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BLIND PREDICTION OF SHAKE TABLE TESTS ON A FULL-SCALE UNSTRENGTHENED MASONRY CROSS VAULT: COMBINED FINITE-DISCRETE ELEMENT MODEL

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Abstract

This work presents the blind results of the non-linear dynamic response of a three-dimensional finite-discrete element model of a full-scale unstrengthened masonry cross vault tested on a shake table. The cross vault is generated by the intersection of two semi-circular barrel brick vaults. The boundary conditions are asymmetric, with two piers fixed to the shake table and a steel frame between them to simulate the presence of a continuous lateral wall, while the other two piers are free to move along all directions on wheels to simulate the colonnade-between-naves condition. The seismic action was applied along the longitudinal direction of the specimen (parallel to the simulated lateral wall), and several sequential seismic tests, with increasing amplitude, were scheduled. The model used to blindly predict the test response allows for the elastic finite element modelling of masonry units, or blocks, and the discrete element modelling of the interfaces between them. A tension cut-off governs the interaction at a standard block interface: once this strength is exceeded, no tensile stress can be transmitted,. Shear response is defined by cohesion and friction, and once the cohesive contribution is exceeded, the interface interacts in shear according to Coulomb's behavior. The model was analysed under the scheduled experimental sequence of four amplitude intensity steps. The analysis at a 75% intensity of the selected natural ground motion accelerogram showed the activation of a shear mechanism, with hinges at about one-third of the span of the lateral circular arch, with negligible permanent displacement at the end of the analysis. The model capacity was reached at an amplitude intensity to 200% of the natural accelerogram.

Keywords: Numerical analysis, Church; Seismic behaviour, Unreinforced Masonry, Sequence

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1 INTRODUCTION

Recent earthquakes have emphasised the high vulnerability of vaulted structures. The behaviour of the vaults plays a relevant role in the seismic response of heritage masonry buildings, as observed in Italy for the 2009 L'Aquila earthquake [1], the 2012 Emilia earthquakes [2] and the 2016–2017 Central Italy earthquake sequence [3], but also for ancient events [4]. The verification of the safety of vaults is crucial [5] as collapse may either entail local damage or affect the whole building the vaults belong to, because of their heavy mass and the significant horizontal thrust on supporting elements. Despite the importance of this topic, evaluating the complex three-dimensional behaviour of vaults is still a significant challenge for both practitioners and researchers.

With the aim of investigating the dynamic response of vaulted structures, a full-scale unstrengthened masonry cross vault was tested on a shake table (Figure 1a) at the Civil Engineering National Laboratory (LNEC) in Lisbon, Portugal, within the Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe (SERA) program and a blind prediction competition was organised [6]. The cross vault was generated by the intersection of two semi-circular barrel brick vaults with constant thickness 0.12 m; the size in plan is about 3.50 × 3.50 m^2. The boundary conditions are asymmetric, as costumary in churches, with two piers fixed to the shake table and a steel frame between them to simulate the presence of a continuous lateral wall, whereas the other two piers are free to move on wheels along all directions to simulate the colonnade-between-naves condition. In addition, each pier is connected to the adjacent ones by a couple of steel tie rods to restrain support rotation. Steel plates were added along the height of the supports to increase their stiffness and avoid local failure. An infill, made of brickwork with horizontal courses, is placed on top of each pier, stabilising the vault thrust. Finally, steel masses were added to the supports to simulate the roof weight.

The seismic action was applied along the longitudinal direction of the specimen (parallel to the simulated lateral wall), and an accelerogram, recorded at AQA station during the 2009 L'Aquila earthquake, with peak ground acceleration (PGA) of 4.85 m/s^2, was used. Incremental shake-table tests were planned by applying to the natural ground motion the following scale factors: 25, 50, 75 and 100% [6].

In this contribution, a finite-discrete element modelling is used, similar to what done in previous studies on other unreinforced masonry structures [7–12], but never before of a cross vaults.

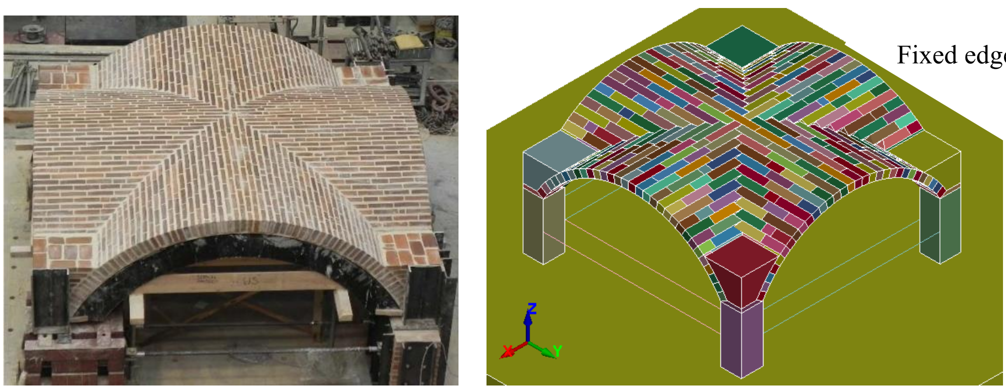
a)  b)

Figure 1Fig. 1 3D view: a) Physical model [6] and b) Numerical model. Blocks are coloured only for presentation purposes, their material properties being homogeneous.

2 OVERVIEW OF THE MODELING STRATEGY

The model foundation, as the shake table, was restrained along the *X* and *Z* directions. The overall geometry of the vault model is the same as the prototype, but all piers have a cross-section equal to that at the interface with the vault, neglecting steel additions (Figure 1b). The four piers are monolithic and cannot fracture along their height to account for the steel elements used in the physical prototype. Two piers are fixed at the base whereas the other two, corresponding to those resting on wheels in the prototype, have a very low strength interface at their base. The steel frame between piers A and C (Figure 2b) was not modelled, but the top of these piers experiences the same displacement of the shake table along the *Y* direction.

The model was implemented within LS-DYNA [13–15], allowing for the elastic finite element modelling of masonry units and the discrete modelling of the interfaces between them. Units are modelled as linear elastic 8-node solid elements with a single integration point. The major disadvantage of one-point integration is the need to control the zero-energy modes that arise, called hourglassing modes, which might entail divergence of the solution. Hourglass is controlled by utilising a Flanagan-Belytschko stiffness-type stabilisation [13]. The model does not explicitly consider mortar; instead, contact interfaces are used. Masonry blocks have the following boxing size: 250 × 120 × 118 mm^3, basically having the same length and width and double the average thickness of actual units. The blocks are wedge-shaped because interfaces have zero thickness and cannot account for the variable thickness of prototype mortar joints. The total number of blocks is 313, the total number of finite elements is 1147, and the total number of nodes is 4952. Two cutting planes are present along the groins, as in the actual construction (Figure 3).

The properties of the materials are reported in Table 1. Given the differences between the physical prototype and the numerical model, and in order to get the same mass, the non-fixed-base piers have a density equal to 26 438 kg/m^3, whereas in the fixed ones the density is equal to 3645 kg/m^3.

The tie rods are modelled as discrete elastic springs and, in order to account for the different lengths, the four rods along the *X* direction have an elastic stiffness equal to 7.23 10^4 kN/m, whereas in the case of the two rods along the *Y* direction, the stiffness is equal to 6.16 10^4 kN/m (Figure 1). No prestress force was applied.

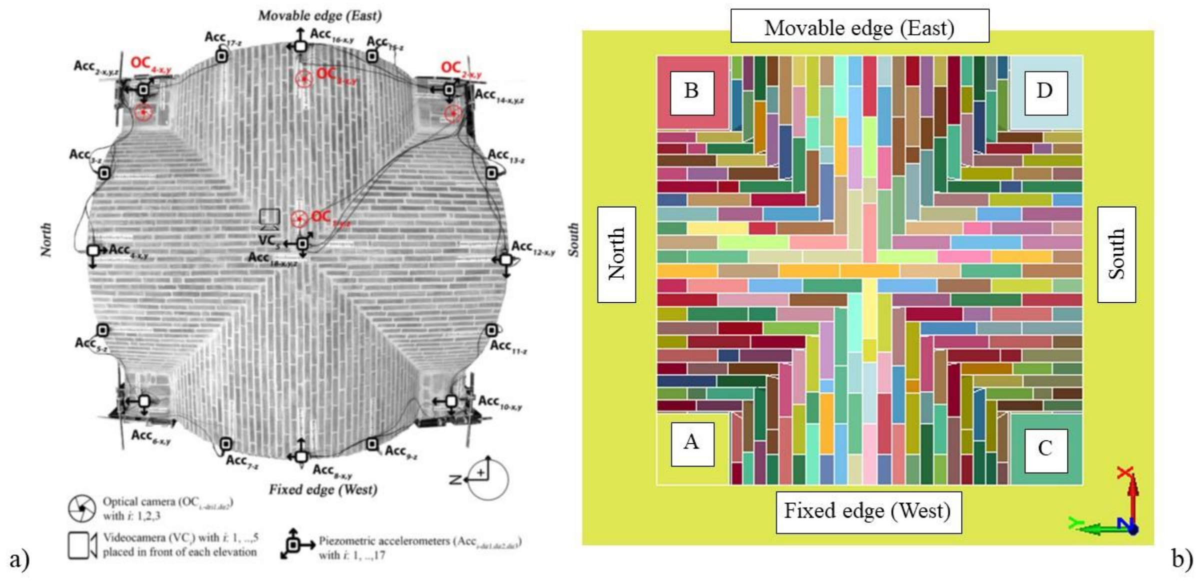


Figure 2 top view: a) Setup of the configuration [6] and b) Numerical model.

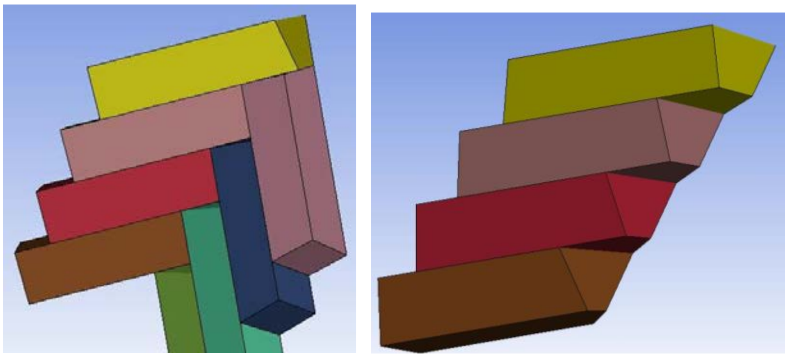


Figure 3 Blocks interlocked along the intersection between the webs of the vault.

parameter	block(1)	non-fixed-base piers (2)	fixed-base piers (2)
Density, ρ (kg/m^3)	2225	26 438	3645
Young's modulus, E (GPa)	2.223	2.223	2.223
Poisson's ratio, ν (-)	0.25	0.25	0.25

(1) informative leaflet [6]; (2) estimated

Table 1 Material properties

The adopted failure criterion at a standard block interface is based on tensile normal stress, i.e. failure occurs when the following inequality is satisfied:

$$\sigma_n \geq NFLS \tag{1}$$

where:
σ_n is the normal stress on the contact surface;
NFLS is the normal (tensile) failure limit stress.

Once this strength is exceeded, complete separation occurs at the interface, and the latter can interact only in compression once the crack is closed. The shear response is defined by cohesion, *SFLS*. Once cohesion is exceeded, the interface interacts in shear according to Coulomb's friction behaviour, depending on static and dynamic friction coefficients (*FS*, *FD*). All interfaces present the same properties, with the only exception of the interface at the base of the non-fixed piers which has negligible cohesion. The most relevant parameters are reported in Table 2. Finally, a 5% stiffness-proportional damping coefficient was assumed for all materials.

parameter	interface between blocks	interface at the base of the two non-fixed piers
NFLS (kPa)	31.000(1)	0.100(2)
SFLS (kPa)	31.000(2)	0.100(2)
FS, *FD* (-)	0.785(2)	0.785 (2)

(1) informative leaflet [6]; (2) estimated

Table 2 Interface parameters

3 PREDICTIONS

The model was analysed under a sequence of four amplitude intensity steps (10, 25, 50 and 75%) of the ground accelerogram, according to the scheduled testing sequence. In Figure 4 the maximum response during the analysis at 75% shaking intensity is shown. The analysis at the last step shows the activation of an in-plane shear failure mechanism with hinges at about one-third of the length of the free span of the lateral circular arch, as recognisable from the East and West views in Figure 4. No cracks along the diagonals or close to the groins are present. At the end of the analysis, permanent displacements are negligible.

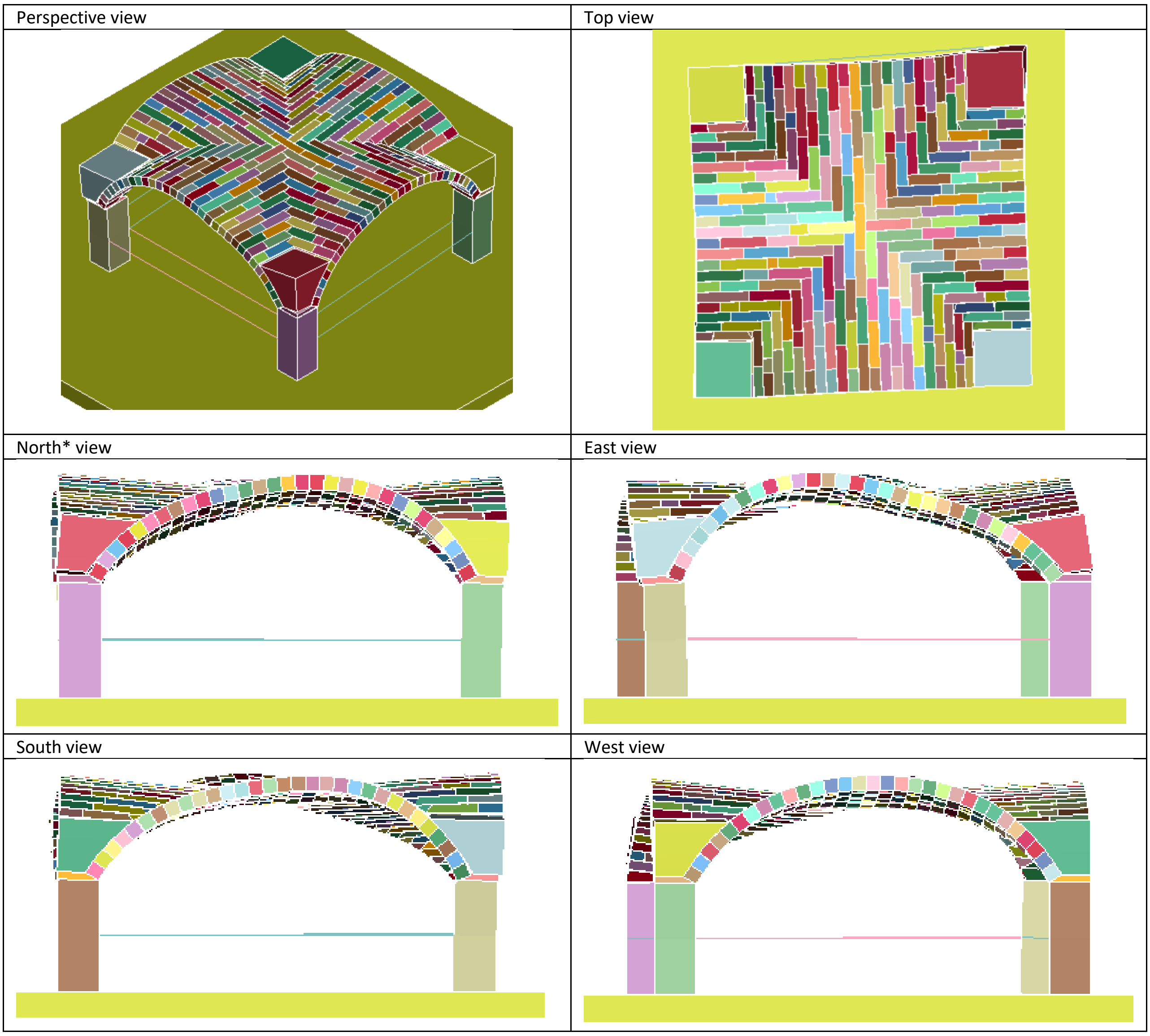


Figure 4 Numerical prediction of damage (75% intensity of shaking). Scale factor 5 applied to the deformations.

Further sequential analyses were performed according to the following intensity steps: 100, 125, 150, 175, and 200%. The collapse occurred at the very beginning of the last step, with PGA about 9.70 m/s^2 (Figure 5).

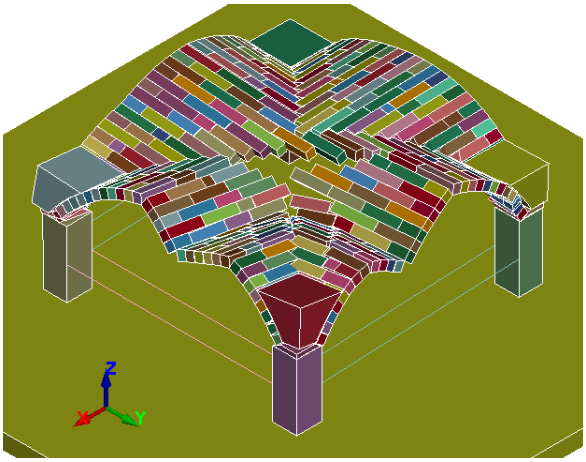


Figure 5 Numerical prediction of damage (200% shaking intensity). Scale factor 5 applied to the deformations.

4 CONCLUSIONS

The blind test prediction competition of a full-scale unstrengthened masonry cross vault tested in Lisbon represented the opportunity to implement a finite-discrete element model, which accounts for crack formation, complete separation and new contact formation. The modelling strategy was used to evaluate the ultimate displacement, the load-bearing capacity and the progressive deformation of the vault up to collapse under seismic action. While material properties were set equal to the experimental ones, the main limitation of the model resides in the size of the typical block, which is larger than its physical counterpart. In addition, the interface parameters were estimated to simulate the presence of wheels at the base of the two non-fixed piers. These features may lead to an overestimation of the capacity of the vault.

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