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SEISMIC VULNERABILITY ASSESSMENT OF EXISTING MASONRY BUILDINGS: A COMPARISON BETWEEN TWO DIFFERENT MODELING APPROACHES

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Abstract

Seismic vulnerability assessment of existing masonry buildings requires to implement numerical models that are both reliable and computationally efficient. To date various modeling approaches have been proposed in literature in order to evaluate the global response of such structures within a Finite Element Model (FEM) approach, such as the Equivalent Frame Model (EFM) and the Continuum Model (CM). The first is based on a simplified scheme comprising a system of mono-dimensional non-linear frame elements (such as piers and spandrels), by obtaining a significant structure simplification with an important reduction of the computational costs. Whereas, the second approach involves to model the structure through two- or three-dimensional continuum finite elements (as shell or solid). In this way a more refined model is obtained, even if the computational burden is drastically incremented and a large amount of data is required, too.
The paper presents a comparison between the two modeling approaches described above, used to implement and to analyze a case study, an existing masonry building located in Italy being irregular both in plan, since it is L-shaped, and in the elevation. Non-linear pushover static analyses are performed, and comparisons among the obtained results by means of both the models considered are shown and commented.

Keywords: Seismic Assessment; Equivalent Frame Models; Finite Element Methods; Non-linear Static Analysis

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1 INTRODUCTION

The Italian territory is characterized by a high presence of masonry buildings, having ordinary or strategic function [1-2], and also very often historical value such as churches, castles and towers [3-4].

Due to their intrinsic vulnerability to seismic actions, masonry structures are now a major research field in civil engineering and safeguarding the built heritage. In order to evaluate the seismic vulnerability of older masonry structures, to implement numerical models, which balance accuracy and speed of calculation.

The choice of the most appropriate modelling approach plays a crucial role to simulate their seismic response. Mainly, it depends on the failure mode to investigate taking into account the computational costs [5].

This study focuses on the application of two different FEM (Finite Element Method) modeling approaches to evaluate the global response of existing masonry buildings, that are: Equivalent Frame Model (EFM) [6], and the Continuum Model (CM) [7].

The first, identifies the piers (e.g., vertical panels) and spandrels (e.g., masonry beams that connect piers) e rigid nodes [8]. The walls are interconnected by horizontal floor diaphragms (floors).

Piers and spandrels are modeled using two-dimensional finite macro-elements, representative of masonry panels, with two nodes and three degrees of freedom per node (u_x, u_z, r_y). The remaining portions of the wall are then considered as two-dimensional rigid nodes of finite dimensions, to which the macro-elements are connected. These macro-elements transmit the actions along the three in-plane degrees of freedom to each of the connected nodes [9-10].

The subdivision, into nodes and elements, permits considering the wall models as a plane frame.

Whereas, the CM consider the model of masonry as fictitiously a homogeneous material, and the structure is represented through a continuous mesh of two- or three-dimensional Finite Elements [11]. For this type of modeling, the relevance of the analysis increases with the greater the level of detail of the mesh used.

As for the implementation and analysis with the EFM, the '3 Muri' software is used [12]. It is specially devoted to seismic analysis of masonry structures, permitting the easy integration of different type of masonry walls, as well as offering alternative options for pushover analysis. In addition, it allows the explicit representation of flexible horizontal diaphragms, which are particularly common in historic buildings [9-10].

While, as for the CM in this study numerical simulations are performed with Midas Fea NX [13]. In this case a smeared crack model for material is assumed, simulating cracks by regulating internal stress and stiffness calculations, effectively capturing material damage and crack behavior without modeling cracks or changing the basic shape of the model [14].

The study represents the preliminary results of an ongoing research aimed at comparing the results obtained if different numerical approaches are considered.

The work focuses on the study and application of Finite Element and Equivalent Frame Models for performing non-linear seismic analysis on masonry structures.

2. CONSTITUTIVE LAWS

Constitutive laws characterize the material behavior under acting loadings. They define the development of stresses and strains, influencing the distribution of internal forces, damage progression, and possible collapse mechanisms. In the following a description of the constitutive laws utilized for the two numerical models considered in this study is reported.

2.1 The EFM material

The EFM non-linear behavior is determined by the individual non-linear behavior of its masonry elements, specifically piers, and how these elements behave with respect to bending combined with axial load and shear stresses.

Figure 1 shows the nonlinear force–displacement relationships assigned to the piers, expressed in terms of the ratio between internal force and maximum strength. As for the shear constitute law (Figure 1a), it is characterized by an elastic phase followed by a plastic one. Then a drop in strength is assigned until the failure when the maximum displacement capacity is reached.

Similarly, also for the bending, the constitute law includes an elastic-perfectly plastic relationship with a drop until the failure (Figure 1b).

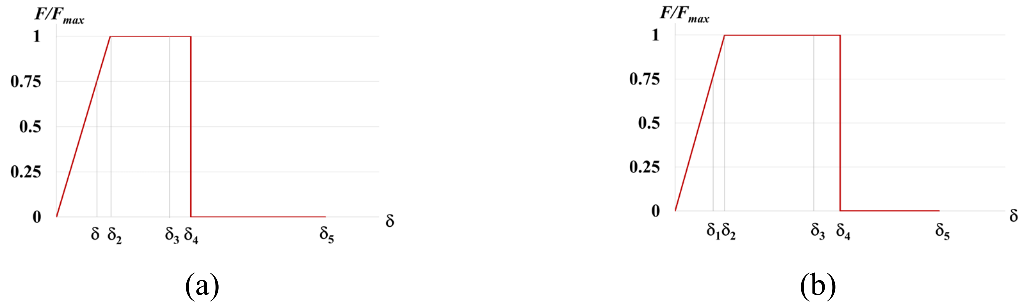


Figure 1: Constitute laws of piers, (a) shear, (b) bending

2.2 The CM material

For the CM the Concrete Smeared Crack Model (CSCM) is used, whose constitute laws are shown in Figure 2.

In particular, Figure 2a reports the constitute law in compression, where f_c is the compression strength, E is the Young Modulus, G_C is the compression fracture energy, and h is the mesh size.

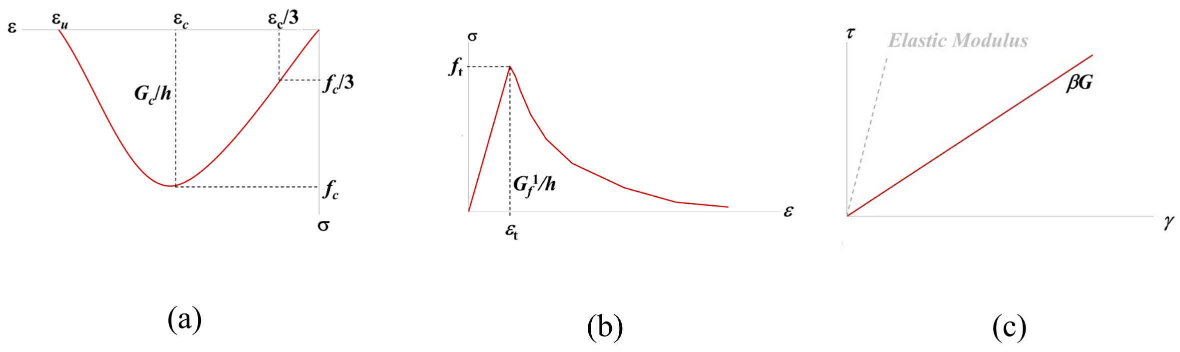


Figure 2: Function considered for the smeared crack model for (a) compression, (b) tension and (c) shear.

The fracture energy indicates the amount of energy required to generate a cracked unit surface area [15-16]. It is calculated according to the following equation, suggested in [18].

$$G_c = 15 + 0{,}43 \cdot 0{,}43 \cdot f_m - 0{,}0036 \cdot f_m^2 \tag{1}$$

On the contrary, the tension function has a linear branch up to the tensile strength, followed by an exponential softening. Figure 2b reports the constitute law assumed, where f_t is the masonry tensile strength, G_t is the tensile fracture energy. The latter can be obtained with the relation in the following [17-18]:

$$G_t = d_{u,t} \cdot f_t \tag{2}$$

in which $d_{u,t}$ is the tensile ductility factor, and f_t is the masonry tensile strength, expressed as 1, 2, 5 or 10% of the compressive strength; In this study, 10% is used. As for $d_{u,t}$, it is suggested to assume values ranged within the interval 0,018 mm $\leq d_{u,t} \leq$ 0,040 mm [17-18].

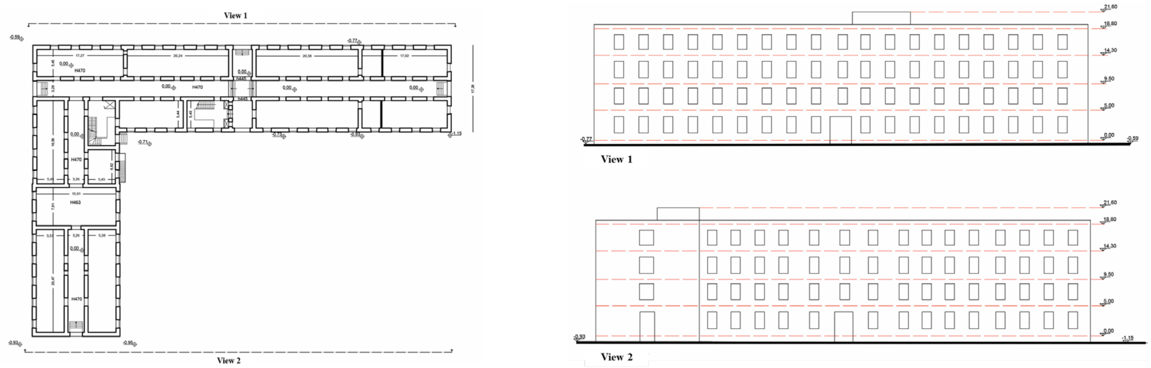
Finally, a linear relationship for shear is assigned (Figure 2c). After cracking, a (hear stiffness reduction coefficient β equal to 0.05 [14] is assumed.

3. APPLICATION ON A CASE STUDY

The two numerical approaches considered in this study are applied to a case study, that is a masonry building located in Italy, within a seismic zone III [19].

The structure is L-shaped as visible in Figure 3a, with dimensions of approximately 17.00 m x 83.19 m x 57.75 m distributed over 3 levels for the entire surface area, and there is a fourth level that has reduced dimensions on the short wing 17.00 m x 83.19 m x 36.56 m. In addition, there are two turrets at the stairwell, one on the short and one on the long side.

The building has a total height of approximately 19 m above the street level (excluding the turrets). The maximum intermediate floor height is approximately 4.43 m and rather regular openings as the main elevations (Figure 3b), while the net height of the turret is approximately 2.80 m. The perimeter walls are 80 cm thick on the first level, 70 cm on the second level and 60 on the third and fourth levels, while the internal walls are 75 cm thick on the first level, 65 cm on the second level and 60 on the third and fourth levels.



(a) (b)

Figure 3: Case study considered, (a) representative floor; (b) front views.

The masonry forming the external perimeter and the internal divisions of the turrets is 50 cm thick.

With regard to the floors, the building is composed of hollow bricks and concrete slabs. All the bays have a horizontal floor with a structural thickness of 25.5 cm, with the exception of the bays with a span of 8 m have a horizontal floor with a structural thickness of 28.5 cm.

The loads applied for the two types of floors are summarized below (Table 1).

Type	Height (cm)	G_{k1} (kg/m^2)
A	25.5	220
B	28.5	280

Table 1: Structural permanent loads for the two-floor types.

The non-structural permanent load consists of pavement, screed, plaster, and internal partition walls. Specifically, the non-structural permanent loads for the type A and B floors in the case under consideration are:

	Type A	Type B
Pavement	1.00 kN/m^2	1.70 kN/m^2
Partition walls	1.00 kN/m^2	1.00 kN/m^2
Cover floor	2.50 kN/m^2	2.50 kN/m^2

Table 2: Non-structural permanent loads for the two-floor types.

The live load assumed is reported in the Table 3, according to the Italian standards [19]. It is assumed that the building has an office destination.

Category	Level	Q_K (kN/m^2)
B1	1, 2, 3,4	2.00

Table 3: Live load assumed

3.1 Materials properties

The entire structure is composed of irregular soft stone masonry with the presence of two rows of regular brick recourses of dimensions 27x13x5.5 cm, arranged with mortar of approximately 2 cm at an interval of approximately 1 m and transversal connections arranged at an interval of approximately 31 cm.

Due to the constant and regular presence of recourses in the irregular soft stone masonry, it is assumed for implementing the numerical models, that the masonry may be considered regular, by assigning the mechanical properties described in Table 4 [20].

The Confidence Factor (CF) adopted is equal to 1.2 by assuming a for a Knowledge Level (KL) 2.

	Regular soft stone masonry
E [N/mm^2]	1410
G [N/mm^2]	450
w [kN/m^3]	16
f [N/mm^2]	2.60
f_{v0} [N/mm^2]	0.145

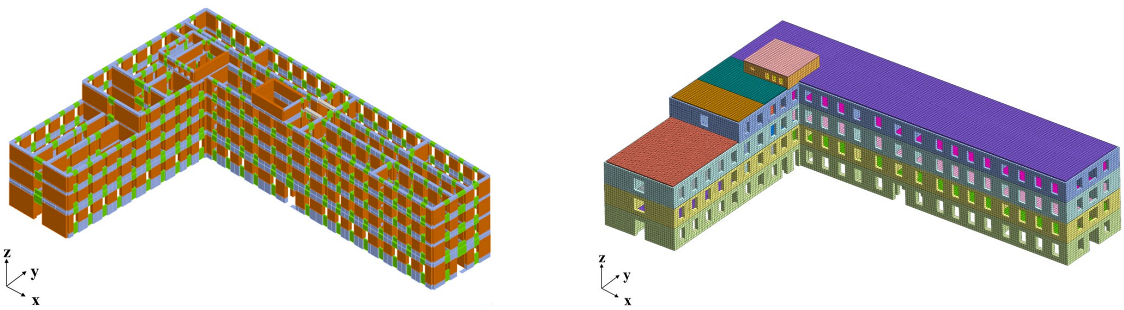
Table 4: Masonry mechanical properties.

3.2 Numerical models

Figure 4a shows the EFM implemented for the case study considered, where the piers are in brown, the spandrels in green and the rigid beams in blue. The floors are modelled with slab elements connected to the three-dimensional nodes and loaded with structural, non-structural, and live loads. The non-structural loads also take into account the internal infills. The model implemented is fixed at the base.

In the CM masonry elements are modelled by using solid elements for both the walls and the horizontal floors (Figure 4b). Breaking-weld contact is applied between the floors meshes to guarantee a semi-rigid constraint. The buildings are fully restrained at base, too. Non-linear behavior is assigned to all masonry elements, while horizontal floors, modelled as slabs with thickness 5 cm, are assumed elastic.

As previously introduced, to compare the two modeling approaches, nonlinear analyses are carried out, considering, where possible, the same hypotheses, loading, boundary conditions, and seismic forces.



(a) (b)

Figure 4: FEM models implemented (a) Equivalent Frame and (b) Continuum Model.

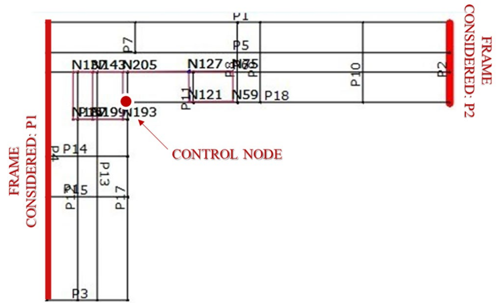
4. RESULTS AND COMPARISONS

In this section, the preliminary results obtained by following the two modelling approaches considered are compared and commented.

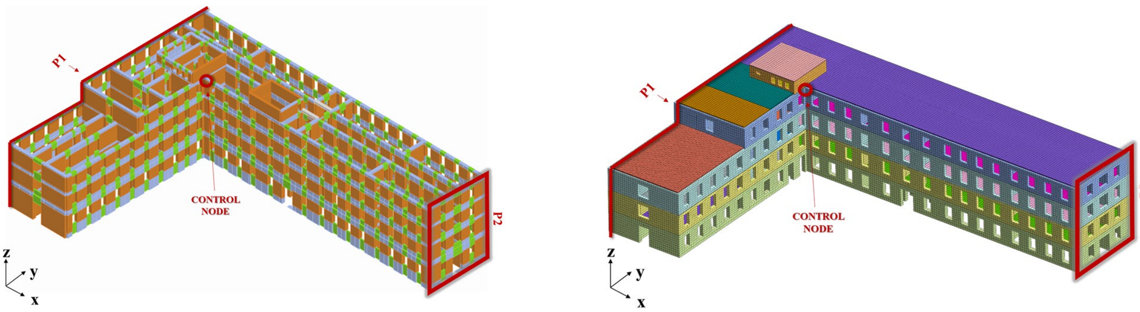
Pushover analysis along the +Y and -Y directions with uniform seismic loading without eccentricity are conducted for the numerical models implemented. The analyses are conducted in displacement control, by assuming the corner node as 'control node', as highlighted in Figure 5a.

The results obtained are compared in terms of the pushover curve and also in terms of damage state in several steps of the pushover analysis.

To this scope, two masonry frames have been chosen as an illustrative example. The frames under consideration, highlighted in Figure 5a have been selected since they are the external frame along the X direction, having its simple and regular opening configuration, and uniform height. Moreover, the damage state considered is related to the analysis step highlighted in yellow on the push-over curve. For the damage state comparisons, push-over curves refer to the same horizontal displacement imposed.



(a)

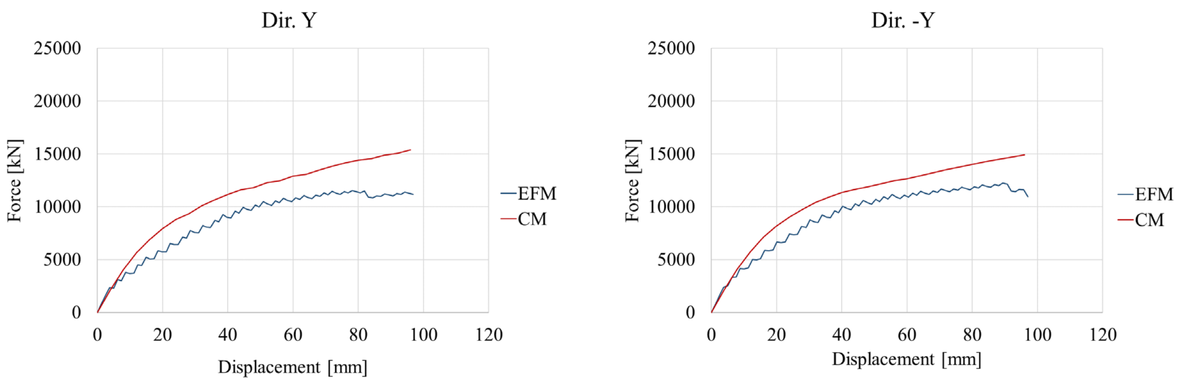


(b) (c)

Figure 5: Frame considered for the comparison localization

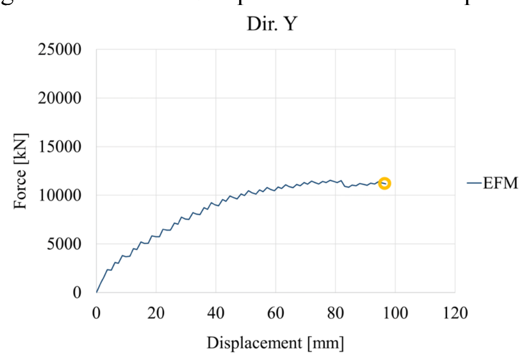
By analyzing the preliminary pushover curves for the +Y, and -Y directions (Figure 6), it is clear to one that the ones along +Y (Figure 6c) and -Y directions (Figure 6d) have a scatter between them, probably due to differing floor modeling criteria. In the Continuous Model (CM), each horizontal floor is represented as a slab, representing an element collaborating with the spandrels under the seismic action, reducing in the latter the acting internal forces. This interaction, consequently, leads to a stiffer numerical model where the spandrels failure is delayed.

Furthermore, the comparisons between the preliminary non-linear analyses show that for the pushover analyses in the +Y direction (Figure 7 and Figure 8) and -Y direction (Figure 9 and Figure 10), the results are similar in terms of both the pushover curve (Figure 7a and Figure 8a for Y and Figure 9a and Figure 10a for -Y) and the response mechanism. The non-linear response is due to failures of both the spandrels and the piers for the two walls analyzed (P1, Figure 7b - Figure 8b - Figure 9b - Figure 10b and P2, Figure 7c - Figure 8c - Figure 9c - Figure 10c).

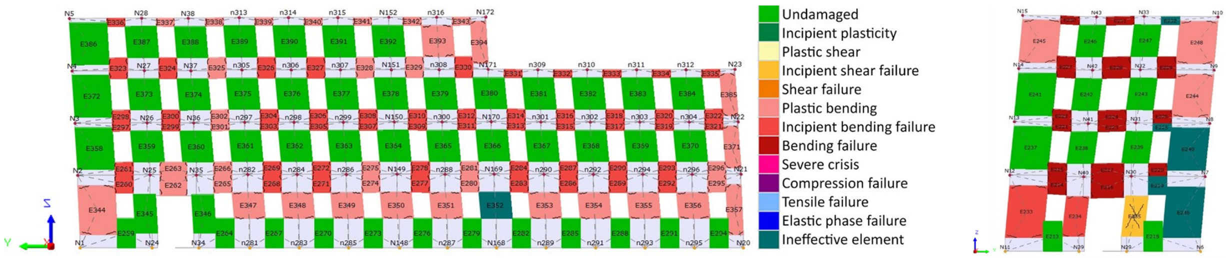


(a) (b)

Figure 6: EFM and CM pushover curves comparisons

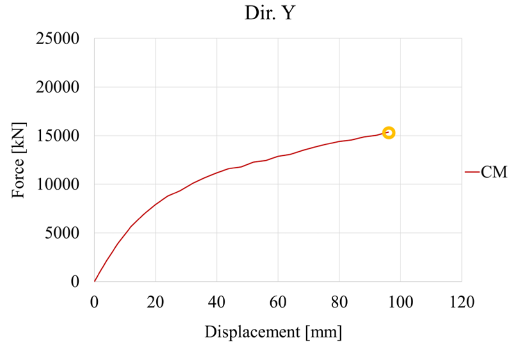


(a)

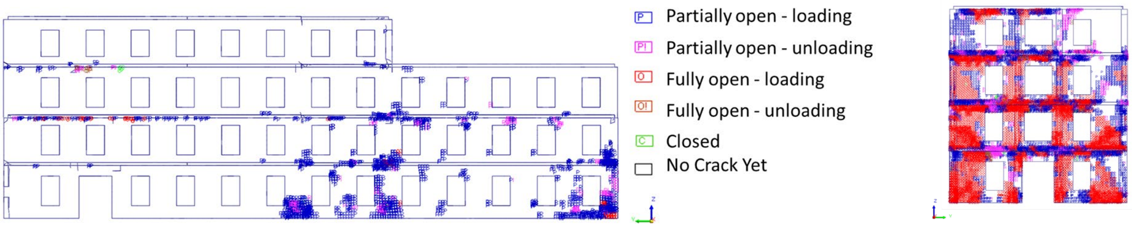


P1 (b) P2 (c)

Figure 7: EFM (a) pushover curve and damage mechanism for + Y direction for (b) P1 and (c) P2

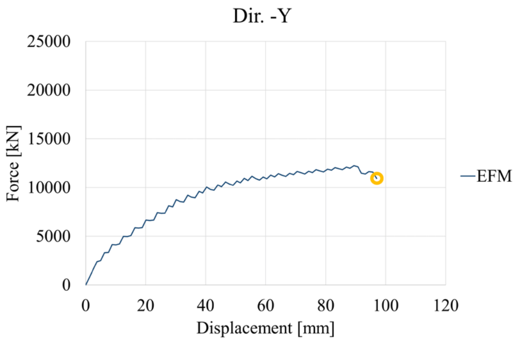


(a)

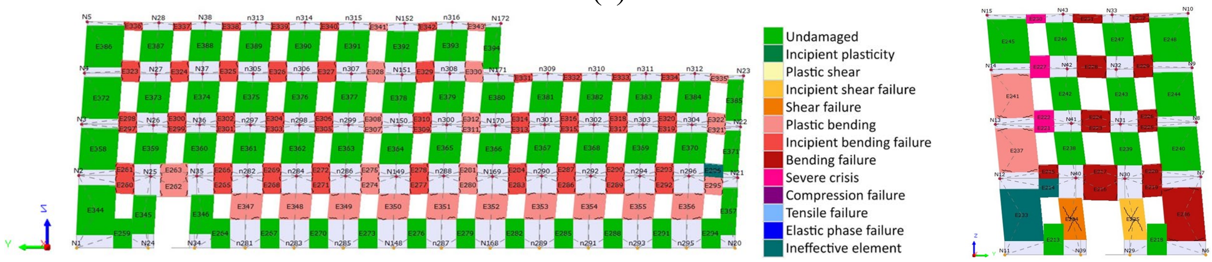


P1 (b) P2 (c)

Figure 8: CM (a) pushover curve and damage mechanism for + Y direction for (b) P1 and (c) P2

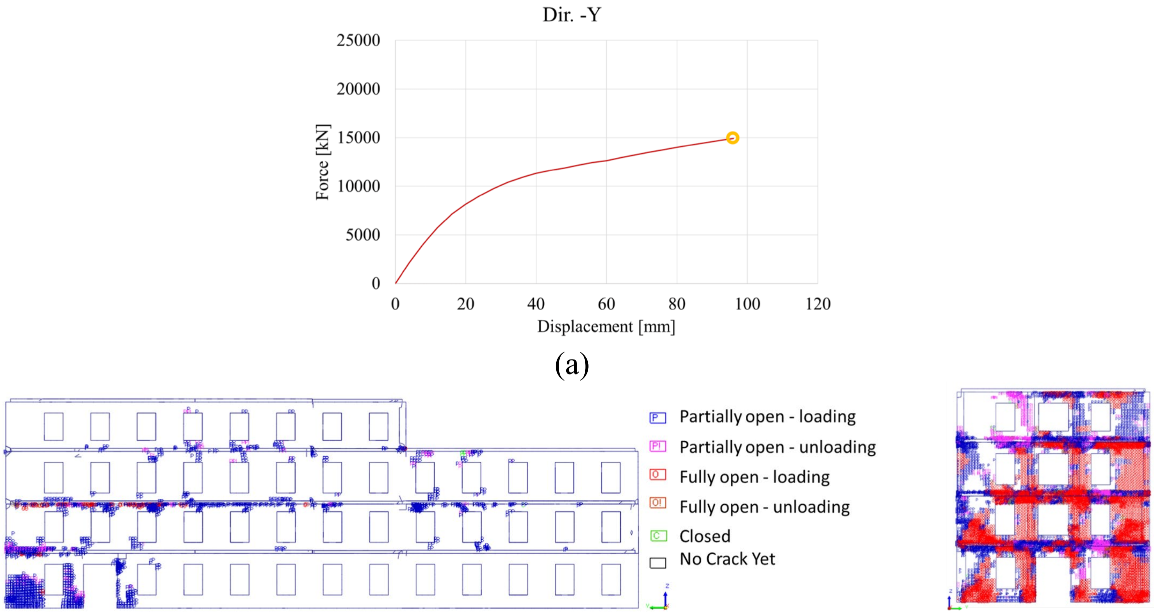


(a)



P1 (b) P2 (c)

Figure 9: EFM (a) pushover curve and damage mechanism for - Y direction for (b) P1 and (c) P2



P1 (c) P2 (d)

Figure 10: CM (a) pushover curve and damage mechanism for - Y direction for (b) P1 and (c) P2

5. CONCLUSION

This paper shows a comparative analysis between Equivalent Frame Model (EFM) and Continuous Model (CM) for evaluating the seismic response of masonry structures.

At first, a description of the two-model approaches with a focus on the two different non-linear constitutive laws is carried out. In particular, non-linear and smeared crack constitutive models are used for EFM and CM, respectively.

In order to understand the differences in terms of structural response, a 4-storey L-shaped masonry case study is considered and pushover analyses are performed with the same assumption.

The preliminary results obtained show that a scatter among the results may be obtained if different numerical models are used, highlighting the importance of the modelling criteria to realistically simulate the seismic response of existing masonry buildings.

Starting from these preliminary results, further numerical investigations will be performed in order to evaluate the sensitivity of the numerical models with respect to the input parameters, identifying the primary ones in the assessment of the seismic performance.

6. REFERENCES

[1] A. Palmiotta, R. Sulla, M. D’Amato, R. Gigliotti, R. Seismic Risk Assessment Of Masonry Buildings. Part I: Application To A Case Study According To Italian Standards. In 22ND INTERNATIONAL CONFERENCE OF NUMERICAL ANALYSIS AND APPLIED MATHEMATICS, 2024.

[2] R. Sulla, A. Palmiotta, M. D’Amato, R. Gigliotti, R. Seismic Risk Assessment Of Masonry Buildings. Part II: Parametric Investigations. In 22ND INTERNATIONAL CONFERENCE OF NUMERICAL ANALYSIS AND APPLIED MATHEMATICS, 2024.

[3] M. D’Amato & R. Sulla. Investigations of masonry churches seismic performance with numerical models: Application to a case study. *Archives of Civil and Mechanical Engineering*, *21*, 1-26, 2021.

[4] M. D'Amato, R. Gigliotti, R., R. Laguardia. Comparative seismic assessment of ancient masonry churches. Frontiers in Built Environment 5, 56, 2019.

[5] S. Cattari, D. Camilletti, A. M. D'Altri, S. Lagomarsino, S. On the use of continuum Finite Element and Equivalent Frame models for the seismic assessment of masonry walls. *Journal of Building Engineering*, *43*, 102519, 2021.

[6] E. Quagliarini, G. Maracchini, F. Clementi. Uses and limits of the Equivalent Frame Model on existing unreinforced masonry buildings for assessing their seismic risk: A review. *Journal of Building Engineering*, 10: 166-182, 2017.

[7] L. Gambarotta, & S. Lagomarsino, S. Damage models for the seismic response of brick masonry shear walls. Part II: the continuum model and its applications. *Earthquake engineering & structural dynamics*, *26*(4), 441-462,1997.

[8] R. Sulla, M. D'Amato, R. Gigliotti, D. Liberatore. Modeling criteria for the seismic assessment of existing masonry buildings. *Procedia Structural Integrity*, *44*, 998-1005, 2023.

[9] S. Lagomarsino, A. Penna, A. Galasco, S. Cattari. TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings. *Engineering structures*, *56*, 1787-1799, 2013.

[10] 3MURI. User manual. www.stadata.it.

[11] P. Roca, M. Cervera, G. Gariup, L. Pela', L. Structural analysis of masonry historical constructions. Classical and advanced approaches. *Archives of computational methods in engineering*, *17*, 299-325, 2010.

[12] 3MURI Professional – v. 13.5.0.0

[13] Software Midas Fea NX 2023 – v.1.1 – MIDAS Information Technology Co., Ltd

[14] Midas Fea NX User Manual – Chapter 4 Material, Section 2 - Plastic material properties (pp. 207-210)

[15]Midas Fea NX – Example of application

[16] DIANA-10.0 User's Manual - Material Library, First ed.

[17] CEB-FIP (1993).Model Code 1990, Comité Euro-International du Béton.

[18] Degree Thesis: UGOLINI, M., & SALVATONI, P. (2015). Comportamento di elementi in muratura fino a collasso: prove sperimentali e modellazione numerica.

[19] MIT, Ministerial Decree M.D. (17/01/2018), Aggiornamento delle "Norme Tecniche per le costruzioni" (in Italian). Rome, Italy: Italian Ministry of Infrastructure and Transportation, 2018.

[20] MIT, Instructions for the application of the Italian building code M.D. (17/01/2018), Istruzioni per l'applicazione dell'«Aggiornamento delle "Norme tecniche per le costruzioni"» di cui al D.M. 17 gennaio 2018 (in Italian). Rome, Italy: Italian Ministry of Infrastructure and Transportation, 2019.