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DISTINCT ELEMENT MODELLING OF THE SEISMIC RESPONSE OF HISTORICAL MASONRY CONSTRUCTIONS: INSIGHT ON THE OUT-OF-PLANE COLLAPSE OF FAÇADES

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Abstract. Façades belonging to historical masonry constructions typically fail by out-of-plane mechanisms. The estimate of their out-of-plane capacity is not a trivial task, due to the different possible collapse modes (overturning, bending, disaggregation, leaf separation, sliding) and to the discontinuous nature of masonry, influencing the non-linear seismic behaviour of walls. Simplified approaches, proposed by building codes, mainly based on the mechanics of the rigid block, may not always be suitable for the purpose. Indeed, they disregard the real morphology of masonry, which instead influences weaker failure mechanisms (such as disaggregation and leaf separation). Furthermore, they neglect the interaction of the façade with the rest of the building and its interlocking with transversal walls.

These shortcomings can be overcome resorting to distinct element method (DEM), in which masonry is modelled as an aggregation of discrete units and no-thickness interfaces and the actual morphology of constructions is considered.

In this paper, DEM is adopted to investigate the out-of-plane seismic behaviour of façades through non-linear analyses, by focusing on vertical bending and overturning failure mechanisms. The former is studied by comparing results of shake table tests on both single-leaf and double-leaf masonry walls to dynamic simulations in which real accelerograms are applied. The latter is analysed by performing non-linear static analyses on the Romanesque church of St. Maria Maggiore in Tuscania, Italy, by focusing on its façade.

Distinct element method provided a realistic description of the behaviour of façades under earthquake loadings, in terms of both seismic capacity, crack pattern and failure mode.

1 INTRODUCTION

Although a large amount of existing masonry constructions is still in service today, recent seismic events have drastically highlighted how masonry structures are vulnerable to earthquake [1,2].

Amongst all the possible failure modes exhibited by masonry buildings under seismic action, the one that most commonly occurs is the out-of-plane failure of perimeter walls and façades.

Depending on mechanical properties and morphology of masonry, connections between walls and boundary conditions, the out-of-plane collapse of walls may occur through a mechanism, such as overturning and vertical/horizontal bending, or by weaker failure modes, as in the case of disaggregation and leaf separation [3,4]. These latter failure modes are typical of historical masonry constructions, commonly made of poor-quality masonry [5].

Structural analysis based on the assumption a priori of collapse mechanisms and on the mechanics of the rigid block, as suggested by most of existing building codes [6], may not properly catch the actual seismic behaviour of walls. Indeed, no interaction between the façade and transversal walls is considered and the actual morphology of masonry, typically influencing weaker failure modes such as disaggregation and leaf separation, occurring in old masonry buildings, is disregarded. On the basis of the above, this analysis approach may overestimate the seismic capacity of walls, thus resulting unconservative.

The distinct element method (DEM), a micro-modelling approach that represents masonry as an assembly of discrete blocks, in contact through non-linear zero-thickness joints, may overcome these limitations [7,8]. Indeed, DEM allows to account for the effective geometrical characteristics of masonry – shape and arrangement of blocks – and is able to reproduce the opening of cracks and joint sliding, which cause structural damages and collapse. Moreover, it permits to follow the development of the collapse mechanism, from its onset up to the achievement of failure. Additionally, DEM is suitable for non-linear dynamic analyses, since its resolution algorithm is based on the explicit integration of motion equations. All these features make DEM notably suitable for modelling the behaviour of masonry walls under seismic actions.

Within this framework, the present work describes two applications of DEM for studying the out-of-plane failure of masonry façades under seismic loading. Two failure mechanisms are investigated: vertical bending and overturning. The former is analysed through the simulation of shake table tests carried out on two full-scale masonry walls, a single leaf tuff wall and a double-leaf stone wall, subject to real accelerograms. The latter is investigated by performing a non-linear static – pushover – analysis to the façade of the Romanesque church of St. Maria Maggiore in Tuscania, Italy. Both the applications were carried out with the code UDEC® [9].

2 DISTINCT ELEMENT METHOD

The distinct element method (DEM), belonging to the category of micro-modelling approaches, considers masonry as an aggregation of discrete units (the blocks) – either rigid or deformable – that interact at contact points, located at the edges of zero-thickness discontinuities (the joints) [7]. In the most common case of rigid blocks, all the mechanical properties of masonry (friction angle, cohesion, tensile strength, and normal and shear stiffness) are lamped into contact points, in which two nonlinear springs are located, one representative of the normal response and the other one of the tangential response.

In DEM resolution algorithm two sets of equations are considered: the constitutive law of joints and the equation of motion, the latter being explicitly integrated every time steps to provide the current position of each block [10].

The method is widely used for analyzing the structural behaviour of masonry constructions, since it allows to accurately represent both shape and arrangement of blocks, to catch crack formation and sliding phenomena, also accounting for large displacements and rotations. Furthermore, its resolution algorithm makes it suitable for non-linear dynamic analyses under earthquake.

3 INVESTIGATION OF THE OUT-OF-PLANE VERTICAL BENDING FAILURE MECHANISM

The first case study deals with the DEM simulation of shake table tests for studying the out-of-plane vertical bending failure mode of masonry walls. The experimental response of two full-scale masonry panels, a double-leaf rubble stone masonry wall and a single-leaf regular tuff masonry wall, was simulated through nonlinear dynamic analyses.

In this section some of the main outcomes of the research are shown. For further details the reader is referred to [11,12].

3.1 Shake table experimentation

Shake table tests were carried out on two full-scale masonry panels, 3.48m high, 1.53m wide and 0.25m thick, one made of two leaves of irregular-shaped lime stone blocks partially connected by headers and the other one consisting of a single leaf of regular tuff blocks (Figure 1). Tuff masonry had a weight density of 24.2kN/m^3, a compressive strength of 14.3N/mm^2 and a compressive elastic modulus of 4522N/mm^2. Stone masonry had 12.3kN/m^3 weight density, 5.9N/mm^2 compressive, and 1575N/mm^2 Young's modulus.

The panels were built on the same reinforced concrete foundation, connected to the shake table; in this way they were tested together and subject to the same dynamic input. A crowning beam made of brick masonry reinforced with Steel Reinforced Grout [13] was built on top of each wall, and connected to it by steel connectors to prevent sliding.

In order to simulate the presence of a light timber roof, each wall was provided on top with a 600kg extra mass (steel plates). Furthermore, with the aim of triggering the out-of-plane vertical bending mechanism, the first masonry layer was embedded in the foundation (preventing cracking and sliding at the interface between masonry and concrete) whilst the crowning beam was not allowed to move horizontally, thanks to the presence of two rigid steel frames, which instead left its vertical displacements and rotations unrestricted [14].

Five input signals referring to the strongest recent Italian earthquakes were selected from the European Strong Motion Database (ESD) (Table 1) and applied in the out-of-plane horizontal direction and in the vertical direction, with increasing scale factors until a considerable damage level (near-collapse condition) was reached.

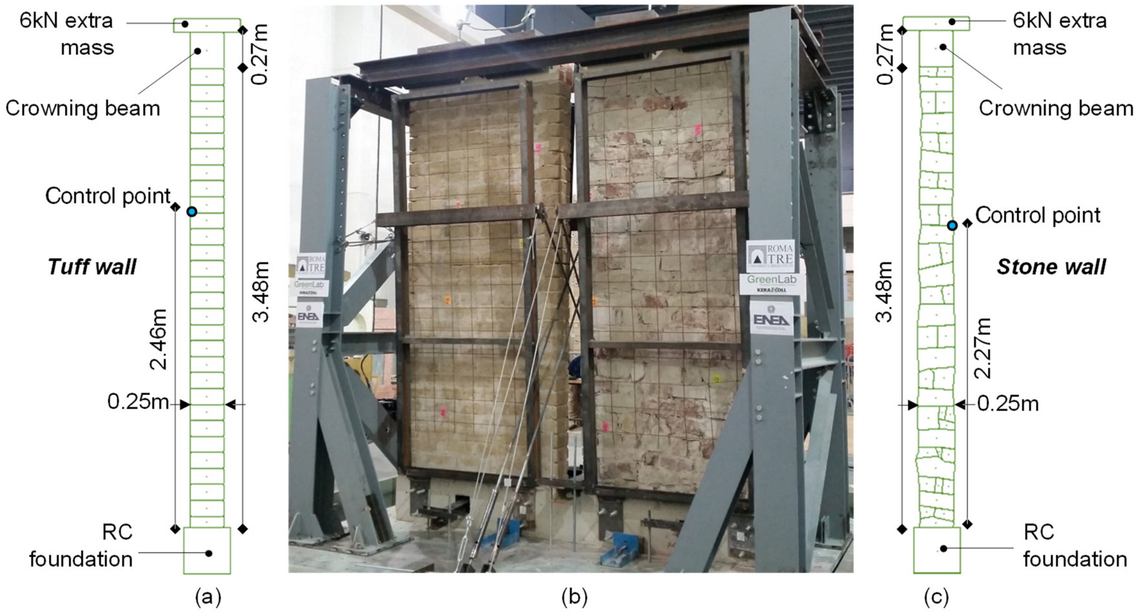

Figure 1: DEM models (a,c) and specimens tested on the shake table (b).

Table 1: Accelerograms selected from European Strong Motion Database (ESD) to carry out the shake table tests.

Seismic Event	Moment magnitude	PGA-horizontal [g] [(b)]	PGA-vertical [g]
23/11/1980, Irpinia (BGI)	6.9	0.181	0.101
20/05/2012, Emilia (MRN)	6.1	0.267	0.303
2016, Amatrice (AMT) [(a)]	6.0	0.376	0.399
26/09/1997, Umbria-Marche (NCR)	6.0	0.492	0.398
06/04/2009, L'Aquila (AQV)	6.1	0.644	0.486

[(a)] The AMT record scaling was corrected in the ESD after the performance of the shake table tests. The amplitude of the correct accelerogram is twice as that used in the experimental investigation. [(b)] PGA = Peak Ground Acceleration.

3.2 Nonlinear dynamic simulations

A 2D DEM model was implemented for each of the two panels, by accurately reproducing masonry morphology (shape, size and arrangement of blocks) on the basis of a survey of the lateral side of walls. The RC foundation, the top brickwork beam and the extra masses were also explicitly considered in the models (Figure 1).

Due to the two-dimensional nature of the model, a unitary width is assumed in the third dimension. The third dimension is accounted for by amplifying model mass density, stiffness, cohesion and tensile strength by the wall width [15,16]. To simulate the presence of the rigid steel frame, the block on top was assumed to have the same horizontal displacement of the foundation.

Blocks were assumed as rigid and a Mohr-Coulomb criterion with zero residual tensile strength and cohesion was adopted for joints to account for the accumulation of damage experienced by the walls during tests [15]. Normal and shear stiffnesses were calibrated by comparing experimental and numerical frequencies of walls under white noise input signals. For both the panels, a friction angle of 35° and a cohesion of $0.5N/mm^2/m$ were assumed, whereas the tensile strength was set equal to $0.3N/mm^2/m$ and $0.4N/mm^2/m$ for the tuff wall and the stone wall respectively. In order for the hinge to form in a position close to that found during experimentation, cohesion and tensile strength of joints cracked in the tests were imposed equal to zero. A Rayleigh damping ratio of 3% for the tuff panel and 1% for the stone one, in correspondence of a critical frequency equal to the fundamental frequency, was assumed [8].

Nonlinear dynamic analyses were carried out in two steps. First gravitational loads were applied and then velocity time histories were assigned to the foundation block (both horizontal and vertical velocity) and to that on top (horizontal velocity only). These histories were obtained through the integration of the accelerations recorded at the foundation during experimental tests. To account for the progressive accumulation of seismic induced damage, the entire sequence of tests was simulated with DEM, by restoring, for the initial state of each test, the deformed geometry and residual stresses-and-displacements of the previous one.

3.3 Results

Only the last series of shake table tests, that is those with a scale factor of 75% for the stone wall and 125% for the tuff wall, were modelled.

Figures 2 and 3 show the comparison between experimental and numerical results in terms of deflection profiles (Figure 2) and horizontal displacement time histories (Figure 3) of a control point at about mid-height of the wall. For both the walls, the outcomes related to the application of the strongest signals – NCR and AQV – are illustrated. A good agreement was found in terms of deflection profiles and position of the hinge along the wall height (Figure 2), although the maximum displacement exhibited by the wall, in some cases, was not accurately estimated by the model (Figures 2c,d). Numerical relative displacement time histories demonstrated a good match with the experimental ones in terms of frequency and, in most cases, also in terms amplitude of displacement (Figure 3).

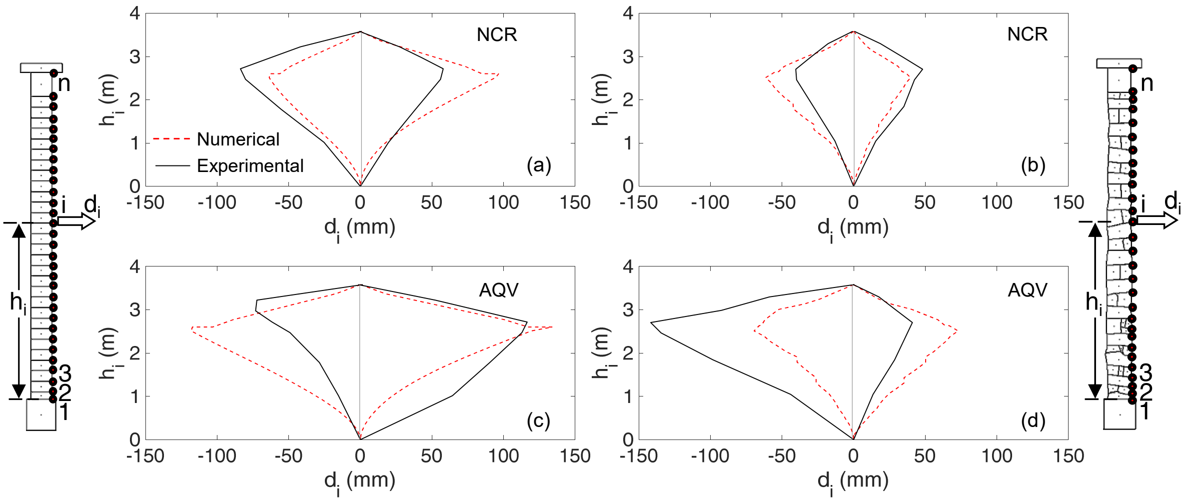

Figure 2: Deflection profiles results for tuff (a,c) and stone (b,d) walls: experimental vs. numerical results.

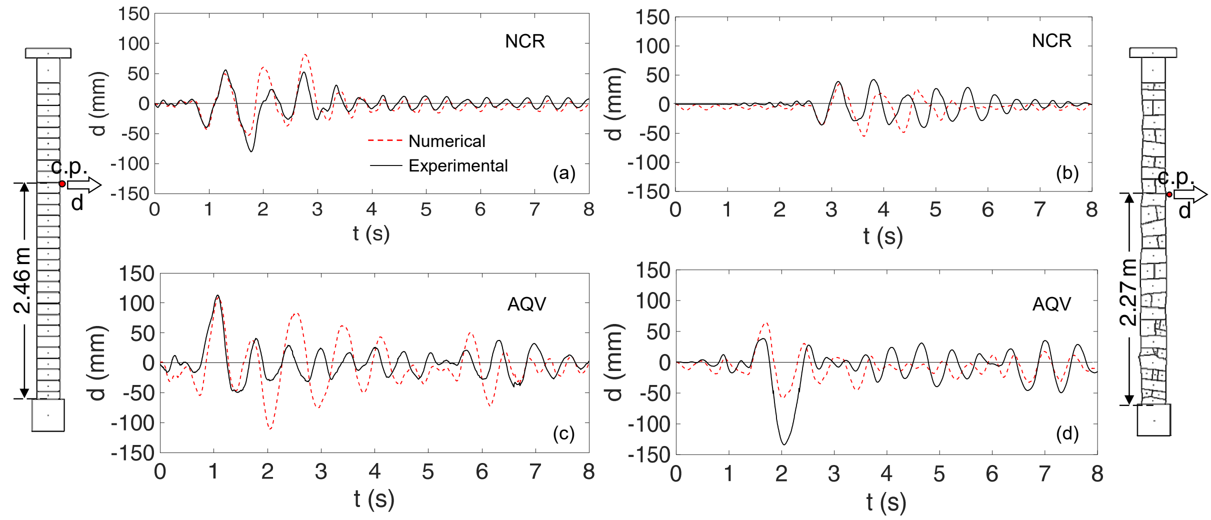

Figure 3: Experimental vs. numerical results relative to horizontal displacement time histories for tuff (a,c) and stone (b,d) walls.

4 INVESTIGATION OF THE OUT OF PLANE OVERTURNING FAILURE MECHANISM

The second application deals with the study of the out-of-plane overturning mechanism of masonry façades. This investigation was performed through nonlinear static (pushover) analysis. The case study is the Church of St. Maria Maggiore in Tuscania, Italy.

4.1 The case study of St. Maria Maggiore in Tuscania

St. Maria Maggiore is a Romanesque church, built in the IX Century and located in Tuscania, Central Italy.

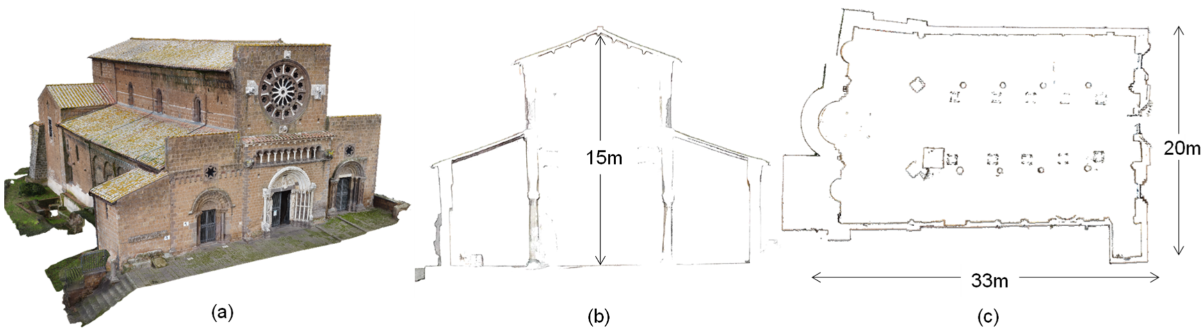

Figure 4: The case study of St. Maria Maggiore in Tuscania, Italy: the church (a), transversal section (b) and plan (c).

After many transformations over years, the latest of which was due to the 1971 Tuscania earthquake, the present configuration of the church is represented in Figure 4. It is made of three naves with timber roof, the tallest of which (central nave) is 15m high, and of a presbytery comprised of three apses. In plan, the church is 20m wide and 33m long. Almost all the structural elements of the church are made of tuff masonry and the average thickness of the walls is about 0.7-0.8m.

As is quite common in churches, one of the most vulnerable parts of St. Maria Maggiore is represented by the main façade, object of the present study.

4.2 Integrated methodology for non-linear static analysis

An integrated methodology was used to investigate the out-of-plane overturning failure mode of the main façade of the church (Figure 5).

This consists of a complete photogrammetric survey – both aerial and ground – of the building and its Distinct Element modelling. In the specific case of the present work, the methodology was used to study the seismic behaviour of the façade, but it can be adopted also for all the other parts of the church.

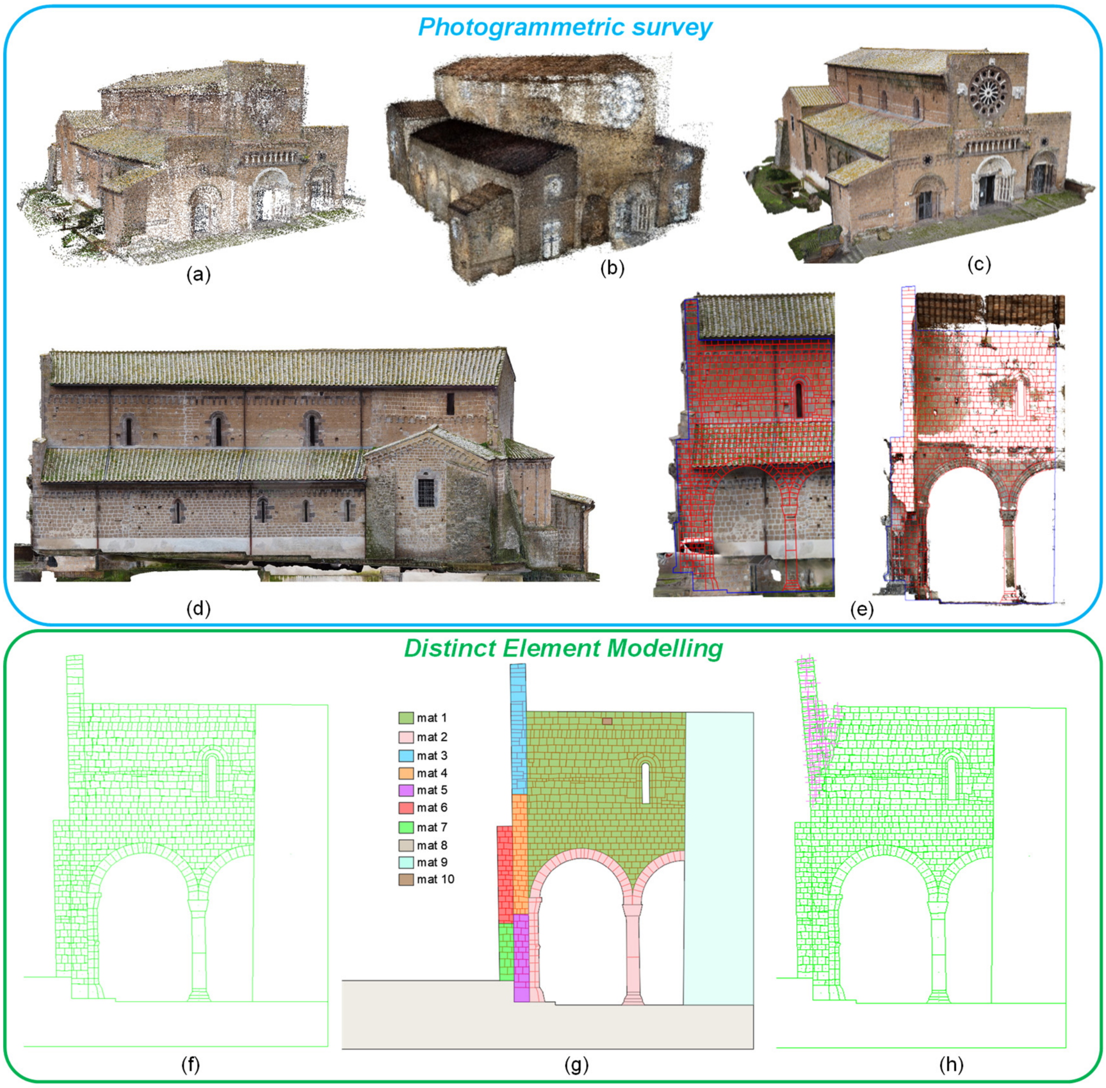

Figure 5: Integrated methodology for non-linear static analysis. Photogrammetric survey – point cloud of the exterior (a) and the interior (b) of the church, 3D meshed-and-textured model (c), orthophoto (d), block mesh representation in CAD (e); DEM modelling – geometry (f), material assignment (g), pushover analysis (h).

The photogrammetric survey allows to obtain both the point clouds (Figures 5a,b) and the meshed-and-textured model of the church (Figure 5c). From the textured model it is possible to get orthophotos (Figure 5d), which are imported in CAD to derive the block mesh of the element of interest (in this case, the façade and a part of the transversal walls, Figure 5e).

The block mesh is then imported in the code UDEC® through an automatic open-source online tool and the geometric model is obtained (Figure 5f). Next, material properties are assigned (Figure 5g), by taking into account the third dimension of the model (see §3.2) and the Pushover analysis is performed (Figure 5e).

An automatic algorithm, published in Gobbin et al. (2021) [17], is used to perform the pushover analysis of the façade. This procedure is able to provide the whole capacity curve of the structure, namely both the ascending and the softening branch, representing the acceleration and the horizontal displacement of a control point in equilibrium. The non-linear static analysis is basically performed in three main steps. First, gravity is applied to the model. Afterwards, a distribution of mass-proportional horizontal forces is applied. By incrementally increasing these forces, ensuring that the quasi-static equilibrium is maintained for each force value, the ascending branch is obtained. Once the peak acceleration is achieved, the mechanism is activated, and the softening branch is gained by gradually updating the horizontal forces, so that the kinetic energy of the structure remains limited and the quasi-static condition is ensured. The last point of the softening branch (zero acceleration of the structure) corresponds to the displacement capacity.

4.3 Results

The methodology described in §4.2 provided, as final result of the pushover analysis, both the collapse mechanism (Figure 6a) and the capacity curve (Figure 6b) of the façade.

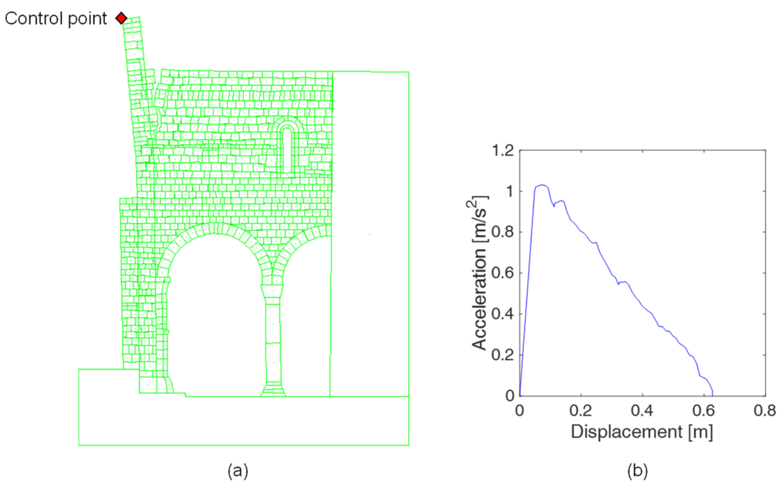

Figure 6: Non-linear static (pushover) analysis: failure mechanism (a) and capacity curve (b).

Under the quasi-static out-of-plane horizontal loads the façade demonstrated to fail by the overturning of its upper part, due to the poor interlocking with the orthogonal walls, as highlighted by the formation of an almost vertical crack in the failure zone.

To this failure mechanism, the capacity curve plotted in Figure 6a corresponds. In this curve, the imposed acceleration is plotted against the horizontal displacement of a control point, positioned on top of the wall. The acceleration capacity of the façade resulted to amount to about $1 m/s^2$, whereas the maximum displacement attained by the control point is slightly higher than 0.6m.

5 CONCLUSIONS

In this paper, the potentialities of Distinct Element Method (DEM) for investigating the out-of-plane seismic behaviour of masonry façades were illustrated. In particular, the attention was focused on the vertical bending failure mode and on the overturning mechanism.

The out-of-plane vertical bending mechanism was studied by simulating shake table tests performed on two full-scale masonry walls. DEM proved able to catch with good accuracy both the shape and the position of the hinge of the wall under real input dynamic excitations. A good matching was also found between experimental and numerical horizontal displacement time histories of some selected control points, both in terms of frequency and displacement amplitude.

For the out-of-plane overturning failure mode, the Romanesque church of St. Maria Maggiore in Tuscania, Italy, was selected as case study. In particular, a pushover analysis was performed to a part of the construction to investigate the behaviour of its main façade. For this application, an integrated methodology, which is still under refinement, combining the photogrammetric survey with an automatic pushover algorithm developed in the distinct element code UDEC®, was adopted.

The outcomes of the present research proved the suitability of DEM for studying the out-of-plane seismic behaviour of masonry walls. Indeed, with DEM it is possible to:

(i) Perform both static and dynamic non-linear analyses;

(ii) Reproduce the real morphology of masonry, account for the interlocking/connections between walls and for the actual boundary conditions;

(iii) Catch the actual crack pattern and follow the evolution of the collapse mechanism during the analysis, by also accounting for sliding and disaggregation phenomena.

All these features allow to overcome the limitation of traditional approaches, based on the rigid block mechanics and on the definition a-priori of the collapse mechanism.

Although the proposed DEM modelling procedures, developed in a two-dimensional context, offer reasonable results with low computational costs, an extension to three-dimensional analysis could be developed to make these procedures more suitable for more complex problems.

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