SEISMIC ASSESSMENT OF STONE MASONRY HISTORIC BUILDINGS USING DISCRETE ELEMENT METHOD

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Abstract: Reliable numerical analysis strategies for historical unreinforced masonry buildings under seismic action represent essential tools in engineering practice in order to assess their actual resilience and study their vulnerability to earthquakes. In this note, an approach based on the Discrete Element Method is proposed to assess the seismic fragility of stone masonry buildings: stone masonry is regarded as a set of small discrete elements (masonry units) modelled as rigid blocks, while the nonlinear behaviour is limited to that of the joints. A procedure for performing push-over analysis and obtaining the capacity curve is implemented using the software UDEC, and the results are compared to the classical mechanism method. Eventually, the approach is applied to the library of the Casamari Abbey in Veroli (FR, Italy), a Cistercian complex dating back to the XIII century.

Introduction

The damages suffered by architectural heritage during earthquakes have clearly shown that the seismic fragility of historic masonry building is mainly controlled by local collapse mechanisms: a part of the building detaches from the rest and starts failing according to a rigid body motion. This circumstance was first noticed by Giuffrè (1992) after careful survey of damages suffered by traditional buildings during historical earthquakes. Giuffrè proposed a method for seismic assessment based on kinematic limit analysis. In a first step, the expected failure mechanism is chosen according to the knowledge of the recurring collapse mechanisms or from the survey of the weakness of the specific building. Then, the horizontal acceleration that activates the mechanism is estimated according to the kinematic approach by means of the virtual works principle. This method, which is in continuity with the classical studies of arches and retaining walls mechanics in the XIX sec, succeed in providing a simple analysis tool for the seismic assessment. The limit analysis kinematic approach is now incorporated in the current seismic Italian provisions for evaluating the seismic capacity of local mechanisms. Accordingly, the demand is estimated in terms of acceleration or displacement, having recourse to the concept of substitute structure, i.e. defining the participating mass and stiffness of an equivalent single degree of freedom system. The main drawback of this procedure consists in the dependence of the results on the shape of the collapse mechanism, i.e. the position of the hinges and that of the cracks, which delimitate the portion that fails. An even slight change in the hinge position or in the crack slope could affect the result in terms of collapse acceleration and displacement capacity. Therefore, it is not satisfactory to leave this choice to the "engineering judgment" of the designer, regardless of the quality of masonry or the efficacy of the connections between perpendicular walls. There is the definite need for an assessment tool, which is capable in detecting the effective shape of the collapse mechanism that is expected to take place, according to the geometry and quality of masonry.

The present paper aims at proposing a new approach for the seismic assessment of masonry structures based on the discrete element method. Through a careful reproduction of the effective geometry and arrangement of stone masonry, the discrete element method proves to be a refined assessment tool for seismic behaviour. The proposed approach is finally applied to a case study and compared to current assessment tools.

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Discrete Element Analysis

Numerical modelling of the seismic behaviour of masonry structures represents a very complex problem due to the anisotropic characteristics of masonry and its highly physical and geometrical nonlinear behaviour when subjected to strong ground motion. There are two fundamental approaches to model such a complex material: equivalent continuum, by means of appropriate constitutive relations, and discontinuum idealization, which are also denoted as macro and micro-modelling. The 'discontinuum' approach, in which the material is viewed as an assembly of discrete bodies, is primarily used in research, but it is increasingly applicable to real structures. The approach succeed in reproducing important phenomena such as crack opening and joint sliding, which are responsible of structural damage and collapse. Among the different micro-modelling approaches, the Discrete Element Method (DEM) is presently used for its capability to accurately represent the effective geometrical features of the structure, to follow the deformation process and to detect the expected failure mode (Lemos 2007; Azevedo et al. 2000, de Felice 2011). Furthermore, the DEM is suitable for problems characterized by large displacements and rotations between blocks, since the block position and joint interactions are automatically updated as the calculation progresses.

In the present paper the mechanical behaviour of stone masonry is modelled by means of the software UDEC (Universal Discrete Element Code) in which the blocks are assumed as 2D rigid elements and the joints as 1D interfaces with Coulomb friction. In a first step, the macro-elements that are expected to exhibit collapse mechanisms are selected. For each macro-element, the morphology of the masonry texture and the reproduction of the effective shape and arrangement of the stones within the wall is carried out. For this purpose, starting from a CAD reproduction of the masonry texture, a pre-processing code is developed to automatically generate the mesh. The stones are modelled as polygonal rigid bodies and the joints as interfaces between the blocks, placed at the middle of the joint thickness. Once the geometry is obtained, the mechanical properties of the joints and blocks are defined taking into account the effective depth of the macro-element. Only a few constitutive parameters are requested to define the non-linear behaviour of the joints: the friction angle, normal and tangential stiffness, while the joint cohesion and tensile strength are neglected.

The non-linear static analysis is carried out by applying gravity first and then horizontal increasing acceleration in successive steps. At the end of each step the equilibrium configuration is reached by explicit integration of the equation of motion. During the analysis some contacts may be lost when the blocks move apart and new contacts may be created as the blocks come closer, which are automatically recognized in the solution scheme. This is the principal computational cost in DEM simulation and it introduces nonlinear effects, which are not taken into account by the standard modelling codes based on the small displacement hypothesis. Static solutions are obtained using artificial damping to reach the equilibrium state as soon as possible. Once the last equilibrium path is reached under increasing horizontal acceleration, a further load step activates the collapse mechanism, which can be followed up to the attainment of the ultimate displacement and then to failure. In the following paragraphs the obtained collapse mechanism will be compared in terms of horizontal acceleration and ultimate displacement to the value provided by the kinematical approach carried out according to Italian standards. The comparison allows to evaluate the reliability of the proposed procedure, underlying the main differences with current assessment tools.

The mechanism method

The mechanism method for assessing the out-of-plane seismic capacity of masonry buildings under seismic action was originally proposed by Giuffrè (1992) and was recently incorporated in the Italian standards (CSLPP 2009). The method, based on the limit analysis kinematic approach, involves two main assumptions: i) the choice of the rigid-body collapse mechanism that is expected to occur; ii) the possibility of describing the collapse mechanism as an equivalent Single Degree of Freedom (SDOF) elastic system.

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With regard to the first issue, the choice of the mechanism is questionable. According to limit analysis (Heyman 1982), all the mechanisms should be investigated in order to get a reliable estimation. However, in current practice, only a few mechanisms are usually investigated, moreover, the results could be strongly affected by the quality of masonry and the efficacy of the connections between the masonry walls. Clearly, the expected failure mechanism can be devised by the knowledge of the seismic behavior of similar structures that were already damaged by the earthquake, as well as by taking into account the presence of existing cracks, including non-seismic ones, as preferential weakness planes.

As for the second issue, the equivalence appears to be questionable: many contributions have shown the conceptual differences between rocking systems and elastic oscillators (see for instance: Makris and Kostantinidis 2003) and some attempts to overcome this limits was recently proposed (Mauro et al. 2015).

For a given mechanism, the current assessment method involves the following steps:

- transformation of a portion of the structure in a kinematic chain, through the identification of rigid bodies, defined by fracture planes assumed to be able to rotate or to slide between them;
- evaluation of the horizontal load multiplier α_0, which activates the mechanism;
- estimation of the ultimate displacement $d_{k,0}$, after which failure occurs in the absence of horizontal seismic action ($\alpha = 0$); a reference point of the kinematic chain has to be defined for this purpose, usually chosen at the center of gravity;
- construction of the load displacement (α-d) capacity curve, i.e. the equilibrium path expressing the horizontal load which the structure is able to withstand with the evolving of the mechanism;
- identification of the participating mass M^* and period T_s of the equivalent SDOF system;
- representation of the capacity curve in terms of spectral acceleration a_0^* and displacement d^*;
- security checks, to be carried out either in terms of acceleration or in terms of displacement.

The capacity curve is usually a linear decreasing slope given as follows:

$$\alpha = \alpha_0 (1 - d_k / d_{k,0}) \tag{1}$$

The load-displacement curve is transformed into the capacity curve a_0^*-d^* of an equivalent SDOF system as described by eq.(2) and eq.(5).

According to the Italian standards, the spectral acceleration a_0^* is obtained by multiplying the load multiplier α_0 for the acceleration of gravity g and dividing it by a confidence factor FC, which depends on the level of knowledge of the structure, and by the participating mass fraction e^* of the mechanism:

$$a_0^* = \frac{\alpha_0 \sum_{i=1}^{n} P_i}{M^* FC} = \frac{\alpha_0 g}{e^* FC} \tag{2}$$

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This last quantity, i.e. participating mass fraction, can be expressed as:

$$e^* = \frac{gM^*}{\sum_{i=1}^{n} P_i} \tag{3}$$

where the participating mass M^* is given by the following equation:

$$M^* = \frac{\left(\sum_{i=1}^{n} P_i \delta_{x,i}\right)^2}{g \sum_{i=1}^{n} P_i \delta_{x,i}^2} \tag{4}$$

in which n is the number of the weights P_i involved in the mechanism, whose masses, during the earthquake, generate horizontal forces, while $\delta_{x,i}$ is the horizontal displacement of the point of application of the $i-th$ weight P_i.

Accordingly, the spectral displacement of the equivalent oscillator d^* can be obtained as the average displacement of the different points at which the weights P_i are applied, weighed on P_i.

$$d^* = d_k \frac{\left(\sum_{i=1}^{n} P_i \delta_{x,i}^2\right)}{\delta_{x,k} \sum_{i=1}^{n} P_i \delta_{x,i}} \tag{5}$$

In eq.(5), for a given configuration of the kinematic chain, $\delta_{x,i}$ denotes the horizontal displacement of the points at which the weights are applied, $\delta_{x,k}$ is the corresponding horizontal displacement of the point k, taken as a reference for determining the displacement d_k.

For the strength-based procedure, the check consists in verifying the following condition:

$$a_0^* \geq \frac{a_g S}{q} \tag{6}$$

where a_g is the expected Peak Ground Acceleration (PGA), S is a coefficient taking into account the category of subsoil and topographical conditions, and $q = 2$ is a behaviour factor that takes into account the reserve of capacity of the structure from the activation of motion up to the collapse by overturning.

The more refined security checks according to the displacement-based procedure consists in the comparison between the spectral displacement demand $S_{De}(T_s)$ and spectral displacement capacity d_u^* , estimated as a ratio of the theoretical ultimate spectral capacity ($d_u^* = 0.4 \, d_0^*$):

$$d_u^* \geq S_{De}(T_s) \tag{7}$$

The spectral displacement demand is the ordinate of the elastic displacement spectrum corresponding to the period T_s of the equivalent SDOF system (i.e. the secant period of the mechanism, not that of the building).

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Case study

In the following paragraph, a masonry wall belonging to the Casamari abbey is studied. In figure 1 a global plan of the abbey and a picture of the section chosen are reported. The section is a part of the library and, even if is constrained by buttresses, is particularly vulnerable when subjected to seismic action, due to its height. In the following sections, a comparison between the capacity curve obtained by the DEM approach and those provided by the mechanism method is carried out in order to verify the reliability of the numerical method and to underline the differences with respect to current assessment tools.

The main advantage of the proposed procedure consists in obtaining the collapse mechanism based on the real geometrical and mechanical properties of the masonry, without assuming any given failure crack. The method allows in overcoming the uncertainties on the expected crack pattern that, even in this simple case, proves to affect significantly the overturning capacity and for more complex structures may become hard to detect.
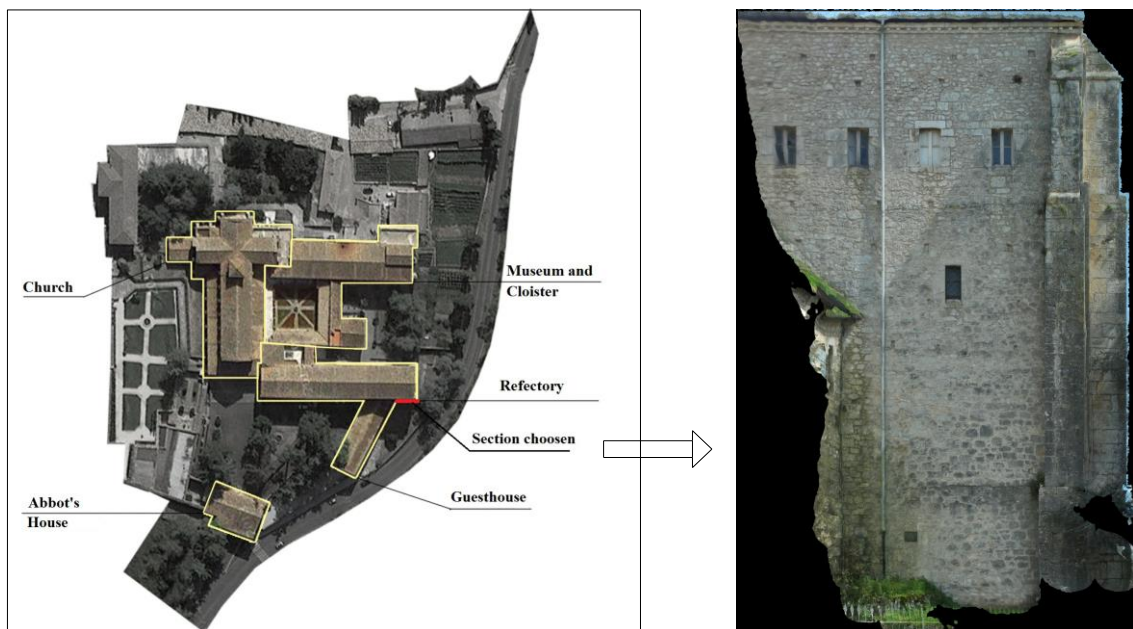

Figure 1: Global plan of Casamari abbey

Definition of the Discrete Element Model

The first step consists in the definition of the discrete element model, according to the real morphology of the façade, by using information obtained from an historical-critical analyses of the masonry types identified and the available photographic survey. As the model is plane, the effective thickness t of masonry is considered for assigning the density ρ of the discrete elements as well as the normal and tangential stiffness k_n and k_s of the joints, in order to take into account the three-dimensionality of the problem. The cohesion and tensile strength of the mortar joint is neglected, while the friction angle is assumed equal to 30° (see figure 2).
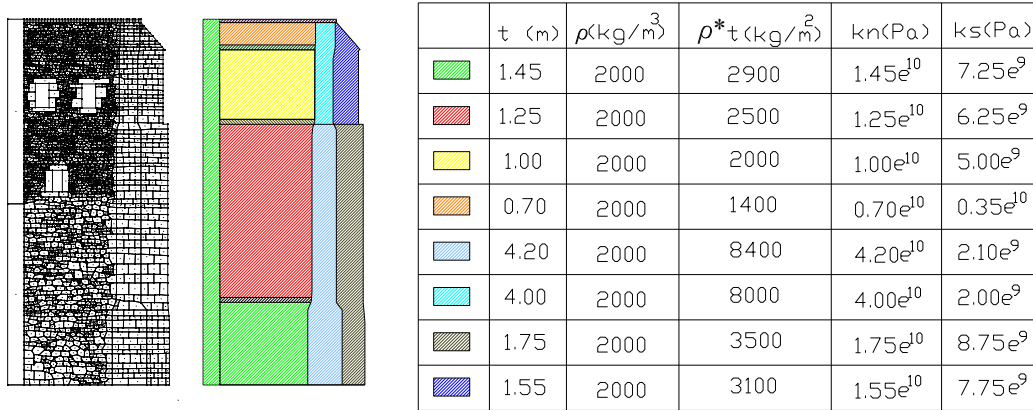
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	t (m)	ρ(kg/m^3)	ρ^*t(kg/m^2)	kn(Pa)	ks(Pa)
(green)	1.45	2000	2900	$1.45e^{10}$	$7.25e^9$
(red)	1.25	2000	2500	$1.25e^{10}$	$6.25e^9$
(yellow)	1.00	2000	2000	$1.00e^{10}$	$5.00e^9$
(orange)	0.70	2000	1400	$0.70e^{10}$	$0.35e^{10}$
(light blue)	4.20	2000	8400	$4.20e^{10}$	$2.10e^9$
(cyan)	4.00	2000	8000	$4.00e^{10}$	$2.00e^9$
(gray)	1.75	2000	3500	$1.75e^{10}$	$8.75e^9$
(blue)	1.55	2000	3100	$1.55e^{10}$	$7.75e^9$

Figure 2: Masonry geometry and mechanical properties

The push-over analysis is carried out by applying increments of horizontal acceleration in successive steps up to overturning. The masonry portion involved in the mechanism is directly provided by the analysis on the basis of the effective arrangement and shape of the stones and the mechanical characteristics of the joints (see figure 3). The corresponding push-over curve is obtained, comprising an ascending branch which collects the equilibrium points reached at each load step of the analysis, and a descending branch obtained by joining the last equilibrium point with the ultimate displacement corresponding to the unstable configuration of the structure (de Felice and Mauro 2010).
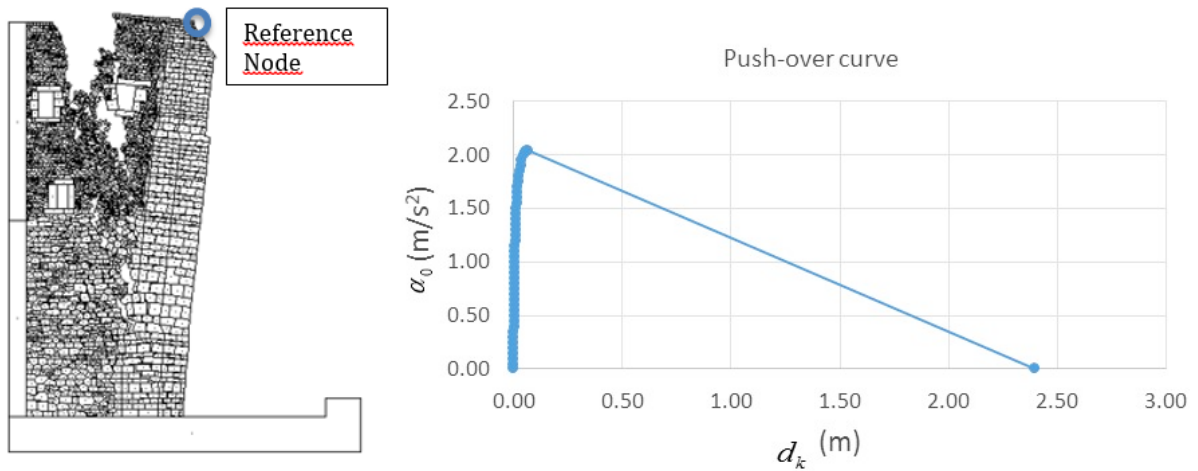

Figure 3: DEM collapse mechanism

Comparison with the mechanism method

The same portion of the Casamary library is analysed by using the commercial software Mc4Loc in which the seismic capacity is evaluated according to the mechanism method as proposed by Italian standards. In particular, three different collapse mechanisms are selected (see figure 4) having increasing slope of the crack, in order to check the influence of the mechanism on the assessment results.
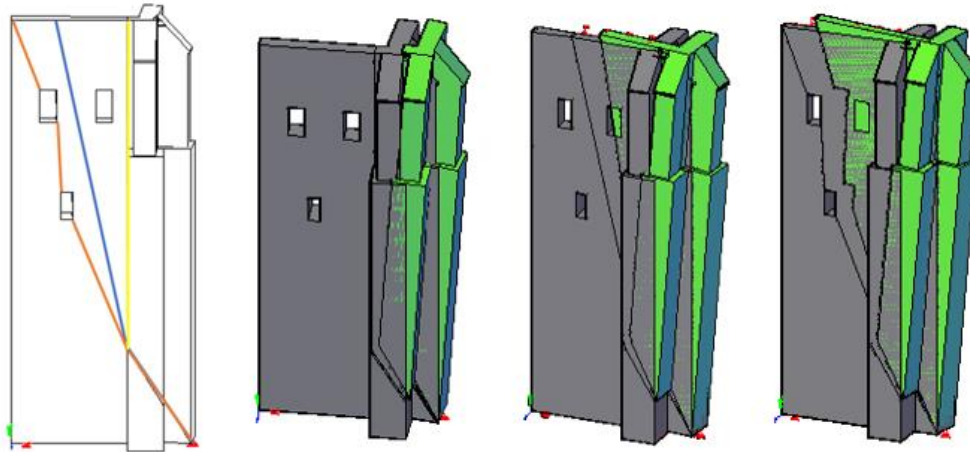
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Figure 4: Different collapse mechanisms

The capacity curves for the three different mechanisms expressed in spectral coordinates a_0^* (see eq.(2)) and d_u^* (see eq.(7)) are plotted in figure 5. Clearly, as far as the slope of the crack decreases, the capacity curve increases correspondingly, both in terms of load multiplier and ultimate displacement. The capacity curve obtained by the DEM analysis fits with those provided by the mechanism method curve 3 in terms of acceleration and curve 1 in terms of displacement capacity.
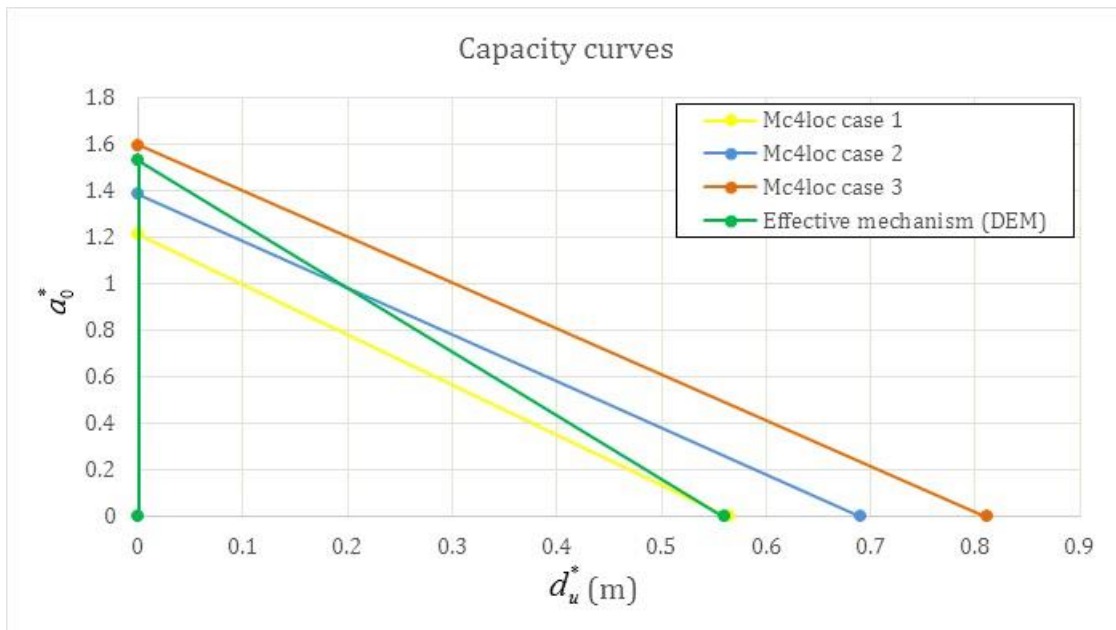

Figure 5: Comparison between capacity curves

For the DEM analysis the participation mass M^* and the participation factor e^* are evaluated according to eq.(4) and eq.(3) respectively, by considering all the weights of the blocks that are involved in the collapse mechanism (see figure 6). However, the values of M^* and e^* vary as the mechanism progresses, according to the effective motion of all the blocks involved in the mechanism. For instance, the variation of the participating mass M^* recorded from the upset of motion up to the limit equilibrium configuration, is plotted in figure 7. As the mechanism

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progresses, the variation of the participating mass M^* is not particularly relevant amounting to the value obtained at the activation of the mechanism.

The comparison between DEM analysis and the mechanism method is summarized in table 2. As far as regards DEM analysis, the quantities listed in Table 2 are related to the acceleration which activates the collapse mechanism while for the mechanism method, the three cases refer to an increasing slope of the crack. The comparison clearly shows the consistency of the DEM approach in providing a reliable estimation of the overall mechanical quantities which define the collapse mechanism and can be used for security checks.

	DEM	Mc4loc - Case 1	Mc4loc - Case 2	Mc4loc - Case 3
M^* (kg)	347620	306696	361179	404598
e^*	0.99	1	1	0.99
T_s (s)	2.60	2.96	3.07	3.10
a_0^* (m/s^2)	1.53	1.21	1.38	1.60
d_u^* (m)	0.56	0.56	0.69	0.81

Table 2: Comparisons between different models

Figure 6: Collapse configuration

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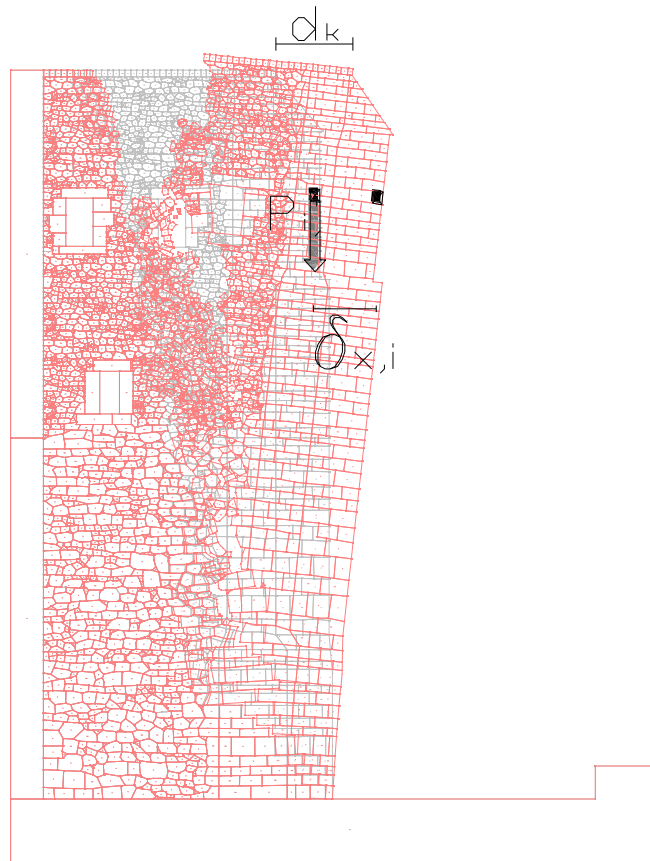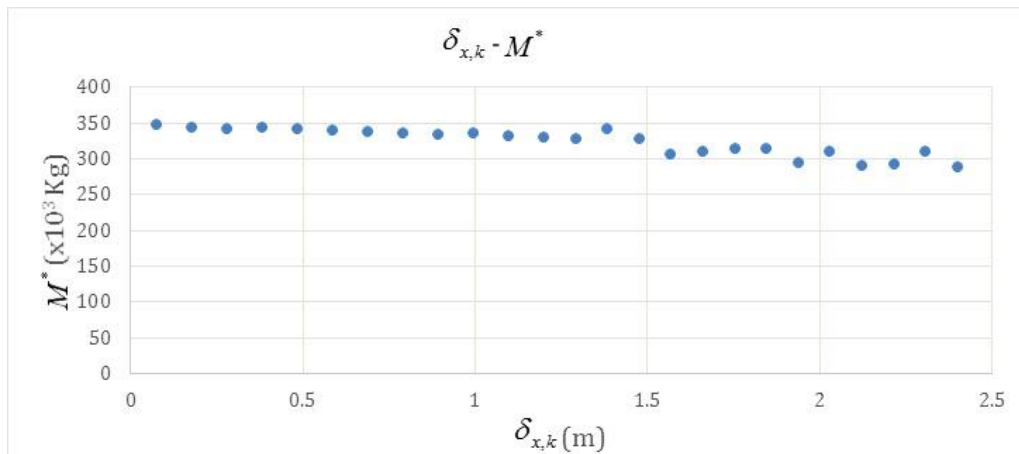
Figure 7: Participation mass in relation to the horizontal displacement of the reference node

Conclusions

An approach based on the Discrete Element Method is proposed for the seismic assessment of historical masonry buildings. The main appeal of this approach lies in its ability to reproduce the effective collapse mechanism of masonry by means of elementary mechanics and a few constitutive parameters with a clear physical meaning. The assessment is essentially based on the effective geometry of the structure and on the quality of masonry, rather than on the strength of the material. Indeed, a careful definition of the discrete element mesh for reproducing the effective arrangement of stone masonry, is required.

As a case study, the evaluation of the seismic capacity of a portion of the library of the Casamari abbey is carried out and the results are compared to current approaches based on the mechanism method. The comparison reveals the reliability of the DEM as a tool for the refined assessment of masonry structures subjected to seismic loading.

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