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NONLINEAR STATIC ANALYSIS OF AN ASYMMETRIC UNREINFORCED MASONRY BUILDING USING MACRO-ELEMENT MODELLING APPROACH

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Abstract *Past seismic events showed that irregular structures are subjected to more damage compared to the regular ones. Although seismic codes prohibit or discourage irregularities by imposing certain penalties, the design of these type of structures can be inevitable due to functional and architectural concerns. In fact, structures designed as regular configuration can also present behavior induced by torsional effects due to progressive damage and irregular load distribution after experiencing seismic events. The present paper focuses on the seismic performance of a half-scale two story unreinforced masonry building with asymmetric structural configuration. Structural irregularity both in plan and as openings in elevation was considered. Nonlinear static analyses were performed using macro-element modelling approach in two different software available for masonry structures, namely 3DMacro and TREMURI. Results obtained from the two software were compared in terms of capacity and damage patterns. It was seen a considerable difference in capacity curves. Additionally, several sensitivity analyses were carried out and sensitivity of the model to certain parameters, such as tensile strength, friction coefficient, and shear strength, was assessed.*

1. INTRODUCTION

Unreinforced masonry buildings (URM) are the oldest structural systems, and they are still one of the more sustainable construction solutions due to its fire resistance, thermal capacity, and durability. A significant portion of the building stock around the world is composed of these particular structural systems as residential or commercial buildings [1]. The design requirements of masonry buildings are mainly based on its capability on compression resistance. However, the majority of the masonry building stock is located in high seismic regions, and these structures are very vulnerable against seismic actions due to low tensile strength, ductility and lack of seismic design rules for URM buildings. In this regard, many research studies have been conducted in order to improve masonry structural systems and key mechanical properties under seismic actions. However, there is still a gap in the literature on seismic design/assessment of unreinforced masonry buildings with structural irregularities. Past earthquakes showed that irregular structural configuration influences the seismic behavior significantly. It is stressed that the derivation of seismic codes for seismic design and analysis of new and existing buildings are based on regular structures whose behavior is dominated by pure translation [2]. However, irregular structures require a more systematic strategy to consider torsional effects imposed by its own configuration under seismic actions [2], [3]. It is known that the most accurate approach is performing nonlinear dynamic analysis in order to simulate the seismic response of the structure. However, implementation of this method in engineering practice is very complex and requires very high computational cost and time. Therefore, more simplified approaches are preferred, such as pushover analysis to perform seismic design and assessment of structures. Advanced computational developments on nonlinear masonry behavior are generally focused on finite element modeling, which requires high computational effort that results in very complex and expensive methodology to adopt in practical applications. Recently, several studies show that simplified numerical approaches, i.e. macro-element modelling approach, have the capability to simulate the seismic response of the masonry structures [4]–[9]. Within this context, nonlinear static analyses of a half-scale two story asymmetric masonry building are presented through two different simplified approaches in the present paper.

2. MODELLING OF MASONRY BUILDINGS

Modern masonry buildings are governed by box-behavior which results in high-performance in-plane and negligible deformation in the out-of-plane direction. Thus, the damage is controlled by in-plane mechanisms on these particular buildings. In this regard, seismic design and assessment of masonry buildings are developed based on box-behavior and this is the fundamental principle for the analysis of modern masonry buildings and also existing masonry buildings in which box-behavior is ensured [10]. As shown in Figure 1, Tomaževič (1999) [11] classifies in-plane failure mechanisms on the masonry walls in three modes of failure. The failure mechanisms depend on several factors, such as the geometry of the wall (height/width ratio), quality of materials, boundary conditions and loads (vertical and horizontal) acting on the walls. Poor mortar quality and low vertical load results in sliding shear failure. Shear failure occurs when the principal tensile stresses higher than the tensile strength of the masonry and it is identified by diagonal cracks develop in the wall. The flexural mode of failure, which results in masonry crushing at the walls bottom corners because of the compressed regions, is observed due to high moment/shear ratio and develops mostly in masonry walls with height to width ratio.

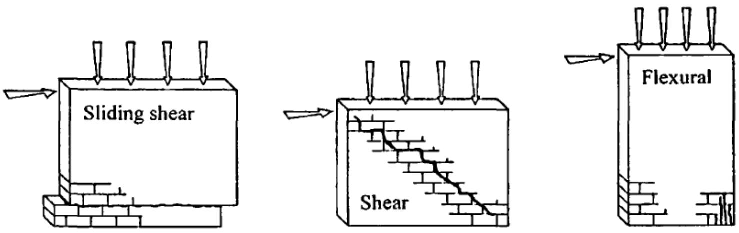
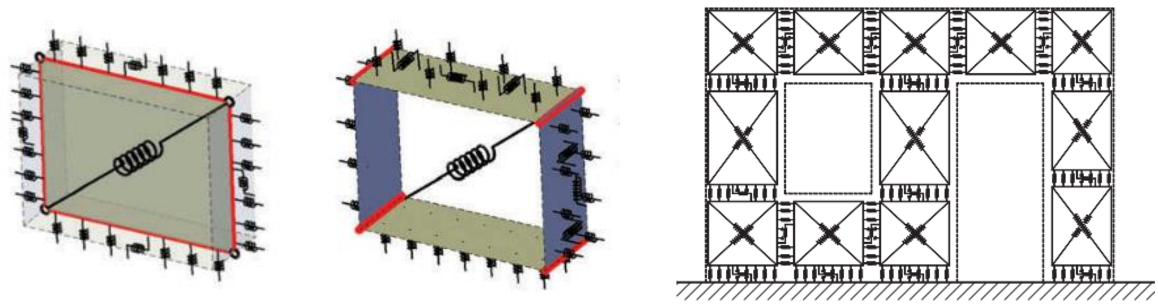


Figure 1. Typical failure modes of unreinforced masonry piers subjected to in-plane loading [11]

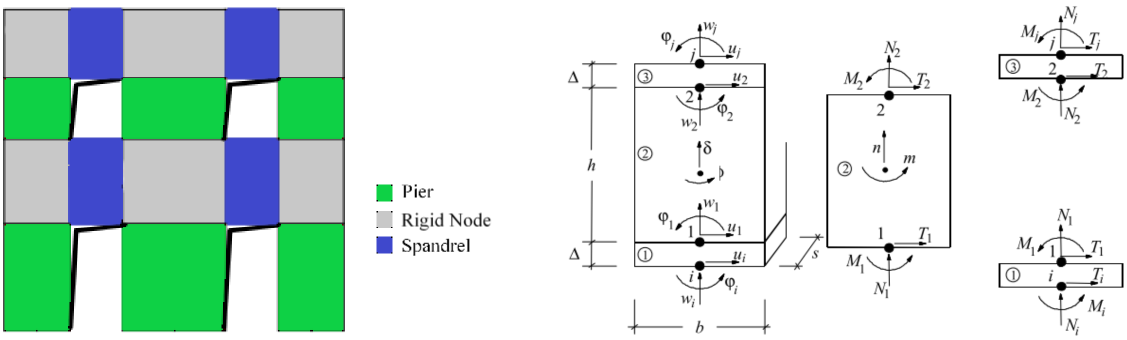
In the present paper, two different macro-element approaches were used, namely, the equivalent frame model (EFM) and macro-element discretization, to model the masonry building. In the first part, seismic analysis based on macro-element approach was performed through 3DMacro software [12]. In the early stages, a plane macro-element, which is a quadrilateral element having four rigid edges, was developed. In order to represent shear failure, nonlinear diagonal link elements are connected to the corners of the quadrilateral, and an interface composed of nonlinear springs are defined to provide interaction between other panels, elements or supports (Figure 2). These elements are only capable of simulating the in-plane response, which is the most relevant resisting mechanism in case of the expected box behavior of masonry buildings subjected to horizontal loads. The plane macro-element was upgraded to three-dimensional macro-element so as to take into account also an out-of-plane resisting mechanism, which takes a major role in case of structures without box-behavior [13]. Finally, the spatial macro-element includes 4 degrees-of-freedom for the in-plane behavior (including plane shear deformation) and 3 degrees-of-freedom for the out-of-plane behavior (Figure 2).



(a) (b) (c)

Figure 2. Macro element model (a) plane element, (b) spatial scheme [4], (c) wall discretization [12]

In the second part, the equivalent frame idealization of masonry building was made through macro-elements developed by Gambarotta and Lagomarsino (1996) [14], which is now available as both commercial and research software 3Muri and TREMURI [15]. The equivalent frame model is an idealized frame which is composed of deformable elements and rigid nodes (Figure 3). The rigid nodes correspond to the parts of the wall that are not subjected to any damage. Thus, the nonlinear response is only represented by two main deformable components which are identified as pier and spandrels. As shown in the kinematic model of the macro-element (Figure 3), in-plane failure mechanisms, namely bending-rocking, and shear-sliding, are represented by means of three sub-structures: inferior and superior layers, and a central part. The inferior and superior layers concentrate the bending and axial effects while shear deformations are concentrated on the central part of the macro-element. Some limitations of the application of the EFM approach to masonry structures with geometric complexity, particularly irregularities in elevation, due to incompatibility with the classical frame representation were addressed by Siano et al. (2018) [16]. In the case of irregular configurations, finite element modelling is suggested to overcome the possible uncertainties related to the definition of the geometry for the equivalent frame components such as piers and spandrels.



(a) (b)

Figure 3. Equivalent frame, (a) idealization, (b) kinematic model [15]

3. CASE STUDY

In this work, it is intended to apply the different numerical approaches to the seismic analysis of a concrete block masonry building previously tested in a shaking table [17]. The two-story masonry building is composed of concrete block units and reinforced concrete slabs. The building has 4.2 m x 3.4 m in the plan and 3.0 m height in total, whereas the slab and wall thickness is 0.1 m. The height of each level is 1.4 m having window and door openings with 0.8m x 0.5m and 0.5 m x 1.1 m, respectively (Figure 4 and Figure 5).

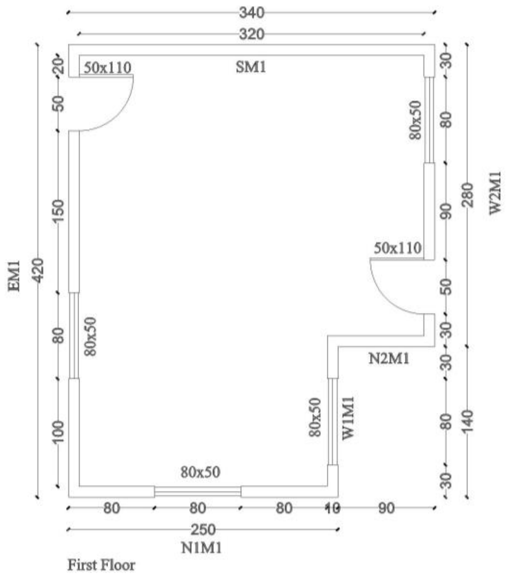
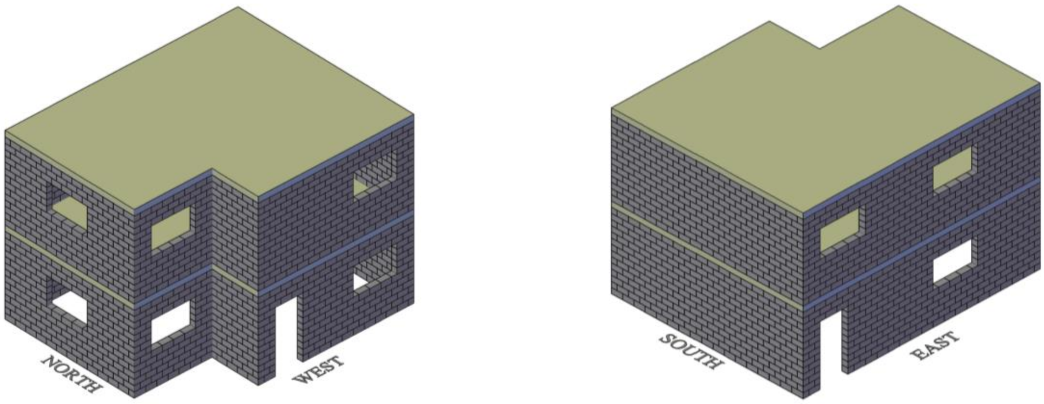


Figure 4. Plan configuration of the unreinforced masonry building (in cm)

The geometry selected for the experimental model intended to represent typical residential houses in Portugal with structural irregularity as a better representation of the building stock which, in fact, shows commonly an irregular structural layout due to architectural and structural concerns. The wall without any opening (south wall) represent a common wall sharing neighboring houses. It is important to note that the asymmetric geometrical configuration satisfies the design and construction limitations for buildings with plan irregularity imposed by Eurocodes which are: (i) bi-directional resistance and stiffness, (ii) torsional resistance and stiffness, (iii) diaphragmatic behavior at story level [18].



(a) (b)

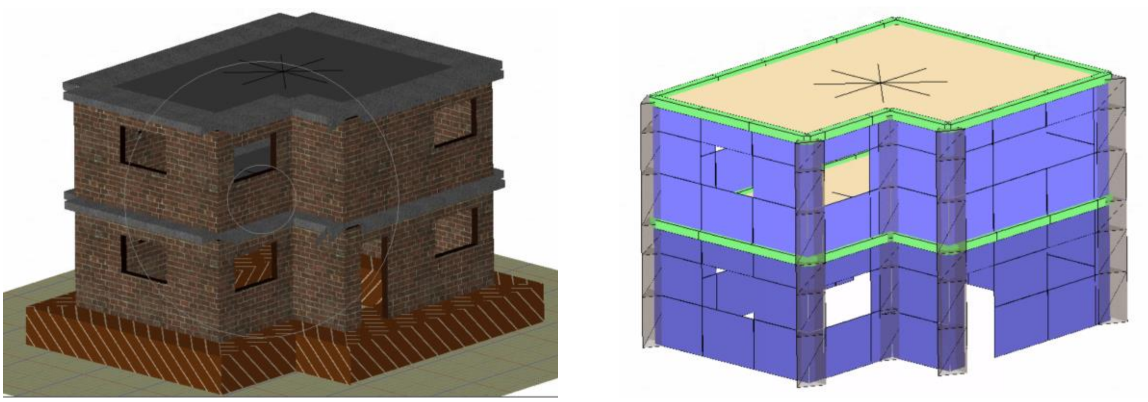
Figure 5. 3D Model of the URM building, (a) north-west façade, (b) south-east façade

4. NONLINEAR STATIC ANALYSIS

4.1. Discrete element approach – 3DMacro (sensitivity analysis)

As previously mentioned, the main objective of this work is to analyze the seismic behavior of an asymmetric masonry building under seismic loading. For this, it is important to estimate the capacity and assess the damage pattern and compare with experimental data available from the previous shaking table test. The capacity curve of the building for each direction of analysis was obtained through nonlinear static (pushover) analysis for which a loading pattern proportional to mass was considered. In a first phase, a sensitivity analysis was carried out with 3DMacro by considering a mesh composed of macro-elements with a size of 80 cm, and Mohr-Coulomb yield criterion surface for the constitutive calibration of non-linear links (Figure 6).

Table 1 and Table 2 present material properties adopted for the reference macro-element model developed in 3DMacro. These values were obtained from experimental data obtained by Velez (2014) [18] and Haach (2009) [19]. The sensitiveness analysis was carried out to have a first glance on the performance of the 3DMacro by considering the influence of different parameters in the response of the building under horizontal loads, namely: (1) quality of the connectivity between intersecting walls, (2) tensile strength of masonry; (3) shear resisting properties, namely friction angle and initial shear strength.



(a) (b)

Figure 6. 3DMacro, (a) 3D model, (b) computational model [20]

	Young's modulus (MPa)	Specific mass (kg/m^3)	Shear modulus (MPa)	Poisson ratio
Masonry walls	5300	1200	1760	-
RC slab	33000	2500	-	0.2

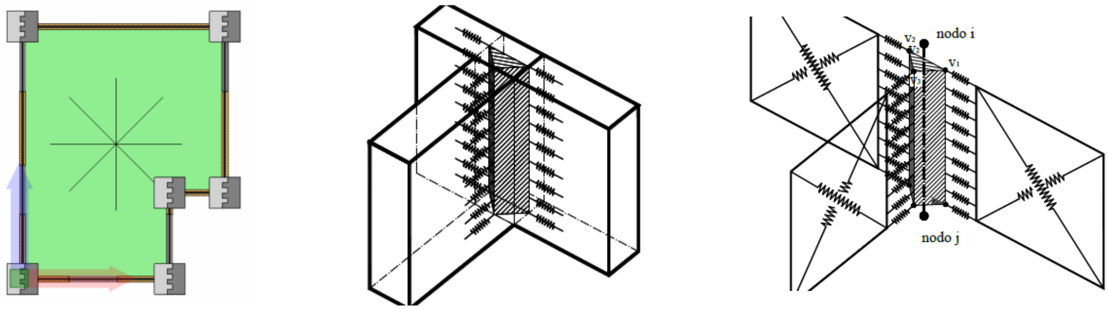
Table 1. Linear material properties for the reference model

	Compressive strength (MPa)	Tensile strength (MPa)	Shear strength (MPa)	Friction coefficient
Masonry walls	5.95	0.12	0.1	0.49

Table 2. Nonlinear material properties for the reference model

4.1.1 Sensitivity analysis results

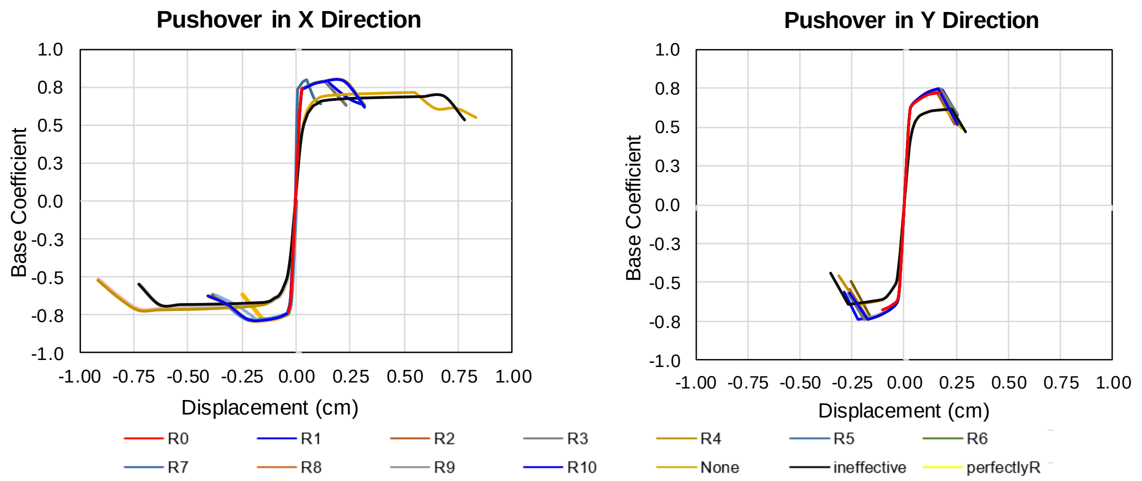
In the first attempt, connections between each structural component were investigated to improve the numerical model further. In 3DMacro, there are two ways to define the interaction between adjacent walls. The first interaction between adjacent walls is ensured through diaphragms and floors. In addition, the interaction between the wall panels is achieved by introducing corner elements via flat interfaces as shown in Figure 7, which enable to model different levels of connectivity between the walls.



(a) (b)

Figure 7. Connectivity, (a) representation of the corner elements, (b) mechanics of corner elements [21]

In fact, the corner elements are used to introduce degrees of freedom to define the interfaces designed to simulate the interaction between adjacent walls and to physically consider the volume corresponding to the interaction area between the walls. In the software, the degree of connectivity is given on a qualitative scale. Therefore, a numerical scale from 0 to 10 is identified in the model as R0 to R10 to represent the level of connectivity. In the present work, the analysis was conducted by defining the type of connectivity in accordance with the qualitative range, and by considering inefficient connection, perfect connections and absence of connection. For the personalized case, the same level of connectivity was defined for three components which are defined for traction, compression, and vertical sliding. The variation of the levels of connectivity enables to understand the influence of the connectivity in the global response of the building under horizontal loading. The results of the capacity curves obtained in the nonlinear static analysis by adopting the loading patterns proportional to mass in both positive and negative transversal (X) and longitudinal (Y) directions of the structure are presented in Figure 8. Although the quality of the connectivity was changed, the pushover curves obtained from each case overlap with each other in the linear range. It appears that the connection degree has influence only in the post-peak range. However, it is important to note that the post-peak behavior of the model with the R0 connectivity is considerably larger than the models with ineffective and without any connections. This is mainly due to the additional spring elements defined in the interfaces which introduce new degrees of freedom that changes the stiffness matrix of the model. On the other hand, the non-consideration of the corner elements (Model None) results in a similar response to the ineffective connection quality.



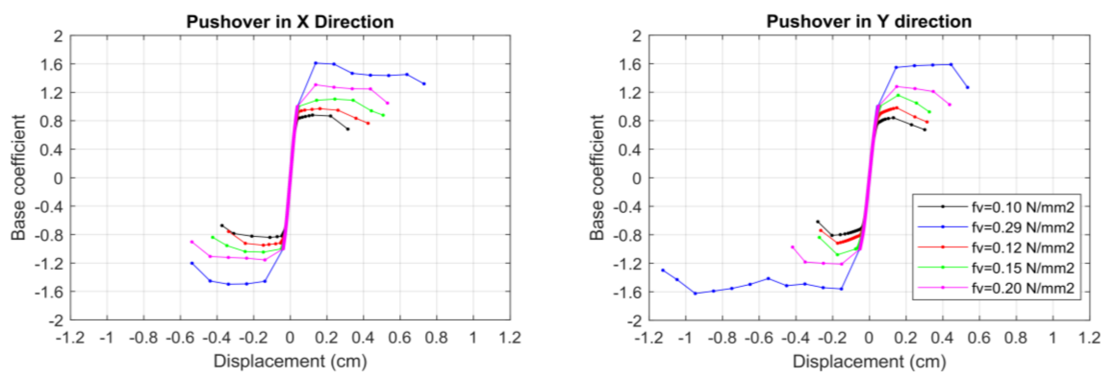
(a) (b)

Figure 8. The sensitivity of the model to the connections, Pushover curve in (a) X direction, (b) Y direction.

The shear and tensile strength properties of masonry varied in the range shown in Table 3. The influence of the variation of the strength properties in the seismic behavior of the building is analyzed based on capacity curves shown in Figure 9, Figure 10 and Figure 11.

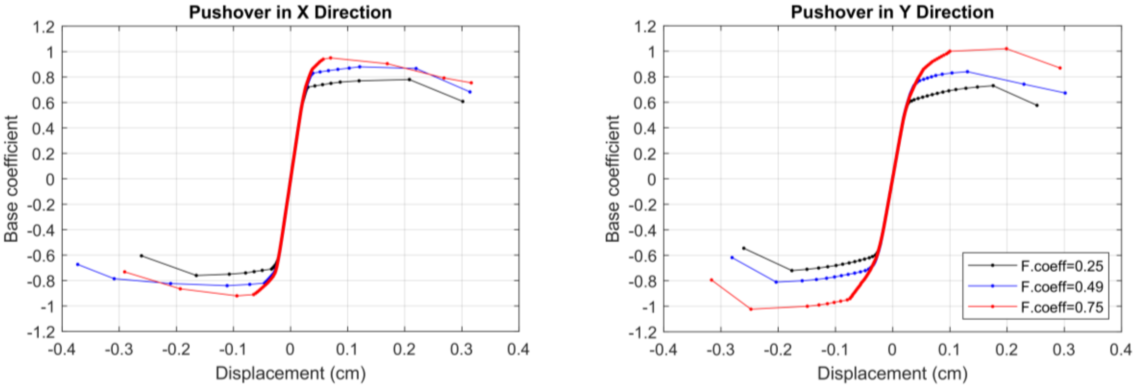
Parameter	Reference value	Input values
Shear strength (MPa)	0.1	0.1, 0.12, 0.15, 0.2, 0.29
Friction coefficient	0.49	0.25, 0.49, 0.75
Tensile strength (MPa)	0.12	0.05, 0.1, 0.12

Table 3. Parameters considered in the pushover sensitivity analysis



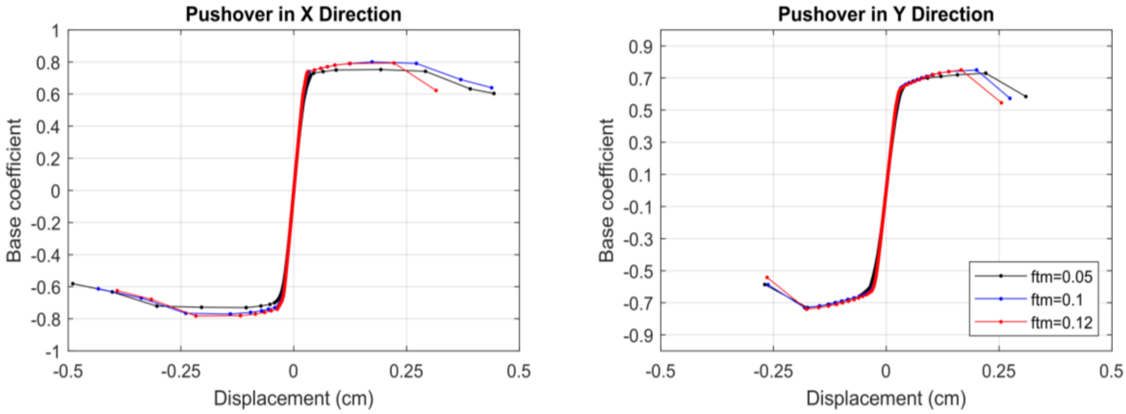
(a) (b)

Figure 9. Variation of initial shear strength, (a) transverse (X) direction, (b) longitudinal (Y) direction



(a) (b)

Figure 10. Variation of friction coefficient, (a) transverse (X) direction, (b) longitudinal (Y) direction



(a) (b)

Figure 11. Variation of tensile strength, (a) transverse (X) direction, (b) longitudinal (Y) direction

It is observed that both shear strength and friction coefficient influence the peak load capacity significantly. The capacity of the model increases as the shear strength and friction coefficient increases. On the contrary, the variation of the tensile strength only has negligible influence in the capacity of the present model.

4.1.2 Calibration of the 3D Macro-model

The final numerical model was achieved by fitting the linear range by means of calibrating the modulus of elasticity. Thus, instead of the value obtained from experimental characterization, which is 5300 MPa, it was decided to adopt an elastic modulus lower than the half value (2000 MPa) to take into account possible cracked masonry. Additionally, tensile strength and shear strength values were also defined by trial and error approach in order to match the lateral load capacity of the model with the experimental output, given that it was seen that these parameters influence considerably the response of the building, see Table 4. It should be stressed that the value of 0.33 adopted for the friction angle was calculated by using formulations recommended by Mann and Muller (1982) [22].

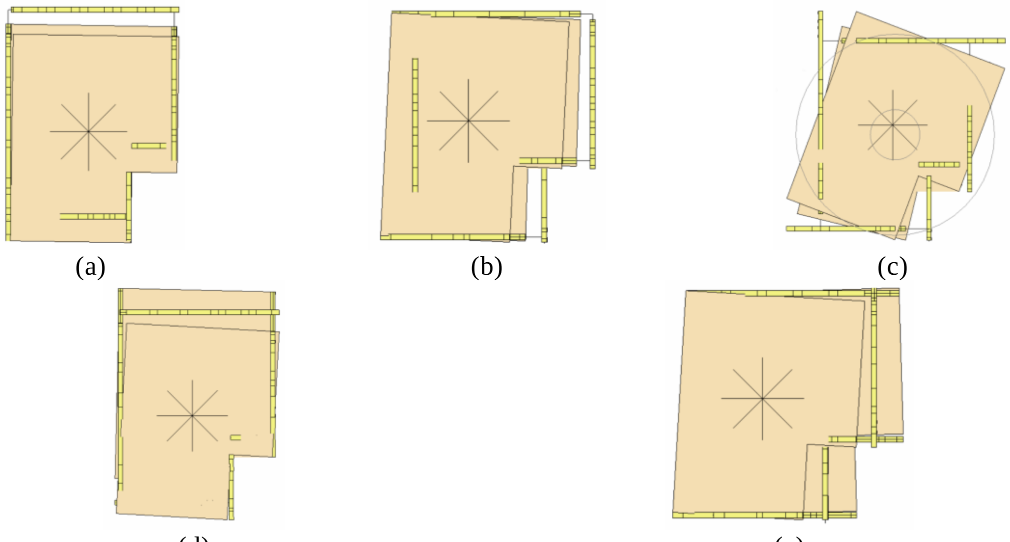
Properties	
Modulus of Elasticity, E (MPa)	2000
Compressive strength, fm (MPa)	5.95
Tensile strength, ftm (MPa)	0.12
Shear Modulus, G (MPa)	800
Shear strength, fv (MPa)	0.15
Friction coefficient	0.33
Shear drift	0.06%
Bending drift	0.08%
Yield surface	Mohr-Coulomb

Table 4. Material properties adopted for the final model

An eigenvalue analysis was conducted in order to identify the dynamic properties of the representative model. The modal properties and modes of vibration of the macro-element model are given in Table 5 and Figure 12, respectively. It is concluded that nearly 88% of the mass of the structure contributes to the dynamic response in the longitudinal (Y) direction in the first mode. Still, modes of vibration present influence of the torsional effects on the response, more particularly in the second and third modes.

Mode	T (s)	f (Hz)	Mx Sum (%)	My Sum (%)	Mz Sum (%)
1	0.060	16.5	0.86	87.99	0.00
2	0.056	17.8	84.55	89.03	0.00
3	0.022	45.8	84.68	94.11	0.01
4	0.019	51.3	90.49	94.63	0.01
5	0.018	55.5	95.70	95.30	0.01

Table 5. Modal properties of the discrete-model

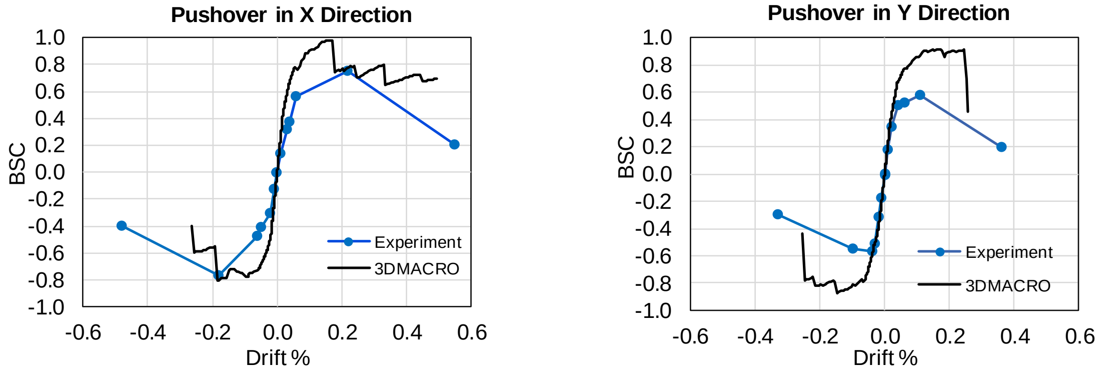


(a) (b) (c)

(d) (e)

Figure 12. Modes of vibrations, (a) 1st mode, (b) 2nd mode, (c) 3rd mode, (d) 4th mode, (e) 5th mode

The calibration of the model is finally carried out based on the comparison between the numerical pushover curve and the envelop calculated from the dynamic test carried out at the shaking table, see Figure 13.

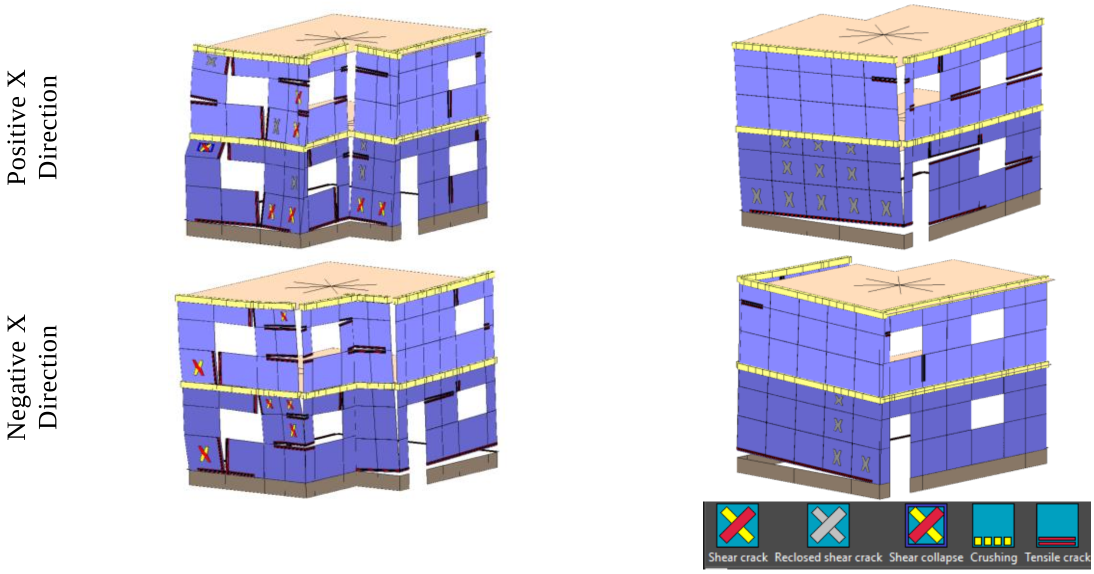


(a) (b)

Figure 13. Capacity curves, (a) transversal (X) direction, (b) longitudinal (Y) direction

The results show that 3DMacro model is in a good agreement with the experimental results in the linear range. In the transversal (X) direction, the numerical model achieves a base shear coefficient of 1.0 and 0.8 in positive and negative directions, respectively. Although the lateral load capacity is nearly 20% higher in the positive direction, the deformation of the structure at the point of peak loading is compatible. On the contrary, there is a considerable difference in the lateral load capacity between experimental and numerical results in longitudinal (Y) direction. The peak load obtained from the discrete-element model is far above than the experimental one (nearly 50%). However, it is important to note that the envelope curve represents one building prototype which was subjected to sequential dynamic testing by increasing the intensity of seismic action. Therefore, the structure was imposed on cumulative damage at each cycle. This can justify in a great extent the differences between experimental and numerical analysis.

Based on the response of the pushover analyses, a comparison of the numerical and experimental damage patterns was made. Figure 14 and Figure 16 show the damage distribution obtained in the discrete-element model at the load step when a drift value of nearly 0.2% and 0.1% in X and Y direction was achieved, respectively.



(a) (b)

Figure 14. Damage patterns at the peak load step, (a) north-west façade, (b) south-east façade

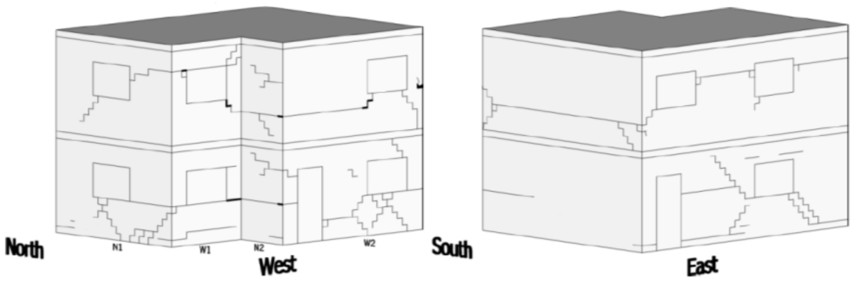


Figure 15. Crack patterns obtained from shaking table test [18]

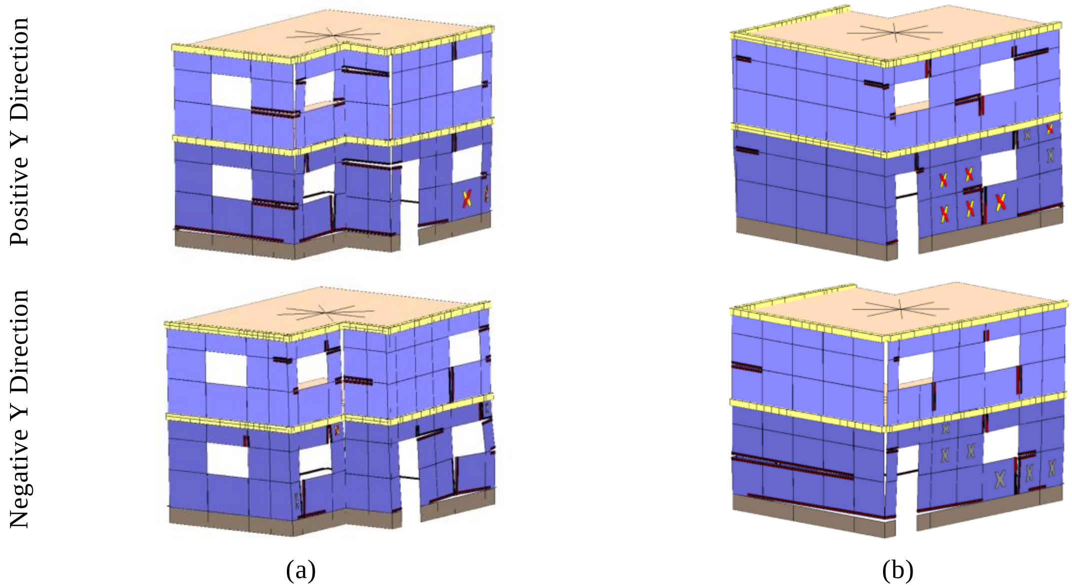
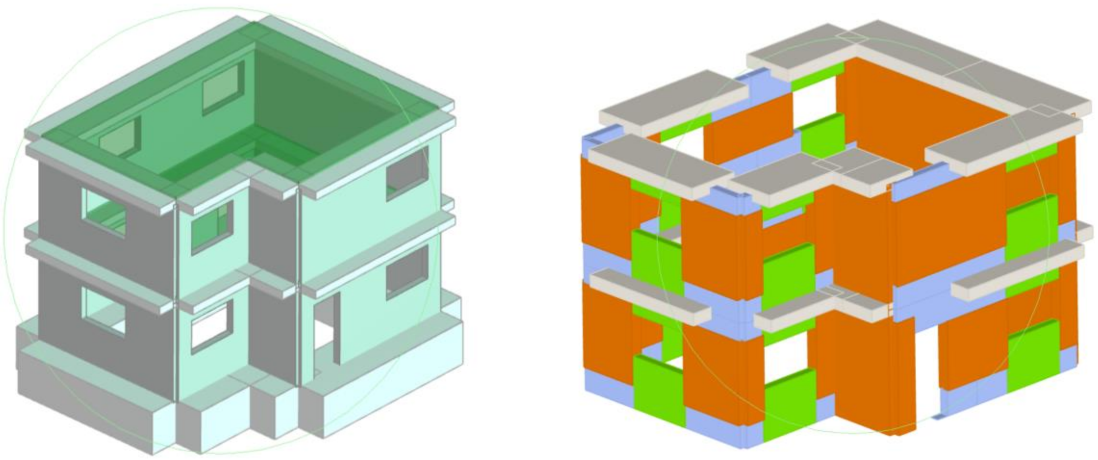


Figure 16. Damage patterns at the peak load step, (a) north-west façade, (b) south-east façade

The main reason for the consideration of these drift values is the difference between the load factors obtained from the numerical and experimental results. Additionally, drift values are directly related to the strains and deformation and, therefore, they are the better measure of the damage state and seismic vulnerabilities comparing to the force-based approach [23]. Thus, crack patterns obtained from the experimental campaign are illustrated in Figure 15, and the scheme shows the distribution of the damage at a drift value of 0.2% (X direction) and 0.1% (Y direction) which, in fact, correspond to peak load for the experimental case. It is possible to conclude that the numerical model is dominated by a rocking mechanism and diagonal shear failure. Particularly, tensile and diagonal shear cracks appear around the window openings on the north façade due to in-plane loading (Figure 14). Additionally, the rocking of the south and east walls can be seen from both experimental and numerical damage patterns (Figure 15 and Figure 16). It is important to note that shear cracks are clearly seen in Figure 16(b) on the east wall piers when it is subjected to in-plane loading (Y direction).

4.2. Equivalent Frame Approach - TREMURI

After the obtainment of the final model in 3DMacro, an equivalent frame model of the structure, shown in Figure 17, was prepared through TREMURI software by considering the same material properties of the final discrete-macro model as presented in Table 6. However, it is important to mention that TREMURI does not consider tensile strength for masonry and, therefore, zero tensile strength is assumed (Table 6). Again, the Mohr-Coulomb yield criterion was considered with full effective length. According to eigenvalue analysis, it is observed that the modal response present differences comparing to the 3DMacro, both regarding mode shapes and frequencies. The frequencies are lower than the ones obtained in the 3D Macro and the modes shapes do not correspond exactly. The highest mass participation ratio is observed in the 2nd mode (15.1Hz) and, in fact, this can be compared with the first mode obtained in 3DMacro (16.5Hz), to which the closest frequency is obtained (Table 7).



(a) (b)

Figure 17. TREMURI, (a) 3D model, (b) computational model

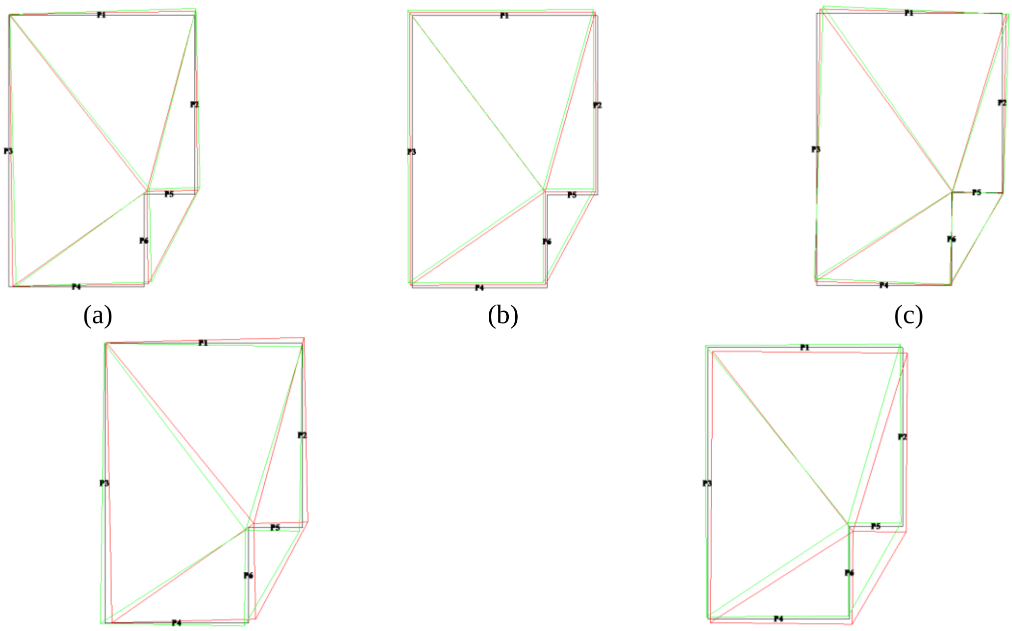
Properties	
Modulus of Elasticity (MPa)	2000
Compressive strength, fm (MPa)	5.95
Tensile strength, ftm (MPa)	-
Shear Modulus (MPa)	800
Shear strength (MPa)	0.15
Friction coefficient	0.33
Yield surface	Mohr-Coulomb, full effective length
Shear drift	0.06%
Bending drift	0.08%

Table 6. Material properties of EFM

Mode	T (s)	f (Hz)	Mx Sum (%)	My Sum (%)	Mz Sum (%)
1	0.079	12.6	35.00	17.00	0.00
2	0.066	15.1	47.00	72.00	0.00
3	0.051	19.7	67.00	81.00	0.00
4	0.03	33.0	72.00	82.00	0.00
5	0.025	39.9	75.00	87.00	2.00

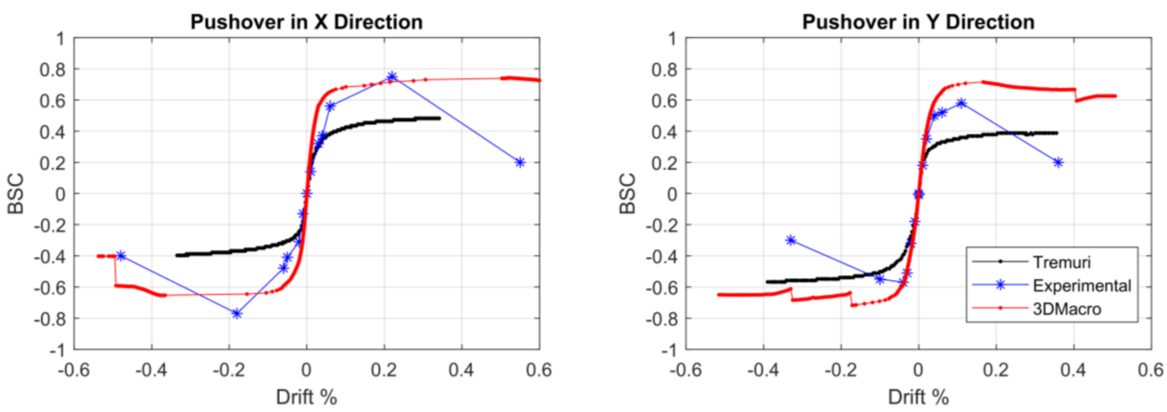
Table 7. Modal properties of the EFM

The nonlinear static analysis considering also a loading pattern proportional to the mass was performed in both positive and negative of the principal X and Y directions. The comparison of the pushover curves obtained from TREMURI, 3DMacro and experimental envelope curve is provided in Figure 19. The lateral load capacity obtained in the EFM is nearly 0.5 and 0.4 in X and Y direction, respectively. Comparing these values to the experimental base shear coefficient, TREMURI gives too conservative lateral capacity. There is a considerable difference between the experimental peak load and EFM peak load, which is nearly 33%. It is noted that a new set of nonlinear static analysis was performed in 3DMacro in which zero tensile strength was adopted for masonry material in order to have a fairer comparison between TREMURI and 3DMacro.



(a) (b) (c) (d) (e)

Figure 18. Modes of vibration in TREMURI, (a) 1st mode, (b) 2nd mode, (c) 3rd mode, (d) 4th mode, (e) 5th mode



(a) (b)

Figure 19. Capacity curves obtained from the different software, (a) transversal (X) direction, (b) longitudinal (Y) direction

The capacity curve presented in Figure 19 differs from the ones presented in Figure 13. Although the sensitivity analysis showed that the change in the tensile strength does not influence the response significantly, the reduction of the tensile strength to zero influences the capacity of the model. In this case, the load factors are nearly up to 0.75 and 0.45 in 3DMacro and TREMURI in the X direction, respectively. On the other hand, the load factors in the Y direction is about 0.75 and 0.4 in positive, and 0.75 and 0.6 in the negative direction. From Figure 19, it is observed that 3DMacro gives higher lateral load capacity comparing to TREMURI, being on average 50% higher (X and Y directions). This result appears to be in agreement with the result obtained by Marques and Lourenço (2011) [5], which pointed out a difference in the load capacity obtained in two software of about 20%. Figure 20 and Figure 21 present damage patterns at a drift value of 0.2% in the transversal direction in TREMURI and 3DMacro, respectively.

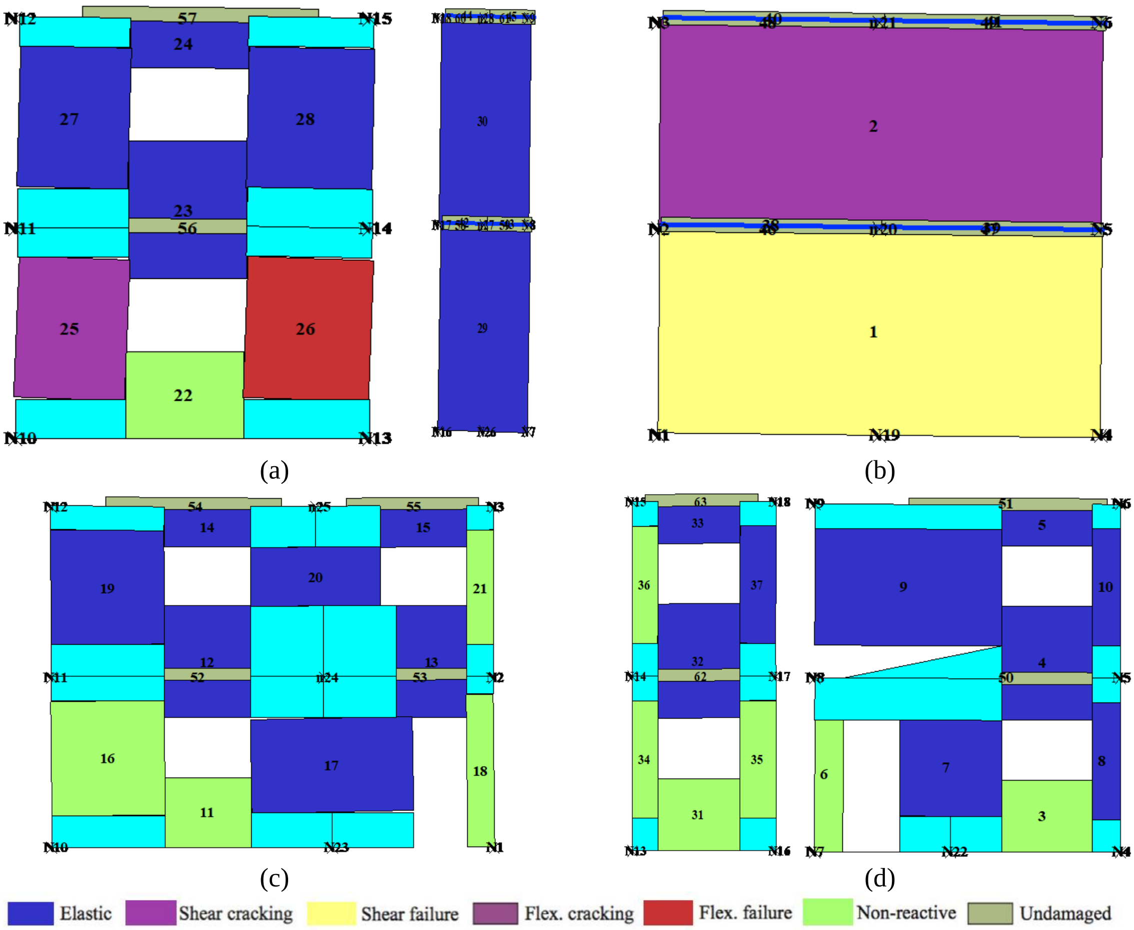


Figure 20. Drift 0.2% in +X direction (TREMURI), (a) north, (b) south, (c) east, (d) west façade

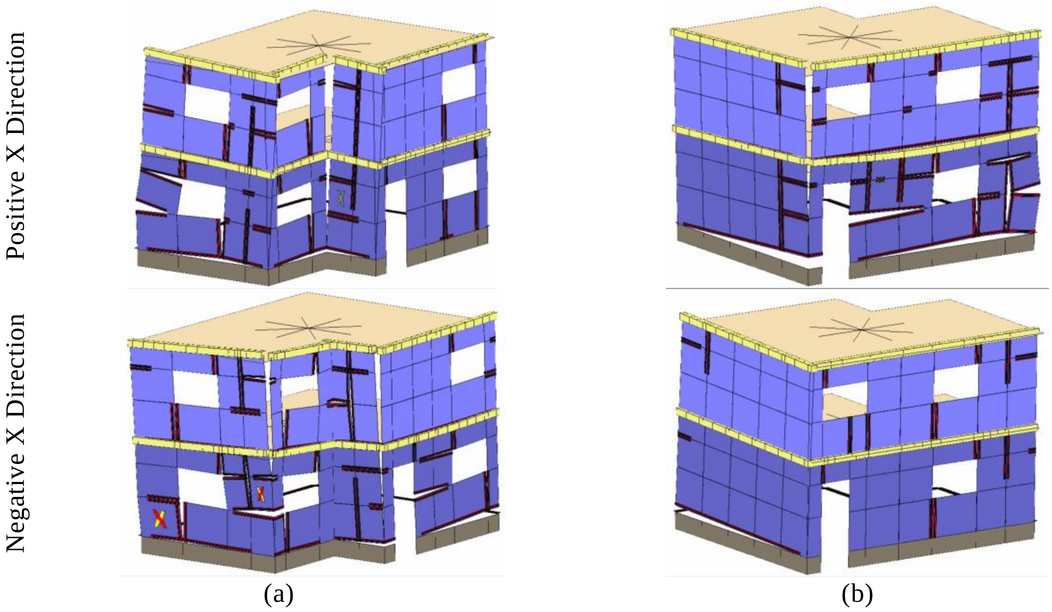


Figure 21. Drift 0.2% in X direction (3DMacro), (a) north-west façade, (b) south-east façade

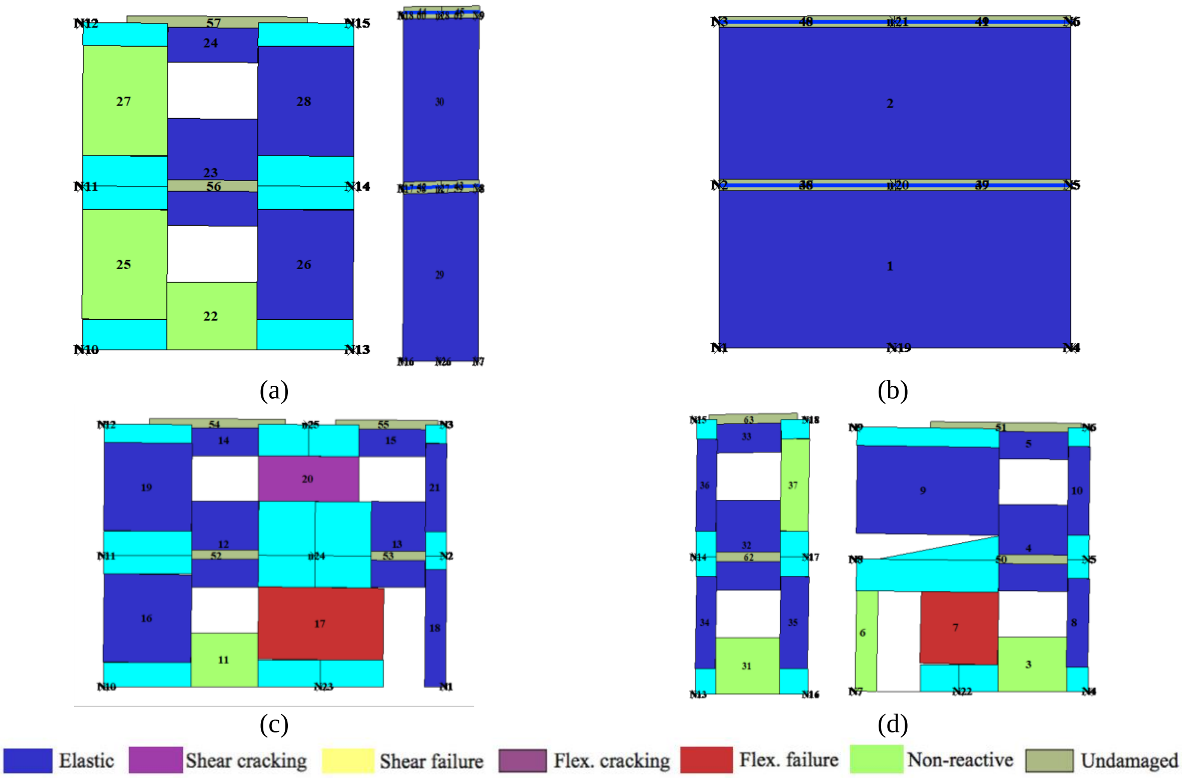
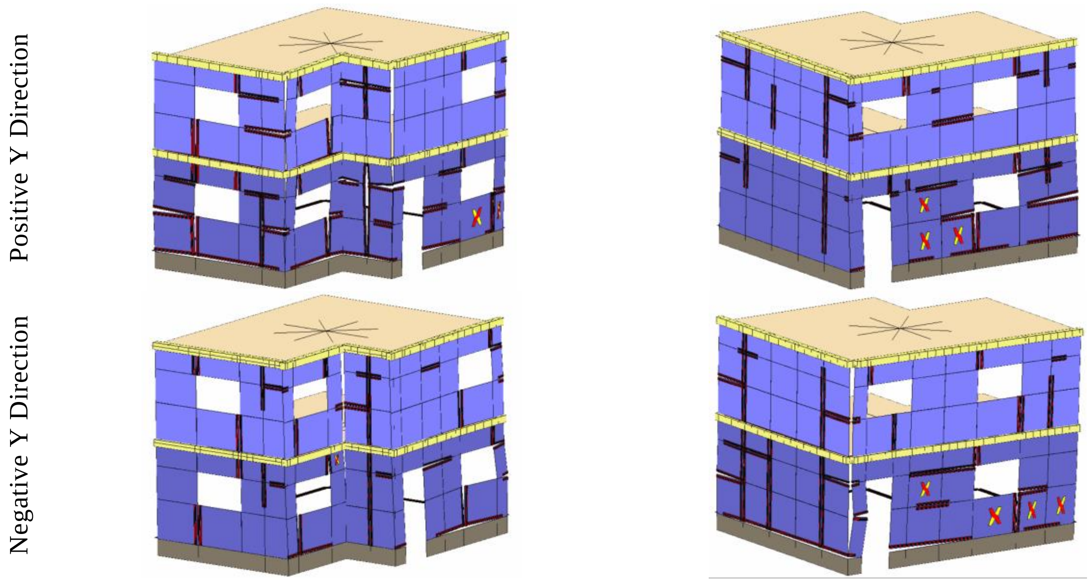


Figure 22. Damage patterns at drift 0.1% in +Y direction (TREMURI), (a) north, (b) south, (c) east, (d) west façade



(a) (b)

Figure 23. Damage patterns at drift 0.1% in Y direction (3DMacro), (a) north-west façade, (b) south-east façade

North and south walls, when subjected to in-plane loading, show mixed flexural and shear failure mechanism in TREMURI (Figure 20(a) and (b)). In parallel, north wall behaves mostly under flexural mechanism in the discrete-element model, and tensile cracks dominate the cracking patterns. Additionally, the rocking mechanism is observed on the south wall, which can result in tensile cracks on west façade even though the plane of the element is in the out-of-plane direction due to torsional effects (Figure 21). East and west façade exhibits flexural mechanism when subjected to in-plane loading (y-direction) on the first floor (Figure 22). The application of the seismic load in the longitudinal direction imposes horizontal and vertical tensile cracks and also shear cracks in 3DMacro (Figure 23).

5. CONCLUSION

Unreinforced masonry buildings have been investigated by many researchers aiming at understanding and improving their seismic performance. Recently, simplified approaches have been developed to perform seismic design and assessment of masonry buildings to promote construction of new low- to mid-rise URM buildings in seismically active regions. The present paper was focused on the application of simplified numerical approaches to performing nonlinear static analysis on unreinforced masonry buildings with geometry complexity, irregularity in plan and different opening distributions in elevation. In this regard, 3DMacro and TREMURI were used to perform pushover analyses.

Regarding the sensitivity analysis in 3DMacro, the definition of shear strength and friction coefficient plays an important role in the lateral load capacity. The calibrated discrete-element model presents higher lateral load capacity and reproduces similar in-plane damage patterns to the experimental ones at the same drift value. Furthermore, it was found that the masonry building modelled through discrete-element presented considerably higher load capacity and deformation ability when compared to the case where the building was analyzed with the equivalent frame model.

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