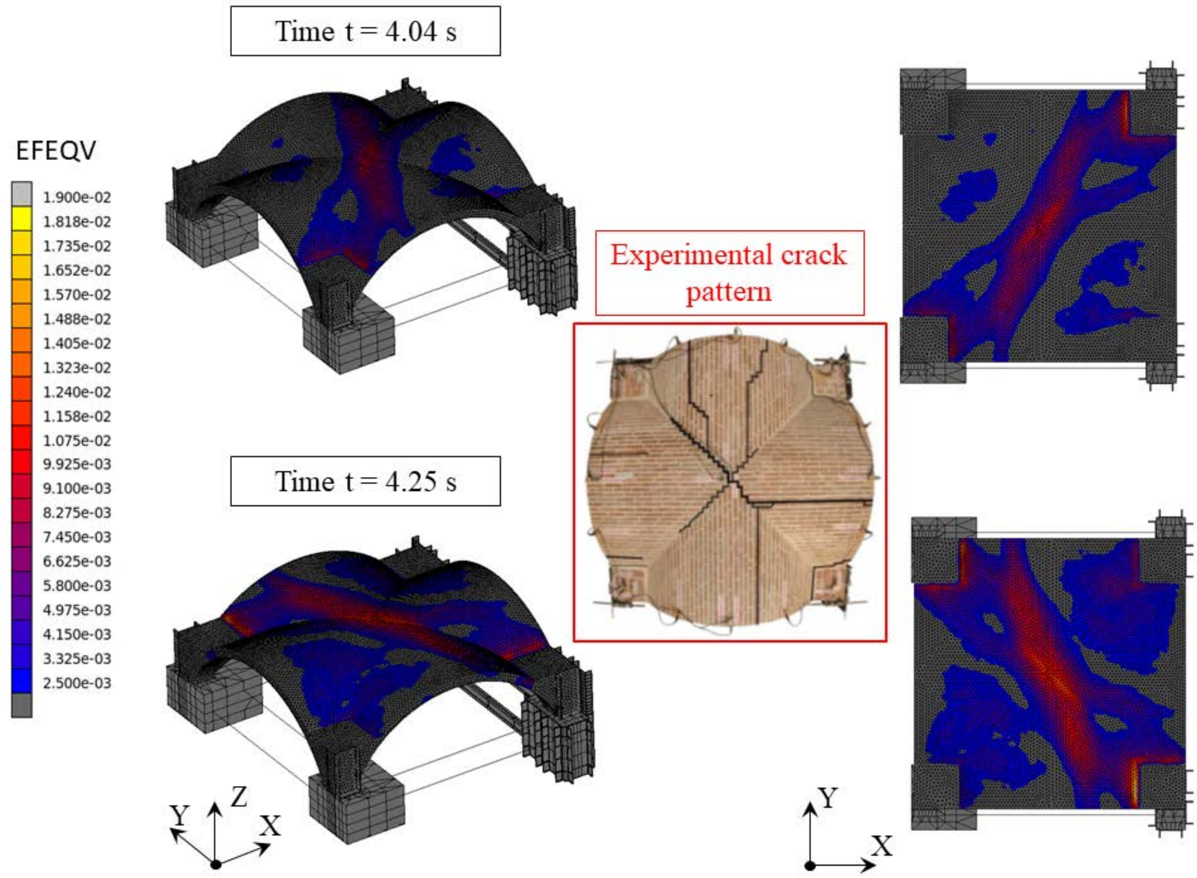

Figure 4: Crack pattern

	Experimental [mm]	Numerical [mm]	Relative error [%]
OC_{2-x}	4.01	0.29	92.77
OC_{4-x}	7.78	0.24	96.92
OC_{1-y}	27.90	31.39	-12.51
OC_{2-y}	70.66	31.40	55.56
OC_{4-y}	35.83	31.37	12.45

Table 3: Maximum total displacements (absolute value in mm)

	Experimental	Numerical	Relative

	[m/s^2]	[m/s^2]	error [%]
Acc_{2-x}	4.39	8.56	-94.99
Acc_{6-x}	3.98	4.63	-16.33
Acc_{12-x}	3.44	4.88	-41.86
Acc_{14-x}	5.23	9.55	-82.60
Acc_{18-x}	3.77	5.03	-33.42
Acc_{2-y}	4.53	5.66	-24.94
Acc_{6-y}	4.51	4.90	-8.65
Acc_{12-y}	4.92	8.89	-80.69
Acc_{14-y}	4.44	6.43	-44.82
Acc_{18-y}	3.44	6.11	-77.62

Table 4: Maximum total accelerations (absolute value in m/s^2)

4 CONCLUSION

The paper describes the results of numerical analyses conducted under the blind pre-diction contest organized within the framework of the SERA.TA project (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe). The seismic response of a 1:1 unstrengthened masonry cross vault specimen, realized and tested at the LNEC laboratory, has been simulated by the NOSA-ITACA code, a non-commercial software developed by ISTI-CNR.

A refined finite-element model of the vault has been created, modelling masonry as a no-tension material with zero tensile strength and limited compressive strength. A preliminary modal analysis has shown the ability of the model to fit the specimen experimental mode shapes despite an overestimation of the associated frequencies.

Then, a nonlinear dynamic analysis was carried out, assigning a scaled real accelerogram at the mesh base. The numerical results have been compared to the experimental ones in terms of crack pattern, failure mechanism, maximum total displacements and accelerations at selected points. The crack pattern fits the experimental damage even if the fractures along the specimen diagonals do not form simultaneously. A damage concentration is also close to the infill, while cracks on the webs are missing.

Regarding the displacements and accelerations, the numerical model generally overestimates their values. The model will be calibrated and improved in the post-diction phase using the experimental modal properties and model updating techniques.

ACKNOWLEDGEMENT

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