12th International Conference on Structural Analysis of Historical Constructions
SAHC 2020
P. Roca, L. Pelà and C. Molins (Eds.)

# SIMULATION OF THE OUT-OF-PLANE BEHAVIOUR OF URM WALLS BY MEANS OF DISCRETE MACRO-ELEMENT METHOD

**CÉSAR CHÁCARA[1*], BARTOLOMEO PANTÒ[2], FRANCESCO CANNIZZARO[3], DAVIDE RAPICAVOLI[3], IVO CALIÒ[3] AND PAULO B. LOURENÇO[4]**

[1] Civil Engineering Division, Department of Engineering,
Pontificia Universidad Católica del Perú (PUCP)
Avenida Universitaria 1801, San Miguel, Lima 15088, Perú
e-mail: c.chacara@pucp.pe, www.pucp.edu.pe (*corresponding author)

[2] Department of Civil and Environmental Engineering, Imperial College London
South Kensington Campus, London SW7 2AZ, United Kingdom
email: bpanto@dica.unict.it, www.imperial.ac.uk

[3] Department of Civil Engineering and Architecture, University of Catania
Viale Andrea Doria 6, 95125 Catania, Italy
e-mail: {francesco.cannizzaro, ivo.calio@unict.it, davide.rapicavoli}@dica.unict.it, www.unict.it

[4] ISISE, Department of Civil Engineering, University of Minho
Azurem, 4800-058 Guimarães, Portugal
email: pbl@civil.uminho.pt, www.uminho.pt

**Keywords:** Nonlinear dynamic analysis, Parametric assessment, HiStrA software

**Abstract.** *The seismic response of masonry structures without box-type behaviour is given by a complex interaction between in-plane and out-of-plane behaviours. Previous earthquakes demonstrated that out-of-plane failure mechanisms represent the main cause of structural collapses of UnReinforced Masonry (URM) and historical structures. Previous experimental and analytical studies, investigating the out-of-plane behaviour of URM structures, mostly considered the effects of one-way bending moment. In this regard, recent experimental campaigns and numerical simulations have been conducted in order to investigate the out-of-plane behaviour of masonry walls subjected to two-way bending. These investigations have demonstrated the complexity of this mechanism and stressed the need for accurate numerical tools capable of providing reliable predictions in terms of ultimate strength and failure mechanisms. This paper focuses on the assessment of the dynamic behaviour of a U-shape URM prototype, subjected to shaking table tests, by means of a simplified computational strategy denoted as Discrete Macro-Element Method (DMEM). In this investigation, a comparison between experimental and numerical results was conducted in order to validate the capabilities of the proposed modelling approach. Subsequently, a parametric analysis was carried aiming at determining the influence that masonry mechanical properties, and additional model parameters, have on the out-of-plane nonlinear dynamic response of URM masonry structures.*

## 1 INTRODUCTION

UnReinforced Masonry (URM) buildings are characterized by a complex behaviour when subjected to dynamic loading. This behaviour is given by the high nonlinearity of the material together with the interaction between in-plane and out-of-plane mechanisms. There is a clear comprehension of the in-plane response of this typology of structures due to the different investigations that have been conducted experimentally, analytically and numerically. The out-of-plane behaviour of URM structures has been investigated by means of analytical and experimental formulations, mainly focusing on the effect of one-way bending loading [1-4]. During the last decades, there has been an increasing interest in understanding the two-way bending out-of-plane behaviour of these structures when subjected to seismic loading, as reported in recent experimental campaigns [5-7]. As reported in [8], numerical simulations were conducted by means of different methods aiming at predicting the response of the two-way bending out-of-plane response of URM structures. However, it was evidenced that this type of behaviour presents a significant complexity due to the diverse results obtained from the numerical simulations. In this sense, there is the necessity of understanding this particular behaviour in order to conduct proper and reliable predictions.

This paper consists of the evaluation of the two-way bending out-of-plane response of an URM structure by means of an original numerical technique known as the Discrete Macro-Element Method (DMEM) [9]. This modelling approach is based on a limited number of elements which allows the application of sophisticated nonlinear analyses with a reduced computational demand. The case study of this investigation corresponded to a U-shape clay brick masonry structure which was subjected to out-of-plane shaking table tests as reported by Graziotti, et al. [7]. Due to the available data of such experimental campaign [10], the numerical model of this structure was subjected to time history analyses based on the application of the signals recorded during the shaking table tests. The comparison between experimental and numerical responses was carried in terms of history of horizontal top displacement. In this investigation, a parametric assessment was conducted in order to determine the influence of some mechanical and model parameters on the nonlinear dynamic response of such a structure.

## 2 DISCRETE MACRO-ELEMENT METHOD (DMEM)

An innovative computational tool for the assessment of the seismic response of masonry structures, based on a reduced number of DOFs, was initially conceived by Caliò, et al. [11]. The initial formulation of this modelling approach was based on the hypothesis that masonry structures are characterized by a box-type behaviour due to the presence of a rigid diaphragm. Such initial formulation consisted of 2-dimensional panels constituted by hinged rectangles assembled with four rigid edges and two diagonal links. As shown in Figure 1a, the interaction between panels is given by zero-thickness interface elements. These interface elements are composed by a discrete distribution of transversal links (orthogonal to its length) and an additional single longitudinal link. The initial formulation of this modelling approach is capable of accurately simulating the main in-plane mechanisms of this typology of buildings: flexural, shear-diagonal, and shear-sliding. At an interface level, the discrete distribution of transversal links rules the flexural response, whereas the longitudinal link simulates the in-plane shear-sliding response. At an element level, the shear-diagonal response is given by a couple of diagonal links. The kinematic of the initial formulation is described by four DOFs: three related to the rigid body motion, and the remaining one associated with the in-plane shear deformability.

Nevertheless, the seismic response of most URM buildings and historical constructions is often governed by the interaction of in-plane and out-of-plane mechanisms. As depicted in Figure 1b, this modelling approach was upgraded aiming at accounting for the out-of-plane mechanisms by the introduction of a new interface element [12]. The interface element is composed by a matrix of transversal links which simulate the bi-flexural response, a single link along its length, which governs the in-plane shear-sliding, and a couple of links along its thickness, which rule the out-of-plane shear deformability and torsion responses. In the upgraded version of this modelling approach, the in-plane shear-diagonal response is simulated by a single diagonal link placed at each 3-dimensional panel. The kinematic of the upgraded panel is described by seven DOFs: six associated with translational and rotational rigid body motion, and one related to the in-plane shear deformability. The proper simulation of these mechanisms requires careful calibration procedures for the different sets of links which are thoroughly described in [9].

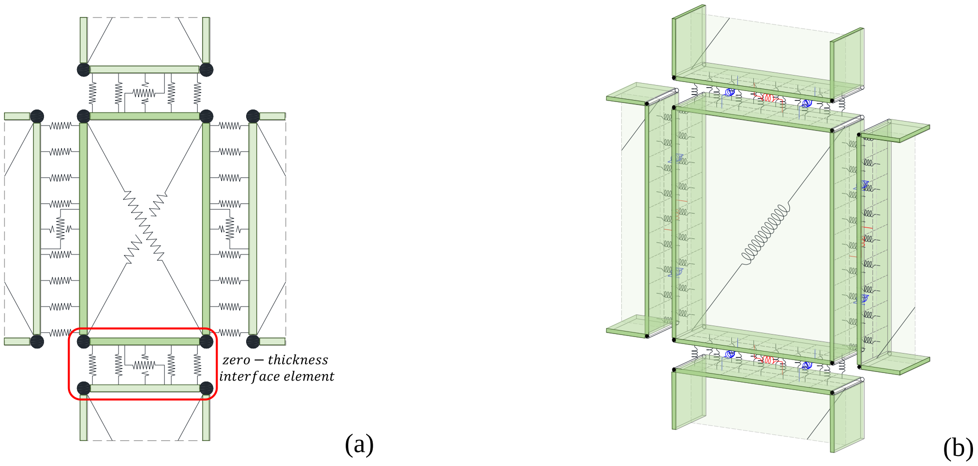
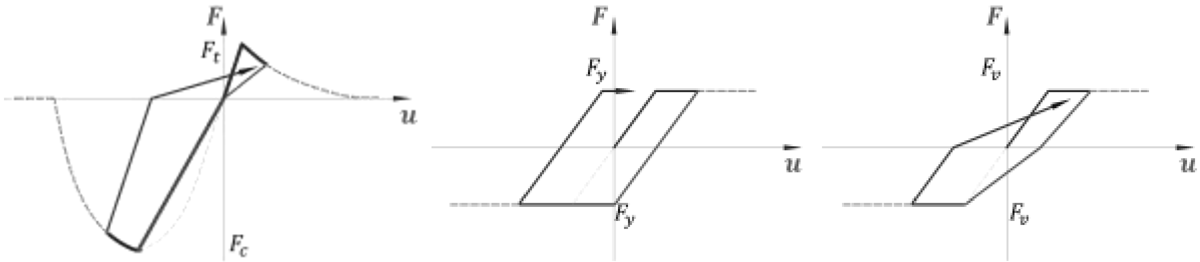



**Figure 1**: Mechanical scheme of the DME modelling approach: (a) two-, and (b) three-dimensional panels.

In the proposed modelling approach, the nonlinear and cyclic behaviours are focused on the different sets of links. In the case of the transversal links, the tensile response is described by an exponential curve, whereas the compressive response is given by a parabolic curve; both described by a fracture energy approach. The hysteretic behaviour of the transversal links is ruled by a Takeda model [13] (see Figure 2a) which is described by an unloading parameter denoted as $\beta$ whose value can range between 0 and 1. When $\beta$ is equal to 1, the unloading cycle is described by a stiffness oriented to the origin, and when $\beta$ is equal to 0, the unloading cycle is ruled by its initial stiffness. A linear combination between the initial stiffness and the stiffness oriented to the origin is defined when the unloading parameters $\beta$ presents a value higher than 0 and lower than 1. The nonlinear behaviour of the sliding links is characterized by a Mohr-Coulomb criterion due to the frictional phenomenon of this type of response, and the cyclic behaviour is given by an elasto-plastic hysteretic model (see Figure 2b) Finally, the nonlinear behaviour of the diagonal link can be described by a Mohr-Coulomb or a Turnsek and Cacovic

[14] yielding criterion, whereas its cyclic behaviour is also ruled by a Takeda model (see Figure 2c). A detailed description of the constitutive and hysteretic models of the different sets of links can be found in [15].This modelling approach has been implemented in the software HiStrA (Historical Structure Analysis) [16].




(a) (b) (c)

**Figure 2**: Cyclic constitutive models of: (a) transversal, (b) sliding, and (c) diagonal nonlinear links.

## 3 OUT-OF-PLANE URM WALL

The present study consisted on the assessment of the seismic response, by means of the DMEM, of one of the U-shape URM structures reported in the experimental campaign conducted by Graziotti, et al. [7]. In such investigation, calcium silicate and clay brick walls, with different boundary conditions and pre-compression loads, were subjected to a series of shaking table tests aiming at evaluating their out-of-plane behaviour. These walls presented two return walls in order to consider the two-way bending effects on their dynamic response. The clay brick structure, denoted as CL-000-RF in [7], was selected as case study for the present investigation. As illustrated in Figure 3a, this structure presented a main wall with a base of 4.02 m, a height of 2.76 m, and a thickness of 0.10 m, which was restrained by two return walls with a length of 1.00 m which were subjected to a pre-compression load of 0.05 MPa. In this sense, it was considered that the main wall was characterized by a boundary condition in which three edges were fixed, and the remaining one was free (see Figure 3b). The CL-000-RF structure was subjected to 23 shaking table tests aiming at assessing its out-of-plane seismic response as well as the behaviour of the first natural frequency due to cumulative damage.

Graziotti, et al. [7] also conducted a mechanical characterization of this type of clay brick masonry aiming at determining its main mechanical properties such as Young's modulus $E$, compressive strength $f_c$, tensile strength $f_t$, initial shear strength $f_{v0}$, and friction coefficient $\mu$. This mechanical characterization consisted of the application of simple compression, direct tensile wrench, and triplet shear tests. Additionally, torsional tests were applied for the estimation of additional nonlinear parameters such as initial torsional strength $f_{v0,tor}$, and torsional friction coefficient $\mu_{tor}$. As reported in [7], the total mass of this structure presented a value equal to 2178 kg, which corresponds to a specific weight $\gamma$ of approximately 13.5 kN/m$^3$. A summary of the mechanical properties obtained in [7] which were used in this investigation is reported in Table 1.

**Table 1**: Mechanical properties of the clay brick masonry reported in [7]

| $E$ [MPa] | $f_c$ [MPa] | $f_t$ [MPa] | $f_{v0}$ [MPa] | $\mu$ [-] | $f_{v0,tor}$ [MPa] | $\mu_{tor}$ [-] |
|---|---|---|---|---|---|---|
| 7500 | 17.41 | 0.41 | 0.18 | 0.63 | 1.13 | 1.63 |

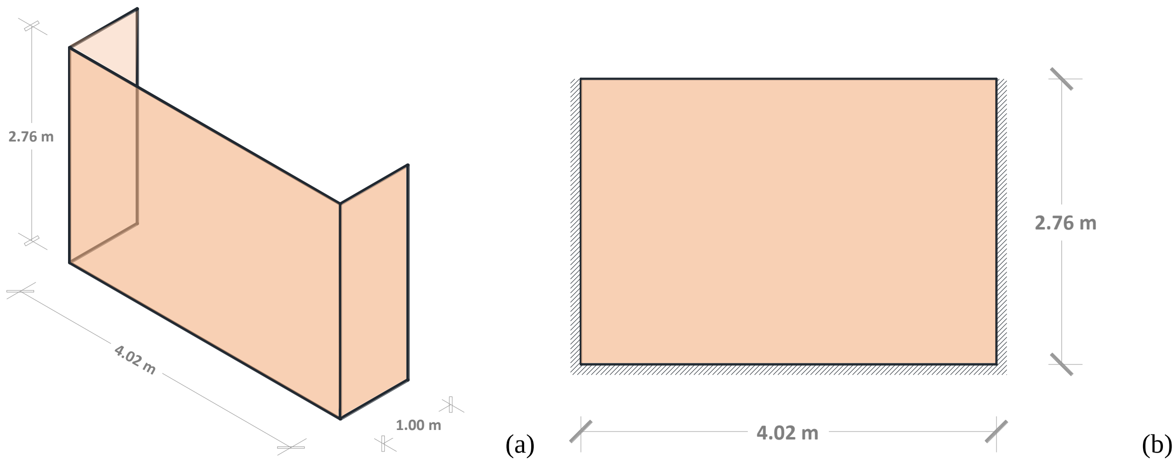



**Figure 3**: Clay brick structure: (a) geometrical characteristics, and (b) boundary conditions

A calibration of the elastic parameters was conducted with the DME model of the U-shape clay brick structure by initially comparing its first natural frequency with the one that was identified in the experimental campaign. Subsequently, the DME model of this structure was subjected to one dynamic loading with low intensity in order to compare the initial elastic response. Such dynamic loading corresponds to the motion registered at the base of the shaking table during Test #4 [10]. The mesh dependency of the DME model on the estimation of the first natural frequency and on the dynamic response was evaluated by considering three different mesh refinements (see Figure 4). Model A presents the less refined discretization, and it is characterized by 168 DOFs, Model B was described by a medium-mesh refinement with 336 DOFs. Finally, Model C represents the most refined mesh presenting 84 elements, and 588 DOFs.

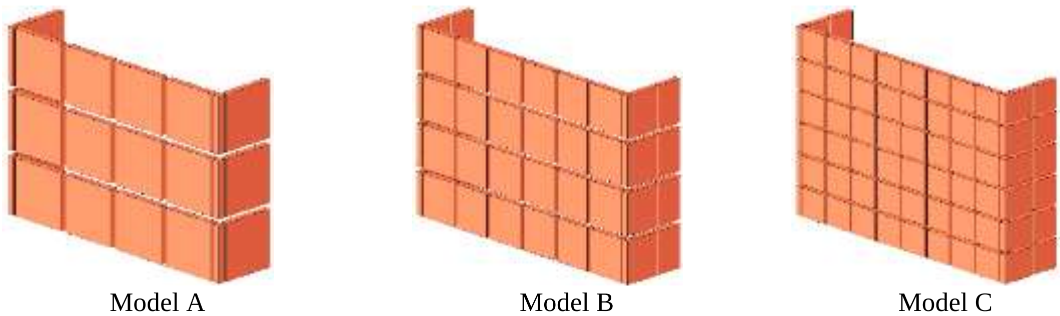



**Figure 4**: DME models of the U-shape clay brick structures

The experimental first natural frequencies of the undamaged CL-000-RF structure, resulted in 12.80 Hz. The first natural frequency of each model was initially evaluated considering the mean value of Young's modulus ($E$ = 7500 MPa*)* obtained from the material characterization and considering a ratio between shear modulus $G$ and $E$ equal to 0.4 (considering Poisson ratio $\nu$ equal to 0.25). The numerical frequencies resulted higher than those experimentally observed. The error was about 15% (15.04 Hz) in the case of Model A and increased when considering more discretized mesh refinements. Namely, an error of 21% (16.14 Hz) and 22% (16.46 Hz) were observed in the case of Model B and Model C, respectively. Hence, it was necessary to decrease the value of the Young's modulus. As reported in Table 2, the dynamic identification was carried out considering two different values of $E$, and it was possible to determine a satisfactory fitting between numerical and experimental results when $E$ = 5000 MPa.

**Table 2**: Natural frequencies of the first vibration mode of the Cl-000-RF structure.

| Young's modulus $E$ [MPa] | Shear modulus $G$ [MPa] | First natural frequency $f$ [Hz] | | |
|---|---|---|---|---|
| | | Model A | Model B | Model C |
| 7500 | 3000 | 15.04 | 16.14 | 16.46 |
| 6250 | 2500 | 13.73 | 14.73 | 15.03 |
| 5000 | 2000 | 12.28 | 13.18 | 13.44 |

Subsequently, the numerical models were subjected to the dynamic motion recorded at the base of the shaking table during the Test #4, along the direction perpendicular to the main wall. This signal was characterized by a low amplitude of intensity, allowing the assessment of the elastic response of the structure. In addition, a Rayleigh damping criterion, with a damping ratio of 3%, and a diagonal mass matrix, in accordance with the kinematics of the proposed modelling approach, were taken into consideration for the application of these analyses. The Rayleigh damping parameters were defined by selecting the first and third natural frequencies of the structure for each numerical model. A constant time step discretization $\Delta t$ = 0.001 sec was adopted for these analyses.

The comparison between numerical and experimental responses, which is illustrated in Figure 5, was focused on the history of horizontal displacement at the top of the main wall. It was observed that Model A presented higher displacements when compared to the experimental ones. These results are due to the large mesh of the models, which does not allow a properly simulation of the torsion effects and the mass distribution. Even though the difference decreased when considering a more refined mesh discretization (Model B), the approximation with the experimental results still lacked a reasonable resemblance. Nonetheless, it was observed that Model C was capable of successfully reproducing the displacements obtained during the experimental campaign; and therefore, this numerical model was selected for the nonlinear parametric assessment.

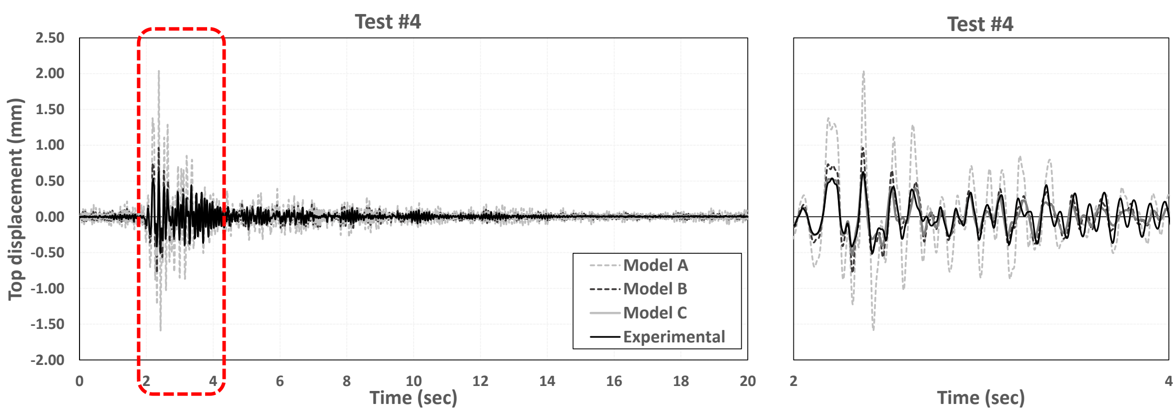



**Figure 5**: Comparison of horizontal top displacement due to the application of Test #4.

## 4 PARAMETRIC STUDY

A parametric assessment was conducted to the Model C considering the signals recorded during Test #18 and Test #21, aiming at reproducing the experimental results for high-intensity motions and identifying the parameters that most influence the nonlinear dynamic response of the prototype. The compressive $f_c$ and tensile $f_t$ strengths of masonry were taken from the experimental mechanical characterization, while the cohesion $c$ and the friction coefficient $\mu$ ruling the shear-sliding mechanism were defined according to the torsional test conducted on brick joints in the experimental campaign. Finally, the strength $f_{y0}$ associated to the shear-diagonal mechanism was considered equal to the tensile strength. A summary of the mechanical properties defined for the DME model is reported in Table 3.

**Table 3**: Mechanical properties of Model C for the parametric assessment

| | | | | |
|---|---|---|---|---|
| Elastic parameters | Young's modulus | $E$ | [MPa] | 5000 |
| | Shear modulus | $G$ | [MPa] | 1667 |
| | Specific weight | $\gamma$ | [kN/m$^3$] | 13.50 |
| Flexural parameters | Compressive strength | $f_c$ | [MPa] | 17.41 |
| | Compressive fracture energy | $G_c$ | [N/mm] | 28.28 |
| | Tensile strength | $f_t$ | [MPa] | 0.41 |
| | Tensile fracture energy | $G_f$ | [N/mm] | 0.012 |
| Shear-diagonal parameters | Shear strength | $f_{y0}$ | [MPa] | 0.41 |
| | Friction coefficient | $\mu_d$ | [-] | 0.6 |
| Shear-sliding parameters | Cohesion | $c$ | [MPa] | 1.13 |
| | Friction coefficient | $\mu_s$ | [-] | 1.63 |

An initial value of tensile fracture energy $G_f$ equal to 0.012 N/mm was considered for the application of these analyses. However, it was noted that the response of the numerical model was characterized by significantly larger displacement for Test #18 and Test #21. In order to assess the influence of $G_f$ on the dynamic response of the brick prototype, a parametric analysis was conducted considering three different values: 0.024 N/mm, 0.036 N/mm and 0.048 N/mm. The results of these analyses are illustrated in Figure 6. It was noted that a tensile fracture energy of 0.024 N/mm provided a good approximation in the case of Test #18; however, the fitting between numerical and experimental results of Test #21 was not in good agreement. An acceptable fitting was obtained adopting the value of 0.036 N/mm and 0.048 N/mm, which were assumed as the referring values of tensile fracture energy. It has to be noted that the latter numerical results presented displacements with a slightly lower amplitude to the experimental ones. In addition, the numerical results were still not capable of simulating the residual displacements that the structure experienced during the shaking table test.

A further parametric analysis aimed at investigating the influence of damping on the nonlinear response of this structure. In this sense, a second frequency associated with a higher vibration mode, with a natural frequency of 131.67 Hz and activating the 50% of the total mass, was selected for the definition of the Rayleigh damping parameters. This variation of the second natural frequency also affected the definition of the incremental time step for the time history analyses, presenting a new value of $\Delta t$ = 0.0005 sec. The comparison between numerical and experimental responses is depicted in Figure 7, with reference to Test #21. It was observed that a significant increment of displacements was obtained when considering $G_f$ = 0.036 N/mm.

This may indicate that the numerical model reached a larger nonlinear response which led to a higher displacement field. On the contrary, this behaviour was not observed when considering $G_f$ = 0.048 N/mm since the increment of top displacement is barely noticeable.

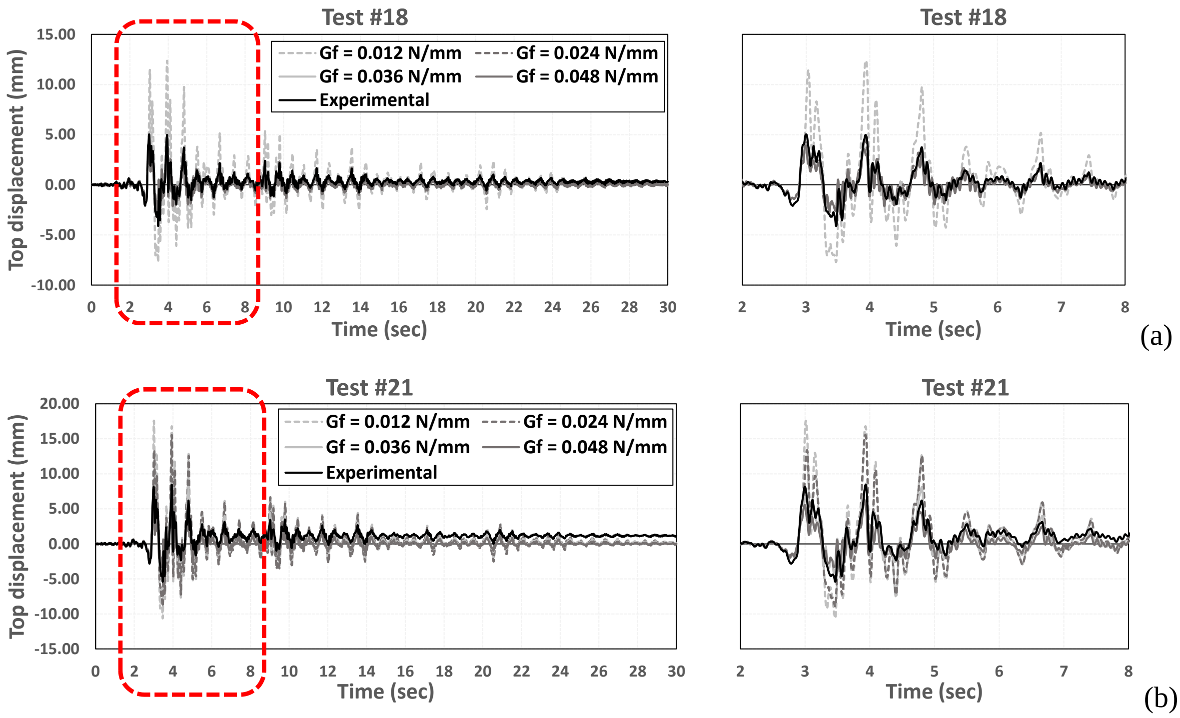



**Figure 6**: Comparison of horizontal top displacement as a function of tensile fracture energy.

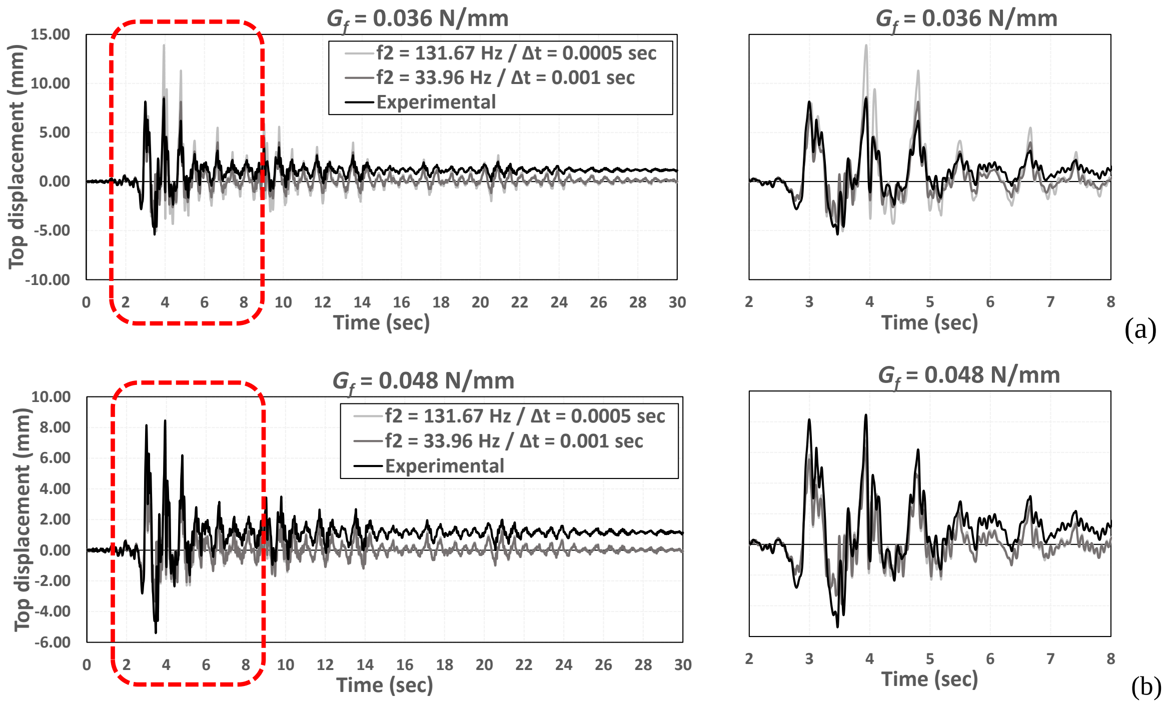



**Figure 7**: Comparison of horizontal top displacement as a function of damping parameters and time step.

Subsequently, the influence of the hysteretic behaviour of the transversal links on the numerical dynamic response was evaluated. The assessment of the influence of the unloading cycles was carried out considering a tensile fracture energy of 0.048 N/mm since it provided a more stable response. It is worth noting that the previous analyses were conducted considering tensile unloading cycles governed by a secant stiffness (oriented to the origin) with an unloading parameter $\beta = 1$. For this parametric assessment, mixed unloading stiffnesses were considered ($\beta = 0.90$ and $\beta = 0.80$). As shown in Figure 8, the numerical model did not experience significantly larger displacements; however, it was possible to evidence that the unloading cycles influenced the dynamic response of the structure in terms of residual displacement. A closer fitting with the experimental results was obtained when considering a tensile unloading parameter $\beta$ equal to 0.80.

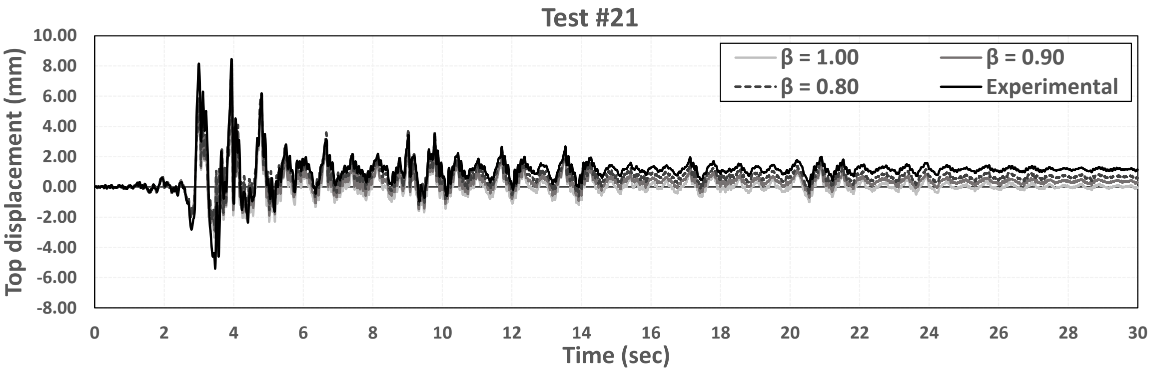



**Figure 8**: Comparison of horizontal top displacement as a function of tensile unloading cycles.

The final parameters investigated were associated with the shear-sliding response of the interfaces which influence not only the out-of-plane shear deformability and torsion behaviour of masonry but also its in-plane shear sliding mechanism. In this sense, an additional value of cohesion $c$ and friction coefficient $\mu$ was considered for this parametric assessment. These values corresponded to those obtained by means of the triplet shear tests during the experimental campaign ($f_{v0} = 0.18$ MPa and $\mu = 0.63$). The variation of the shear-sliding parameters led to a small increment in the horizontal top displacement of the numerical model. In this last parametric assessment, the three tensile unloading conditions were taken into consideration. The one that provided an accurate fitting together with the reduced values of cohesion and friction coefficient corresponded to $\beta = 0.80$ which is illustrated in Figure 9. These results showed a good agreement in terms of residual displacement evidencing the influence on the cyclic constitutive models on the nonlinear dynamic response of URM structures. It is worth to note that the numerical model experienced a slightly larger displacements when compared to the experimental results; however, they are still considered acceptable.

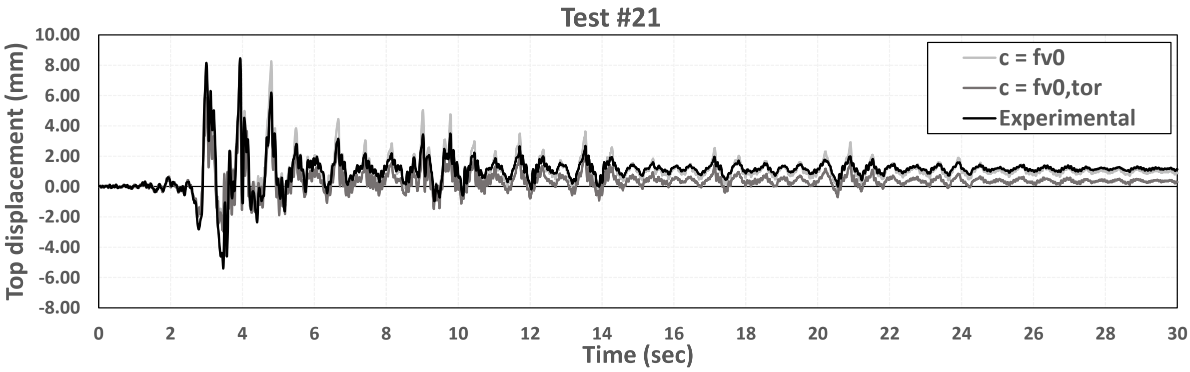



**Figure 9**: Comparison of horizontal top displacement as a function of shear-sliding parameters.

## 5 FINAL CONSIDERATIONS

This paper presents the evaluation of the out-of-plane behaviour of one URM structure using an innovative computational tool based on the Discrete Macro-Element Method (DMEM). This structure was subjected to out-of-plane two-way bending seismic loading by means of shaking table tests. The aim of this investigation focused on the simulation of the experimental response of this structure by means of numerical simulations. An initial calibration procedure was carried out in order to determine the elastic mechanical properties of the masonry material in order to reproduce the first natural frequency. In this initial calibration procedure, the dependency of the mesh discretization of the numerical model was also evaluated by the application of a signal recorded at the base of the shaking table during the experimental campaign. Due to the low intensity of this loading, it was possible to assess the response of the numerical models in the linear elastic field; and to determine an adequate model for the simulation of the dynamic response of the URM structure.

Subsequently, a parametric assessment regarding the nonlinear mechanical properties and additional model characteristic was conducted to the numerical model in order to determine the variables that present a significant influence on the out-of-plane nonlinear dynamic response of URM structures. For this purpose, two additional signals recorded during the experimental campaign were applied to the numerical model. These signals presented a higher intensity allowing the evaluation of the dynamic response in the nonlinear field. The variables that were considered for this parametric assessment were the tensile fracture energy, the tensile unloading cycles, the cohesion, friction coefficient, and the Rayleigh damping parameters. It was evidenced that the tensile fracture energy presented a significant influence on the nonlinear dynamic response of the URM structure. Due to the application of both signals, it was noted that a value of tensile fracture energy of 0.048 N/mm provided a good approximation to the experimental results. The variation of the Rayleigh damping parameters and the incremental time step also influenced on the nonlinear dynamic response of the numerical model of the URM structure, especially when considering a tensile fracture energy of 0.036 N/mm. In the case of a tensile fracture energy of 0.048 N/mm, there was a small increment of the displacements and a better representation of the dynamic response. The damping ratio remained constant during all analyses; however, it is worth noting that the influence of this parameter on the dynamic response of URM structure is also being investigated.

Moreover, it was also noted that a variation of the tensile unloading cycles the dynamic response of the URM structure in terms of residual displacements. When reducing the tensile unloading parameter from 1.0 to 0.8, the numerical model experienced larger residual displacements with a reasonable agreement with the experimental results. An alternative solution for the simulation of the residual displacements could be oriented to the definition on unloading cycles which consist on two stages: the first one associated with an initial stiffness, until a certain reduction of the maximum capacity, and a subsequently unloading with a secant stiffness. The last variable evaluated during this parametric assessment corresponded to the nonlinear properties associated with the shear-sliding response. Two values of cohesion and friction coefficient were taken from triplet shear and torsional tests conducted in the mechanical characterization. It was observed that these parameters also presented a slight influence on the nonlinear dynamic response of the URM structure by increasing the horizontal top displacement and also by contributing to the residual displacement of the numerical model. This parametric assessment stressed the need for a reliable mechanical characterization in order to properly predict the complex nonlinear dynamic response of this type of structures.

Finally, it is worth noting additional capabilities of the proposed modelling approach mainly in terms of computational burden. The analyses with an incremental time step of 0.001 sec (30,000 steps) presented a duration of approximately 40 minutes, whereas the ones with an incremental time step of 0.0005 sec (60,000 steps) lasted about 1 hour and 40 minutes using a conventional laptop. Due to the limited number of elements required for a proper simulation of the nonlinear dynamic response of URM structure; and consequently, a reduced computational demand, it was possible to conduct this parametric assessment. This can be extended for the evaluation of the nonlinear dynamic response of more complex structures for practical purposes, which is not feasible with more sophisticated computational tools.

## REFERENCES

[1] H. Derakhshan, M. C. Griffith, and J. M. Ingham, "Out-of-Plane Behavior of One-Way Spanning Unreinforced Masonry Walls," *Journal of Engineering Mechanics*, vol. 139, pp. 409-417, (2013).

[2] O. Penner and K. J. Elwood, "Out-of-Plane Dynamic Stability of Unreinforced Masonry Walls in One-Way Bending: Shake Table Testing," *Earthquake Spectra*, vol. 32, pp. 1675–1697, (2016).

[3] T. M. Ferreira, A. A. Costa, R. Vicente, and H. Varum, "A simplified four-branch model for the analytical study of the out-of-plane performance of regular stone URM walls," *Engineering Structures,* vol. 83, pp. 140-153, (2015).

[4] F. Graziotti, U. Tomassetti, A. Penna, and G. Magenes, "Out-of-plane shaking table tests on URM single leaf and cavity walls," *Engineering Structures,* vol. 125, (2016).

[5] P. X. Candeias, A. Campos Costa, N. Mendes, A. A. Costa, and P. B. Lourenço, "Experimental Assessment of the Out-of-Plane Performance of Masonry Buildings Through Shaking Table Tests," *International Journal of Architectural Heritage,* vol. 11, pp. 31-58, (2017).

[6] J. Vaculik and M. C. Griffith, "Out-of-plane shaketable testing of unreinforced masonry walls in two-way bending," *Bulletin of Earthquake Engineering,* vol. 16, pp. 2839–2876, (2017).

[7] F. Graziotti, U. Tomassetti, S. Sharma, L. Grottoli, and G. Magenes, "Experimental response of URM single leaf and cavity walls in out-of-plane two-way bending generated by seismic excitation," *Construction and Building Materials*, vol. 195, pp. 650-670, (2019).
[8] G. de Felice, S. De Santis, P. B. Lourenço, and N. Mendes, "Methods and Challenges for the Seismic Assessment of Historic Masonry Structures," *International Journal of Architectural Heritage*, vol. 11, pp. 143-160, (2017).
[9] F. Cannizzaro, B. Pantò, S. Caddemi, and I. Caliò, "A Discrete Macro-Element Method (DMEM) for the nonlinear structural assessment of masonry arches," *Engineering Structures*, vol. 168, pp. 243-256, (2018).
[10] U. Tomassetti, L. Grottoli, S. Sharma, and F. Graziotti, "Dataset from dynamic shake-table testing of five full-scale single leaf and cavity URM walls subjected to out-of-plane two-way bending," *Data in Brief*, vol. 24, (2019).
[11] I. Caliò, M. Marletta, and B. Pantò, "A new discrete element model for the evaluation of the seismic behaviour of unreinforced masonry buildings," *Engineering Structures*, vol. 40, pp. 237-338, (2012).
[12] B. Pantò, F. Cannizzaro, I. Caliò, and P. B. Lourenço, "Numerical and experimental validation of a 3D macro-model for the in-plane and out-of-plane behaviour of unreinforced masonry walls," *International Journal of Architectural Heritage*, (2017). doi: 10.1080/15583058.2017.1325539
[13] T. Takeda, M. A. Sozen, and N. N. Nielsen, "Reinforced concrete response to simulated earthquakes," *Journal of the Structural Division*, vol. 96, pp. 2557-2573, (1970).
[14] V. Turnsek and F. Cacovic, "Some experimental result on the strength of brick masonry walls," in *2nd International Brick Masonry Conference*, Stoke-on-Trent, UK (1971).
[15] C. Chácara, F. Cannizzaro, B. Pantò, I. Caliò, and P. B. Lourenço, "Assessment of the Dynamic Response of Unreinforced Masonry Structures using a Macro-Element Modelling Approach," *Earthquake Engineering & Structural Dynamics*, vol. 47, pp. 2426-46, (2018).
[16] Gruppo Sismica s.r.l., "HiStrA (Historical Structure Analysis)," (2015), Catania, Italy, Release 17.2.3, http://www.histra.it.