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ORIGINAL ARTICLE

Evaluation of the seismic capacity of cultural heritage in case of aggregate buildings: the Castle of Rosignano Marittimo

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Abstract

The partitioning of masonry buildings in Structural Units (SUs), or aggregate of Structural Units, is a useful tool to simplify the analysis of the buildings and the study of static and seismic vulnerability. However, this allocation is affected by uncertainties, simplification, and sometimes unavoidable miscalculation due to the directionality of this subdivision. This procedure exposes the results to discrepancies due to the way the building is divided. The *Castle of Rosignano* is emblematic. In this paper, the dependence of the results on how the Structural Units are considered is analyzed, as well as the role of structural in plan and elevation irregularities on various SUs and their combination. Non-linear static analysis corresponding to different distributions of the building in Structural Units, the uncertainties about material properties, and different directions of the seismic action are analyzed, and the respective results are discussed here. A simplified approach, for the study of aggregate buildings on an urban scale, is based on the so-called "tabular method". Several Authors proposed and improved similar methods applied to different urban contexts. A comparison of simplified methodologies with the results of the detailed FEM analysis is also discussed and presented here. Finally, a simplified approach is proposed based on the regularity parameter of the building in aggregate. Taking the evidence from FEM analysis as a physical-mechanical base, the authors propose the quantitative definition of irregularity parameters and the use of them to determine the building vulnerabilities. The proposed procedure aimed to be a practical tool to determine, in an expeditious manner, the seismic capacity of a masonry building in aggregate. The model proposed in this paper, applied to the case of the study of *Rosignano Marittimo* (a typical situation of aggregate building in a historical context in Tuscany, Italy) shows emblematic results that can be extended to analogous configuration.

Keywords Masonry buildings · Structural units · Pushover analysis · Structural irregularity

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1 Introduction

Italy is one of the countries with the highest seismic risk in Europe. At the same time, Italy is one of the richest countries in terms of cultural heritage, not only by monuments (Ruggieri et al. 2018) but also by historic centres. The centres are mainly organized by aggregates of buildings, in which is not easy to investigate from structural point of view. Masonry buildings in historic centres are commonly the result of different construction periods. In fact, the original structural schemes are frequently modified to meet new architectural needs. This inevitably compromises the initial structural configuration and increases the difficulties in the assessment of the seismic response of such buildings through analytical or numerical procedures. Moreover, the evolution of the masonry buildings over time is often affected by geometrical (plan and elevation) (Alecci et al. 2019, 2020, 2024) and mechanical (texture, blocks, mortar) and material irregularities (Puppio et al. 2017, 2019). Material irregularities is a pivotal point in the seismic assessment of the existing buildings both in r.c. than in masonry (De Stefano et al. 2015; Müller et al. 2017) and can significantly influence the seismic response of the existing buildings.

A first strategy to extremely simplify the analysis of the above irregularities are the use of technical forms, to classify the vulnerability of the buildings of a historical city centre. The aim is to achieve a list of priority, in terms of seismic risk, to optimize the efforts versus the most vulnerable buildings of the centre. A further well-known way to perform a simplified seismic analysis, at the scale of singe building aggregate, is the division of the construction with a subset of Structural Units (SUs). It is also well-known that the division in SUs is affected by uncertainness in terms of results, together with the choice on the most severe earthquake directions to investigate (Magliulo et al. 2014; Cannizzaro et al. 2017).

Referring to the *Tabular Approach* at the scale of set of aggregates, based on vulnerability indexes, the European technical code EMS-98 (European Seismological Commission 1998) introduced the definition of vulnerability classes and intensities classification of earthquakes. The Abruzzo 2009 earthquake (Italy) furnished a wide series of examples in the historical city centre of L'Aquila, in which the classification approach was tested. D'Ayala and Paganoni (2011) focused on the influence of the lay-out and the quality of masonry on the seismic vulnerability of two typologies of buildings. Carocci (2012) observed the level of damage among a large number of historical aggregates in L'Aquila.

Ferreira et al. (2012) proposed a macro seismic approach for the evaluation of the vulnerability of an aggregate in the old city centre of Coimbra; they considered different parameters regarding the structural resisting system: the mechanical properties of the masonry, the presence of irregularities in plan and height, the interactions between buildings, misalignment of floor and roofs, the conservation status as well as other non-structural components. The same parameters were considered by Ferreira et al. (2013) who subsequently proposed a methodology to evaluate the seismic vulnerability, based on over 500 buildings in the old city centre of Seixal, in Portugal: a vulnerability index score was calculated for each building as the weighted sum of 14 parameters. The importance of a seismic vulnerability assessment on ordinary masonry buildings was underlined in Ferreira et al. (2017), through the application and calibration of a simplified method based on statistical approach and damage observation on 192 buildings in the old city centre of Horta, Azores.

Referring to the predictive models for assessing the seismic vulnerability of historic centres, Brando et al. (2017) and Rapone et al. (2018) proposed a predictive model to evaluate the seismic vulnerability of small historic centres. The applicative method was applied to a meaningful case study in Scanno (Abruzzo) and provide likely damage scenarios by means of fragility curves. Plan and vertical irregularities are frequently observed in historic buildings. The addition of one or more floors, may lead to discontinuities in mass, stiffness and strength of the walls along the height. Berti et al. (2017) developed a simplified method to measure the error between the application of the equivalent-frame modelling for the estimation of the in-plane seismic vulnerability in perfectly-regular masonry walls and in presence of geometric irregularities. Click or tap here to enter text. Alecci et al. (2024) propose a simplified numerical procedure to evaluate the influence of vertical irregularities on the seismic response of masonry walls along the building height. The influence of vertical irregularity is also discussed in depth by Magliulo et al. (2012) with interesting insight for the revision of the actual standard.

An interesting study on the torsional effects and geometric irregularities in the assessment of existing unreinforced masonry buildings can be found in Aşıkoğlu et al. (2020). In this study, a set of nonlinear static analyses are performed using three different modeling approaches based on: a continuum model, beam-based model, and spring-based macro-element models. The results and the performance of each method are discussed and a reasonable agreement between experimental evidence and numerical results is presented.

Many authors focused on the assessment of the seismic vulnerability of masonry buildings adopting the *Structural Units Approach* at the scale of single aggregate. The aggregate is commonly analysed by dividing into structural units (SUs): the dynamic response of each SU is strongly influenced by the presence of adjacent ones, may significantly differ from the seismic response of the whole aggregate. It means that the result of the analysis is influenced by the choice of the SUs subsets. Among the contributions available in literature, Senaldi et al. (2010) investigate the seismic response of a series of SUs aggregated in line, by means of non-linear dynamic analysis. Da Porto et al. (2013) carried out historic search, on-site assessment, surveys, modelling and analysis of a clustered building in the historic centre of L'Aquila, hit by the 2009 earthquake. A quick procedure for seismic vulnerability statement of masonry compounds was proposed in Formisano et al. (2015). The procedure was calibrated based on numerical analysis performed at different urban levels, namely small scale (single aggregates) and large scale (parts of historical centres). Finally, the proposed method was applied to some high seismic historical centres of Abruzzo and Campania (Italy) study cases. Fagundes et al. (2017) highlight the importance of modelling the structure with the adjacent buildings' floors at their actual height to assess the real expected damage.

A simplified approach for the seismic assessment of SUs extrapolated from an aggregate was presented in Formisano and Massimilla (2018). The position of the isolated building in the aggregate, as well as the influence of other constructions were considered through the definition of two coefficients corrective of the seismic response of the single unit. This methodology, applied to a simple masonry aggregate, results to be effective for the assessment of the seismic behaviour of a single SU without a deep knowledge of the structural details of the whole compound. Chieffo and Formisano (2019) evaluated the seismic vulnerability of a clustered building hit by the earthquake in 2009 through three different procedures; the aggregate effect on the seismic behaviour was evaluated and the results of the three approaches were compared. Cristofaro et al. (2019) presented the application of an empirical methodology aimed to the assessment of the seismic vulnerability of building aggregates of the historic centre of Scarperia, in Tuscany. For each aggregate, seismic vulnerability curves were obtained, and possible damage scenarios were defined. Degli Abbati et al. (2019) provided an operative tool consisting of a numerical procedure bases on the use of nonlinear static analysis to perform the global seismic assessment of mutually interacting masonry structures. The method was proved to be effective through an application to a medieval fortress damaged by the 2012 Emilia earthquake (Italy). The vulnerability of the perimeter walls, the large displacements of the units at the extremities and of the tall units was shown by Valente et al. (2019) through a 3D FE model. Two complex masonry aggregates were analysed by means a non-linear dynamic analysis, proving that the interaction with the adjacent unit plays a significant role on the structural response of a single unit.

In many cases, to establish a reasonable subset of SUs represents a non-trivial issue. Modelling the entire aggregate may results in significant computational costs, whereas considering an excessively large subset of SUs allows to reduce the computational cost, but it may lead to unrealistic analytical results. Bernardini et al. (2019) developed ad innovative procedure to identify Minimum Unit of Analysis (MUA), the optimal portion of the aggregate that best represents the “aggregate effect”. Two analysis procedures were proposed by Chiumiento and Formisano (2019) for the assessment of the seismic vulnerability of two structural units placed in intermediate and head position of a masonry building compound in the historical centre of Arsita, Teramo. The two approaches consisted of a simplified analysis, based on a vulnerability evaluation quick form, and a more refined one, based on macro-element modelling, implemented through 3Muri software. The analysis lead almost to the same vulnerability indexes and highlights the influence of the unit positions on the seismic vulnerability. Greco et al. (2020) applied an innovative macro-element (Caliò et al. 2005, 2012) approach for the estimation of the seismic vulnerability of historical unreinforced masonry building in Catania, Sicily.

Grillanda et al. (2020) presented an automatic computational procedure for the seismic assessment of masonry aggregate using the code ANUB-Aggregates. The whole aggregate was represented through NURBS (non-uniform rational basis splines) surfaces, allowing low computational efforts even in case of complex constructions including vaulted structures and curved openings. Senaldi et al. (2020) proposed a unidirectional dynamic shake-table test performed on a prototype of a natural stone masonry building aggregate. The half-scale prototype reproduces the features of existing unreinforced stone aggregates. The evolution of the dynamic response and damage mechanisms are illustrated. Battaglia et al. (2021) analysed 36 different structural configurations of a masonry aggregate obtained by changing a set of structural and geometrical parameters: it was observed that the configuration of the internal slabs particularly affects the seismic response of the building. Significant differences are also observed in the seismic performance of isolated and aggregate partially grouted reinforced concrete masonry. An automated mechanical-typological approach was proposed by Leggieri et al. (2021) to evaluate the seismic performance of masonry aggregates. The simplified procedure, based on a vulnerability algorithm, uses few mechanical and geometrical parameters of single buildings as input parameters. By allowing the software MATLAB to interact with a structural analysis software, all possible combination of the input parameters generates and analysed a large set of numerical models. An L-shape masonry aggregate composed by 3 adjacent edifices was examined by Angiolilli et al. (2021). Nonlinear dynamic analysis was used to assess the seismic vulnerability. The investigation on the effects of different connection types between the adjacent units on the structural response stated that the higher the connection level between adjacent structural units, the higher the interaction between in-plane and out-of-plane mechanisms at the collapse limit state. Formisano et al. (2021) assessed the seismic response of a clustered masonry building composed of four SUs considered in different configurations: the whole aggregate, the intermediate structural unit and the head structural unit grouped in aggregate and the same structural units in isolated condition. The results pointed out that strength and stiffness increased proportionally passing from SUs placed in isolated conditions to the corresponding ones grouped in aggregate. On the contrary, the ductility was more accentuated in the isolated SUs than in the same units in the aggregate configurations. Finally, seismic retrofit interventions were foreseen. The fragility curves highlighted a significant improvement of the structural performance of both the whole aggregate and single SU. Torres-Olivares et al. (2024) pointed out that differences in the rigidity and resistance of the modelled connections between adjacent buildings strongly affect the response of the entire aggregate, both in the linear and nonlinear range. The assessment of the "aggregate effect" represents one of the most challenging issues (Leonardi et al. 2024).

This paper aims to give a contribution in the field of optimal division of aggregate into SUs. In this sense the study of seismic vulnerability of a non-trivial case-study of masonry aggregate (the Castle of Rosignano Marittimo—Italy) is proposed. Originated in the IV cent A.D. and developed up to the present destination of municipality services, it consists of a representative chaotic texture of middle-age city centres. The geometry of the Castle has been here considered as a benchmark configuration, from which to investigate the influence of the following aspects:

(a) mechanical properties of masonry structure (by twelve different masonry types),
(b) direction of the earthquake (by polar variation of 18° in twenty different values),
(c) division in SUs subset (by ten different combinations of SUs).

The numerical results have been obtained via 3D Macro code of push-over analysis, presented in terms of Index of seismic Risk (IR). Moreover, a tabular method based on Vulnerability Index has been adopted, comparing the results in view to show the difference between Tabular and Structural Units approaches.

2 Description of the benchmark case study

The *Castle of Rosignano* is a masonry clustered building located at the top of the hill where the borough of Rosignano Marittimo arises (Livorno, central Italy). It consists of six slab levels with a ground extension of 925 m^2 and a maximum height of 17.20 m. The aggregate was built in different eras: the oldest origin is identifiable in the Etruscan/Roman epoch, but the first information on the structure can be dated around the Sixth Century (Longobard base). In medieval times, various destructions and reconstructions followed one another, until reaching the current shape of the structure of 1562.

As specified by the Italian Technical Code (named NTC 2018 and the relative Annex 2019), is relevant to assess the construction process and the historical evolution of the compound of the Castle. However, the historical data are sometimes insufficient to implement a reasonable evolution of the aggregate. It is therefore useful to perform an accurate geometrical and mechanicals survey and preliminary study of the existing buildings. Thus, the

Fig. 1 Drawings of the Castle: **a** East–West section **b** North–South section, **c** ground floor plan

Table 1 Variations of wall thickness and floor elevation

Level	Walls		Storeys		
	t_min	t_max	z_min	z_max	z_mod
	(cm)	(cm)	(cm)	(cm)	(cm)
Ground	30	95	250	265	265
First	30	90	540	640	595
Second	30	70	870	960	935
Third	30	70	1240	1245	1240
Fourth (roof)	30	40	1720	1720	1720

investigation carried out has allowed us to identify the structural elements, their state of conservation, the presence of cracking patterns, and possible ongoing collapse mechanisms.

The drawings of the building (Fig. 1) show significant vertical irregularities, caused by different rooftop heights between the SUs and by different heights of the staggered floors (Table 1). Moreover, the mass change among the levels is also due to vertical variations in the thickness of the walls. In addition, plan irregularities are evident, originated by the erratic texture of the masonry walls.

The aggregate, extracted from a more complex group of constructions as in Figure was then divided into four structural units (SUs) arranged in a planimetric U-shape (Fig. 2), depending on the destinations and the volumetric distribution of the walls and slabs:

- SU1: (RED) Office of the Municipality Government, with the clock Tower (cover 375 m^2);
- SU2: (CIANO) Municipality Council Room (cover 157 m^2);
- SU3: (YELLOW) Guest House (cover 243 m^2);
- SU4: (GREEN) Storage Area (cover 150 m^2);

Different typologies of slabs have been observed. Anyway, considering the recent refurbishment of the entire aggregate, in which steel and concrete reinforcements on each horizontal level are introduced, the hypothesis of rigid floors has been assumed for seismic calculations. Six levels of slabs have been modeled to represent the altimetric configuration, absorbing some negligible differences in the height of floors. The entire building is founded on a bedrock, so the model has been perfectly fixed to the ground.

The range of wall thickness is comprised between 30 and 95 cm as specified in Table 1; the different floor heights are also described in the same table, in which the maximum variation is 95 cm. The conventional height assumed in the model is named "z_mod" and consist of weighted average of the height respect to extension of each plan. In-situ experimental tests have been carried out to assess the mechanical characteristics of the masonry. A significant material heterogeneity is observed: Table 2 describes twelve different materials observed on the walls. The data set of mechanical properties furnished the opportunity of investigating the influence of the material: the Castle was firstly modelled with the real set of twelve materials assigned to each wall (real model).

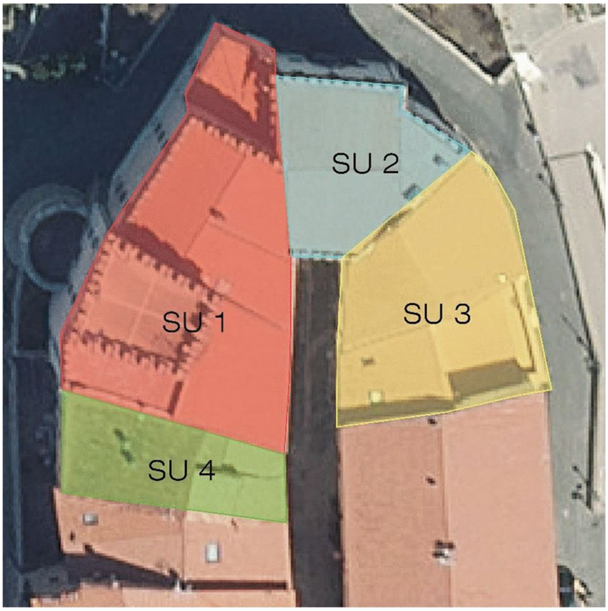


Fig. 2 The aggregates of the Castle and the complex of constructions

Table 2 Mechanical characterization of walls

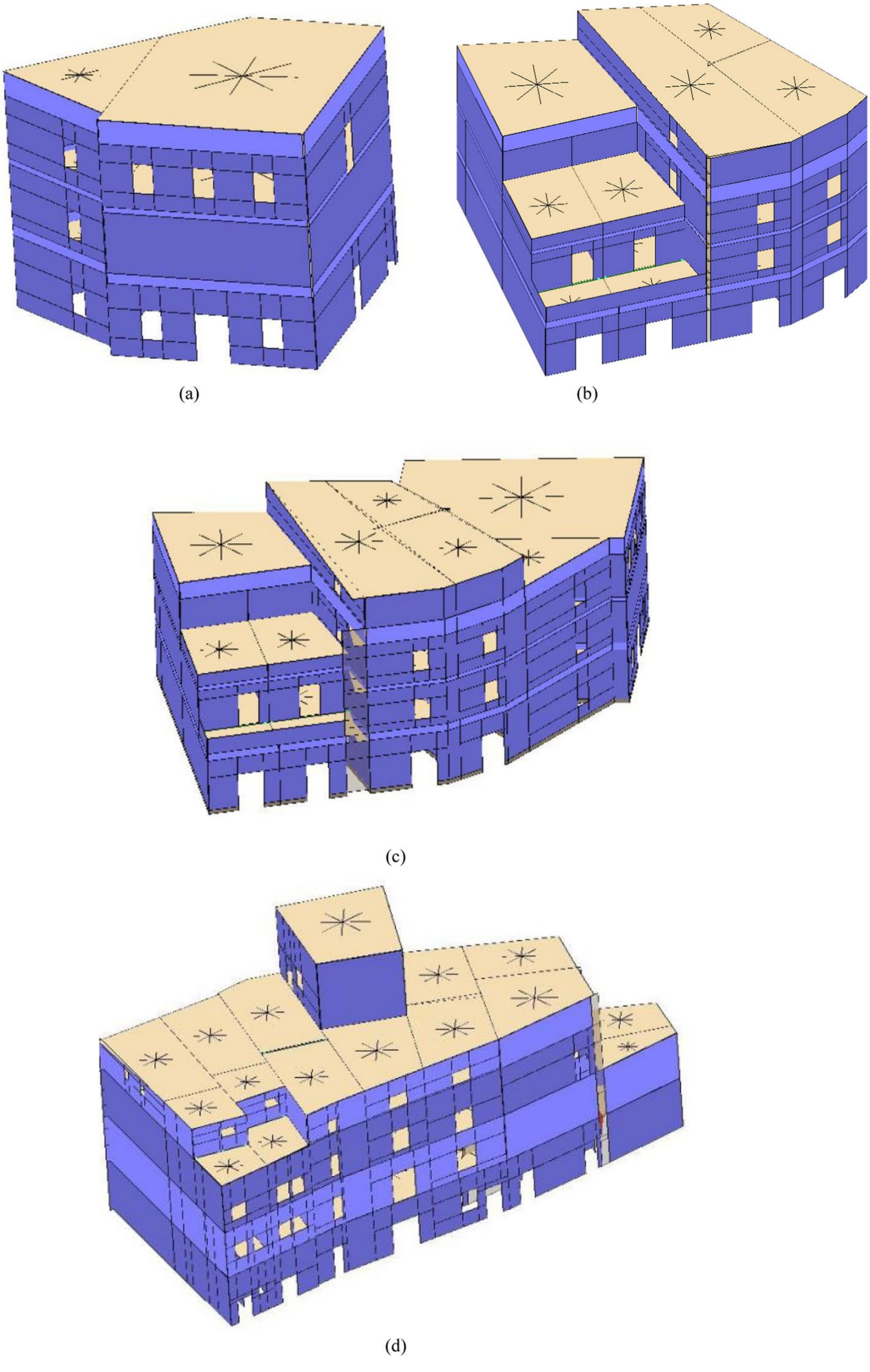
Masonry typologies	Compressive strength f	Shear strength τ_0	Elastic modulus E	Specific weight w
	(N/mm^2)	(N/mm^2)	(N/mm^2)	(kN/m^2)
Mat_01	0.74	0.01	511	19
Mat_02	0.83	0.02	575	19
Mat_03	1.00	0.02	690	19
Mat_04	1.93	0.04	1111	21
Mat_05	2.17	0.05	1250	21
Mat_06	2.60	0.06	1500	21
Mat_07	1.04	0.02	666	16
Mat_08	1.17	0.02	750	16
Mat_09	1.40	0.03	900	16
Mat_10	1.93	0.04	888	18
Mat_11	2.17	0.04	1000	18
Mat_12	2.60	0.05	1200	18

The entire construction was then modelled twelve times, considering the mechanical characteristics from Table 2 uniformly distributed on the entire structure (conventional models). The aim of the work is not only to investigate the seismic vulnerability of the case study itself but to explore, throughout a non-trivial geometric configuration the variability of the results, emphasizing their dependence on several parameters. The parameters examined here are: the mechanical properties of masonry, the direction of the earthquake, and the subset of structural units in which the construction is divided. "Mat_07" represents the predominant mechanical characteristics in the Castle, so the discussion of the other aspects, investigated for all the twelve materials, is here presented only with this material.

3 The structural units approach

The FEM software 3D Macro is here adopted to perform a series of push-over analyses based on macro-elements. The idea was to simulate the usual calculations assessed by professional engineers, exploring all the possible associations of SUs. In this sense ten different structural schemes were obtained: four singles (SU1; SU2; SU3; SU4) three coupled (SU1+SU2; SU1+SU4; SU2+SU3) two triplets (SU1+SU2+SU3; SU1+SU2+SU4), one entire structure (SU1+SU2+SU3+SU4). Some examples are in Fig. 3. A set of 120 Conventional Models was then prepared (12 materials; 10 configurations) comparing the results with the "Real Model" of the entire aggregate configuration composed by the real distribution of the materials. For each model, the directions of the earthquake were applied considering the different directions of the seismic action, named here "seismic rose domain" (Table 3).

The angle variation of 18° respect to the fundamental Northern direction is chosen investigating a set of 20 earthquake directions for each model. This approach expanded the common rule of Italian and EU norms where the main directions X and Y of the earthquake are combined by the rule X+/–0,3 Y; Y+/–0,3 X. For each direction, two different distributions of seismic loads (mode shape and uniform shape) were performed along the height of the structure.



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

Fig. 3 Example of FEM Model SU2 (**a**), SU3 (**b**) and SU2+SU3 (**c**) SU1+SU4 (**d**)

Table 3 Model names and structural units

Models	Structural units	Models	Structural units
Model_01	SU1	Model_06	SU2+SU3
Model_02	SU2	Model_07	SU1+SU2
Model_03	SU3	Model_08	SU1+SU2+SU4
Model_04	SU4	Model_09	SU1+SU2+SU3
Model_05	SU1+SU4	Model_10	SU1+SU2+SU3+SU4

Finally, the investigation involved 12 materials, 10 structural configurations, 20 seismic directions, and 2 seismic modes for push-over, addressing to Life-Safety Limit State (LSLS). An array of 4800 numerical experiments was obtained, deducing a wide distribution for Index of Seismic Risk (IR) given by

$$\mathrm{IR} = \mathrm{d_c}/\mathrm{d_d}$$

where the Capacity displacement of each structure (d_c) is compared with the corresponding Demand (d_d).Fig. 4 shows the variation in the seismic response of MOD_10, in terms of capacity curve, as the masonry material varies, for the earthquake direction of 72°. Each diagram shows the capacity curve associated with a different control point (CP) of the structure. The control points correspond to the centres of gravity of the slabs of the top floor.

Fig. 5 shows the Medium and Minimum IR of all the geometrical configurations for the predominant material (n.07). The medium value of IR is the average value from sixteen IRs obtained from the several directions and distribution of seismic forces. The minimum IR value corresponds to the smallest one from the same set. The medium value of IR takes conventionally into account the response of the SU for all the seismic directions, whilst the minimum value describes the most severe earthquake direction.

For synthesis, the following results are explained on the weakest configurations given by SU1, SU4, and SU1 + SU4. The diagram of the polar description in Fig. 6 shows the great variability of the IR from the SUs and the earthquake direction.

Medium and minimum values of IR for all the twelve materials (SU1; SU4; SU1 + SU4) are in Fig. 7. It highlights the relevant variability of the results from the mechanical properties.

It can observe that the polar description of IR, in terms of the "Seismic Rose domain", improves the investigation concerning the classic ones: the designer can here better address the reinforcements of the walls, increasing the strength of the walls along the weakest directions. So, this element can lead the designer towards a smart seismic retrofitting of an existing building. In the meanwhile, the risk of ignoring weak directions is here virtually excluded. It can be also helpful to evaluate minimum, medium, and characteristic values of IR on the entire distribution along 360°, before and after refurbishment, to achieve more awareness about the seismic risk achieved for the construction.

Moreover, the results in Fig. 5, referring to ten configurations of SUs, shows also that the most irregular configurations (Model 9 and Model 10) are affected by the lowest IR. It mainly depends on the hypothesis of rigid slabs, less reasonable in those irregular configurations: in presence of irregular planimetric shapes, the deformability of slabs should be taken into account. The combination of SUs should be carefully evaluated by the designer: joining different SUs increases the collaboration among walls whilst, at the same time, can increase the planimetric irregularity. Usually, it is difficult to establish the optimal configuration of

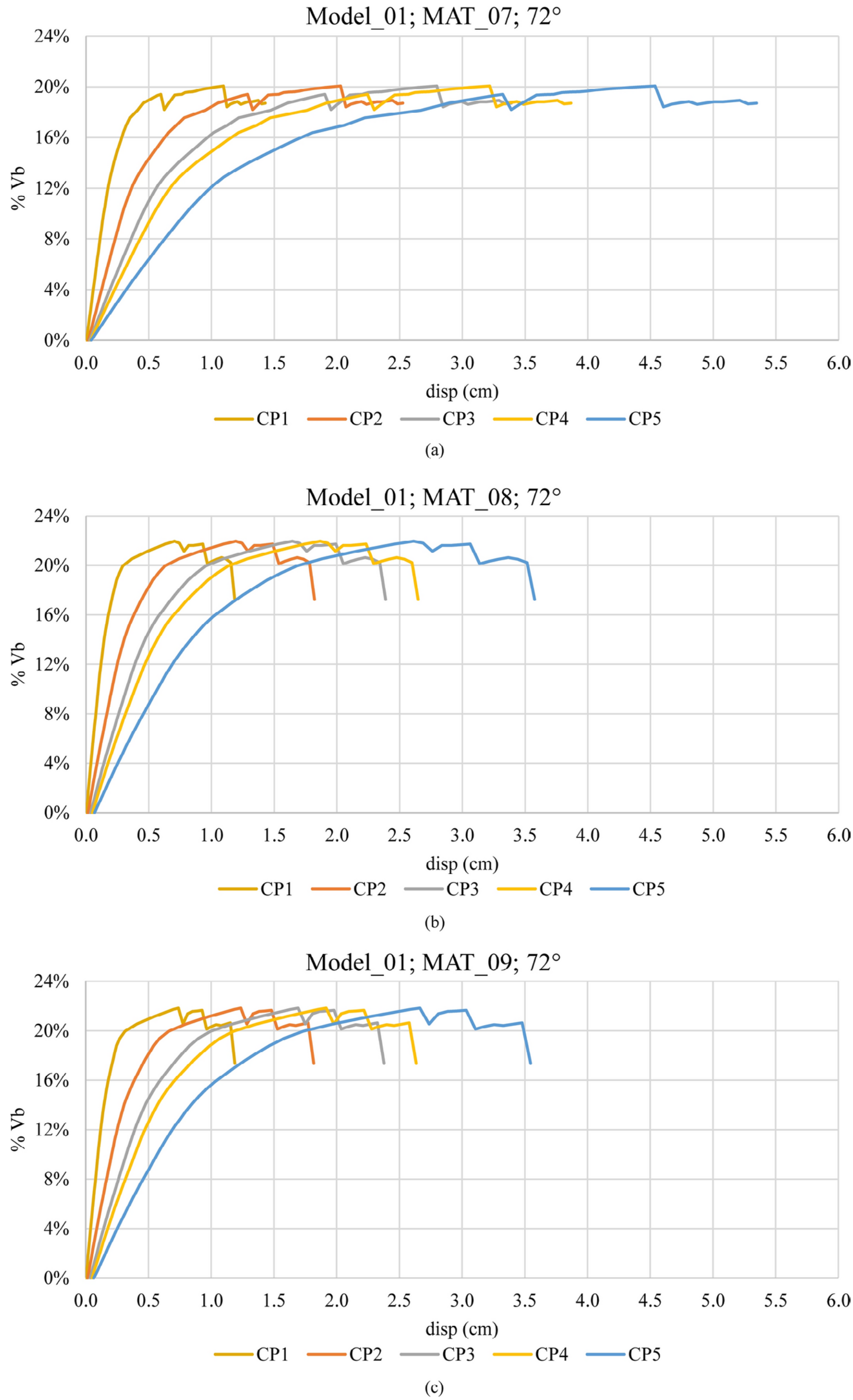


Fig. 4 Example of capacity curvers of "Model_01" and "MAT_07" (**a**), "MAT_08" (**b**) and "MAT_09" (**c**)

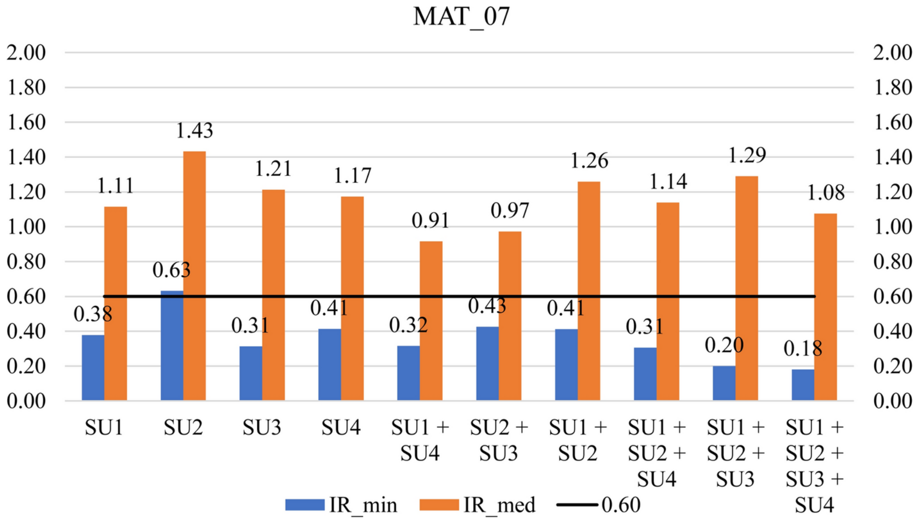


Fig. 5 Index of Risk (IR) for all the configurations in case of material n.07

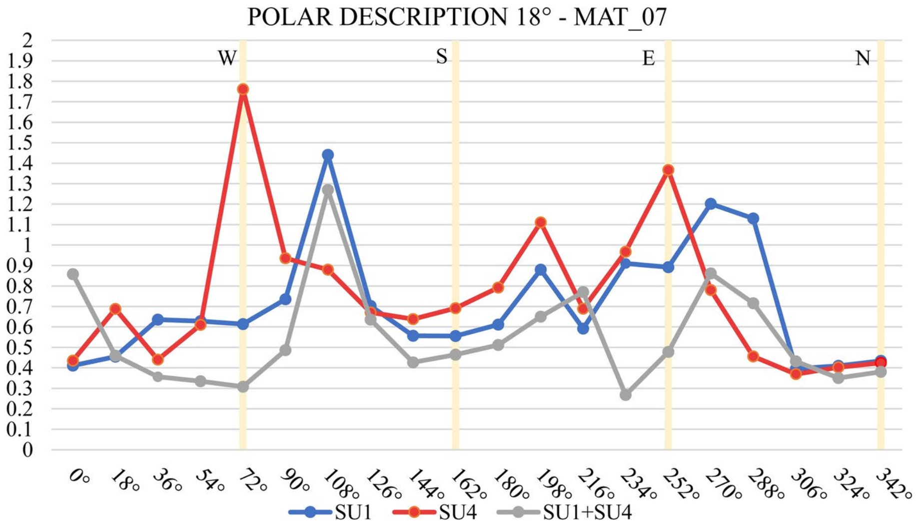


Fig. 6 Polar description of Index of Risk (IR) for SU1; SU4; SU1 + SU4 in case of "MAT_07"

SUs. From a different point of view, the strategy of the Tabular Method, described in the further section, gives interesting insight to the definition of SUs for a more effective evaluation of IR.

In the opinion of the authors these results can be really useful for designers who deals with the topic of modeling floors. In this case two reasons influence the result:

1. The first and most relevant is the presence of staggered levels generates squat elements in the modeling whose role is made more critical in the model with rigid floors.

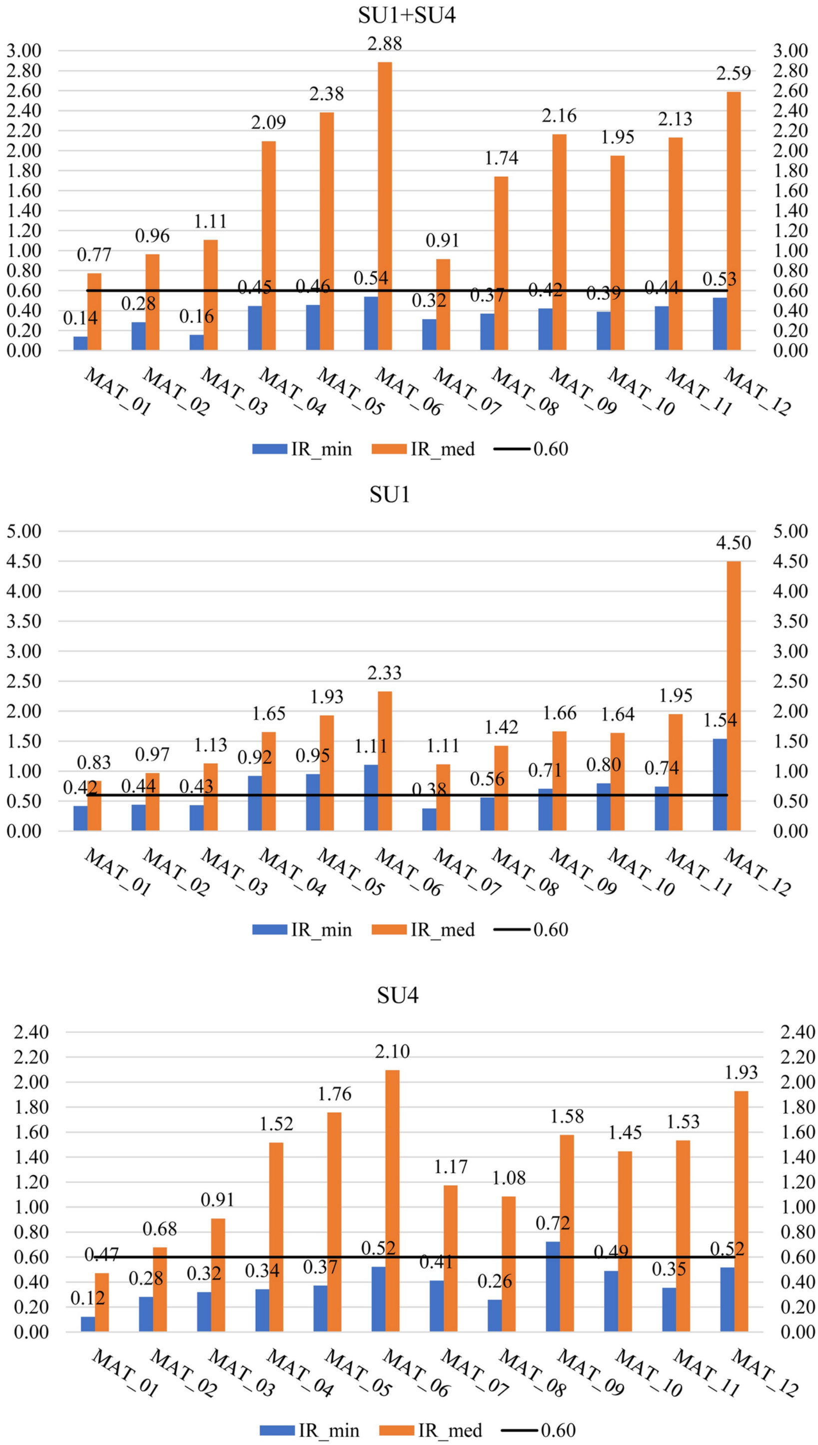


Fig. 7 Medium and minimum Indexes of Risk (IR) for SU1; SU4; SU1 + SU4

2. The irregular geometry of the plane and the considerable extension generate larger torsional effects in the model with rigid floors that could be less reasonable.

4 Tabular method

The study, by way of vulnerability indexes, was originally developed by Benedetti and Petrini (1984), implementing a tabular expeditious method for the assessment of the vulnerability index for isolated masonry buildings. It was later extended to aggregate masonry buildings by the Authors cited in Formisano et al. (2011). The original method consisted of four vulnerability classes (A–B–C–D), defined in ascending order of risk and 10 parameters, representing the geometric and mechanical characteristics of the building. The method was later integrated with 5 new parameters representative of the aggregate condition. The new 5 parameters can increase or reduce the vulnerability of a structural unit inserted within a block of buildings. Each class corresponds to a score, p_i, whereas each parameter corresponds to a weight w_i, which symbolise the influence that the same parameter has on the global vulnerability of the structure. The vulnerability index is calculated as the sum of the scores multiplied by the respective weights.

$$I_{v,i} = \sum_{i=1}^{10} p_i \cdot w_i$$

$$I_{v,c} = \sum_{i=1}^{15} p_i \cdot w_i = I_{v,i} + \sum_{i=11}^{15} p_i \cdot w_i$$

This methodology is suitable for capturing the vulnerability of both isolated and aggregated buildings, as it takes into consideration the structural interaction between adjacent masonry structures.The following table shows the vulnerability indexes for both the isolated SUs, $I_{v,i}$ (Model_01, Model_02, Model_03, Model_04) and the aggregate one, $I_{v,c}$ (Model_10).

It can be noticed that higher risk indexes are obtained for the isolated condition (Model_03). According to the tabular method, the isolated structural units are more vulnerable to earthquakes than the same structural units considered in aggregate. The presence of adjacent buildings gives the individual SU a greater containment of movements, improving the behaviour under seismic action.

The Tabular Method is a pseudo-quantitative method to define the seismic vulnerability of existing buildings. This is based on the categorization of the examined building into classes and the calculation of indexes related to some elements of vulnerability. Considering the example of this paper it can be calibrated to obtain results analogous to FEM models.

4.1 Validation and comparison between Tabular Method and FEM analysis

A comparison between the vulnerability indexes obtained using the tabular method and the mechanical indexes obtained from the finite element methodology was performed to evaluate the validity of the simplified approach proposed by Formisano for the present case study.

Table 4 Vulnerability indexes from Tabular Method (aggregate and isolated conditions)

Vulnerability indexes		$I_{v,c}$	$I_{v,i}$
Model_01	SU1	95	123
Model_02	SU2	75	188
Model_03	SU3	140	218
Model_04	SU4	45	88
Model_10	SU1 + SU2 + SU3 + SU4	105	118

Table 5 Mechanical risk indexes from FEM analysis (aggregate and isolated conditions)

Risk indexes		FEM		TM	
		IR_{min}	IR_{med}	$I_{M,min}$	$I_{M,med}$
Model_01	SU1	0.38	1.11	2.65	0.90
Model_02	SU2	0.63	1.43	1.58	0.70
Model_03	SU3	0.31	1.21	3.20	0.82
Model_04	SU4	0.41	1.17	2.42	0.85
Model_10	SU1 + SU2 + SU3 + SU4	0.18	1.08	5.56	0.93

For this purpose, only "Material 7" was considered, as it represents the material with the most frequently observed average characteristics throughout the entire complex. Referring to the results achieved with the tabular method (Table 4), it can be noted that higher vulnerability indexes (I_v) are achieved when the SUs are studied in the isolated condition, thus resulting in more vulnerability to earthquakes in this specific condition.

The opposite behavior can be observed from the results derived using the mechanical method. The minimum and average risk indexes (IR_{min}, IR_{med}) resulting from FEM analysis of the entire aggregate are lower than those obtained from the models referring to the individual structural units. The values derived from the FEM analysis and the corresponding mechanical indexes, defined in terms of the ratio between demand and capacity, are presented for subsequent comparison with the tabular method (Table 5).

Remembering the physical meaning of the parameters reported in the previous Tables 4 and 5, the I_v values in Fig. 8 are compared.

- IR i.e. the seismic risk index from FEM analysis defined as the ratio between capacity and demand displacements referring to the Life-Safety Limit State (LSLS), $d_{C,LSLS}/d_{D,LSLS}$;
- $I_M = 1/IR$ i.e. the mechanical index understood as the ratio between $d_{D,LSLS}/d_{C,LSLS}$;
- $I_{v,c}$ tabular vulnerability index obtained by considering 15 parameters (aggregate condition);
- $I_{v,i}$ tabular vulnerability index obtained considering 10 parameters (isolated condition).

The results obtained from the two methodologies shows significant discrepancies. It is evident, therefore, how vulnerability analysis for buildings in aggregate requires careful evaluation, as the comprehensive analysis of individual units can lead to markedly different outcomes compared to the analysis of the entire structural complex. This leads to the conclusion that the validity of the simplified methodology for our case study has not been confirmed. In fact, at the scale of the building, the simplified procedure furnishes significant discrepancy to more advanced methods.

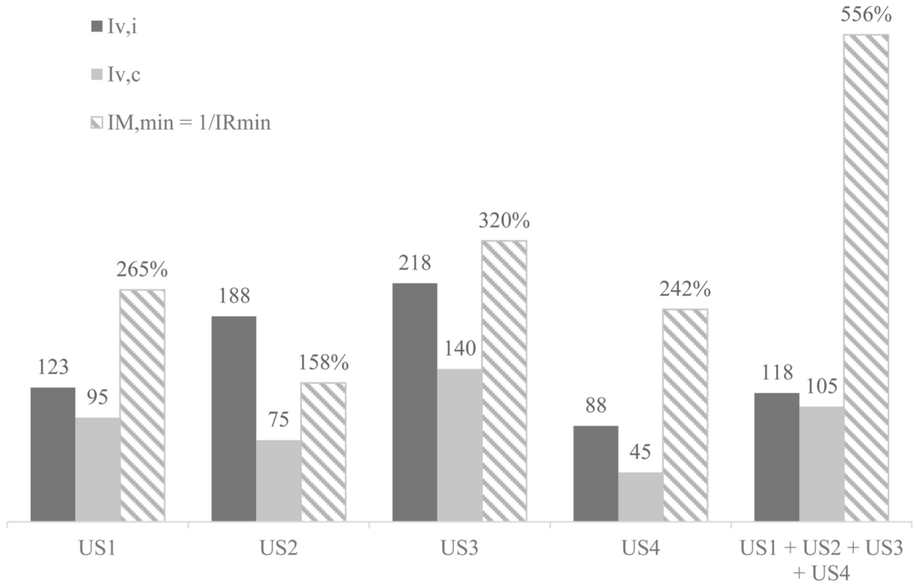


Fig. 8 Comparison between tabular and mechanical method

As mentioned earlier, this is explained by the significant irregularity in plan and elevation observed in our case study. Therefore, it can be asserted that the Formisano method is suitable for buildings that differ from ours and, consequently, exhibit adequate regularity and compactness.

4.2 Correlation between risk indexes and regularity of the aggregate: least squares method

Given the considerations set out in the previous paragraph, the next step was to search for regularity parameters that would correlate the tabular method with the results deriving from the FE modeling of our case study. It was therefore chosen to consider the regularity parameters in plan and elevation of the Italian National Code (2018) in § 7.2.1 and the related Annex (2019) in §C7.2.1, listed in Table 6.

Each parameter has been defined using a percentage rate that expresses the rank of the regularity of the structure. Specifically, higher percentage rates correspond to high regularity, whereas lower rates indicate low regularity. The unit value of the rate coincides with the limitation imposed by the considered technical code.

The following table presents a numerical example of the determination of the plan regularity parameter rate for facade recesses. It is here shown, with the statements and definition of Fig. 9, the variation of this parameter in terms of regularities fixing the A_0 terms. As shown in the following Table 7, for the various example cases, there is a decrease in the percentage rate as the planimetric extension of the recess increases.

All the regularity parameters ($p_{,i}$) in percentage terms defined for the 10 models are then reported in the following Table 8.

Table 6 Regularity parameters

Parameter	Description	Regularity	Expression
1	Presence of setbacks on the facade	Plan	$\frac{A_i}{A_0} \leq 0.05$
2	The ratio between the sides of the rectangle circumscribed around the plan of each slab is less than 4	Plan	$\frac{L_1}{L_2} \leq 4$
3	The distribution of masses and stiffnesses is approximately symmetric with respect to two orthogonal directions	Plan	$\frac{K_x}{K_y} \cong 1$
4	Extension of the resisting systems throughout the entire height of the structure	Elevation	$\frac{H_i}{H_{tot}} \cong 1$
5	Any narrowing of the horizontal section occurs in such a way that the setback of the slab does not exceed 10% of the corresponding dimensions of the slab immediately below, nor 30% of the dimension of the first	Elevation	$\frac{L_{i-1}-L_i\prime}{L_{i-1}} \leq 10\%$ $\frac{L_0-L_i}{L_0} \leq 30\%$
6	Mass variation from one slab to another do not exceed 25%	Elevation	$\Delta M \leq 25\%$
7	The stiffness does not reduce from one slab to the one above my more than 30%	Elevation	$\Delta K^{(-)} \leq 30\%$
8	The stiffness does not increase from one slab to the one above by more than 10%	Elevation	$\Delta K^{(+)} \leq 10\%$

The final step of the comparison process was to fit the 8 regularity parameters adopted using the 10 models that provide the data themselves, to obtain a seismic risk index that was as close as possible to the result derived from the FEM analysis. This goal was achieved using the least squares method. The weights were therefore calculated by minimizing the difference between the risk indexes obtained from the regularity parameters (IR_{tab}) according to the tabular approach and the risk indexes derived from the FEM analysis ($IR_{med,\ FEM}$) (Tables 9 and 10).

$$IR_{tab} = \sum_{i=1}^{8} p_i \cdot w_i$$

This procedure is validated by finding a satisfactory correlation between the weighted sum of the regularity parameters and the safety factors for the 10 models under study. This correlation gave in output a coefficient of determination equal to R2 = 0.997 as shown in Fig. 10 which indicates an interesting adaptation of the 10 samples to the interpolating line and therefore a negligible error in the solution search.

Furthermore, it can be considered to determine the direct correlation between the weighted sum of regularity parameters and safety factors. And, for this purpose, it can be extended to a more relevant statistical set and consequently be useful for the development of future studies.

It can be stated that this method is particularly relevant as it allows to conduct a rough estimate of the global vulnerability of the aggregate and allows, even before proceeding with non-linear modeling and analysis, to have a synthetic indicator of its seismic performance available.

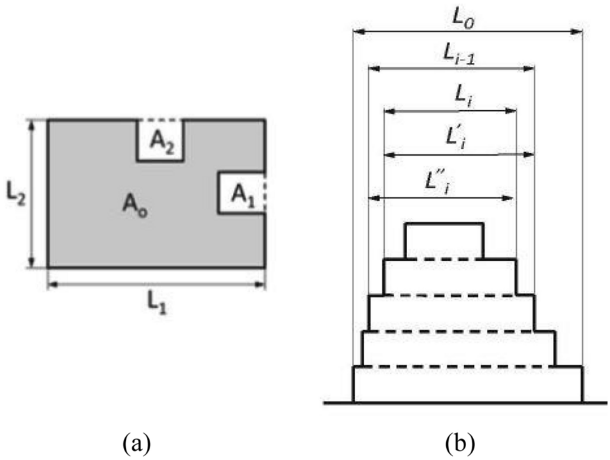


Fig. 9 Parameter 1 e 2 (**a**) and Parameter 5 (**b**)

Table 7 Plan regularity parameter for facade recesses

Case	A_0 (m^2)	A_i (m^2)	A_i/A_0 (–)	$(Ai/A_0)_{lim}$ (–)	$(Ai/A_0)_{lim}/(A_i/A_0)$ (–)	Regularity
1	100	1	0.01	0.05	500%	Yes
2		2	0.02		250%	Yes
3		3	0.03		167%	Yes
4		4	0.04		125%	Yes
5		5	0.05		100%	Yes
6		6	0.06		83%	No
7		10	0.10		50%	No
8		15	0.15		33%	No
9		20	0.20		25%	No
10		30	0.30		17%	No

5 Discussion and conclusions

In this paper, a study of a masonry building in aggregate is performed throughout FEM Macro-element approach and Tabular Method approach. The results of FEM investigation are deeply influenced by the following aspects:

(1) choice of different set of structural units;
(2) mechanical properties of the material;
(3) direction of the seismic action.

The choice of different sets of aggregates can lead to quite different results and thus to discretional design requirements from the analyses. It is then to recommend to perform analysis with several subset of aggregates—with the help of tabular method—before the choice of the one to adopt in the design and verification phases: the example of aggregate in this article can be used as a possible guide-line.

Another aspect to point out is that in case of complex aggregates the distribution of experimental tests should influence the results by different masonry materials: a compared analysis varying the mechanical properties could then reduce the uncertainness of the results for the designer.

Moreover, the evaluation of IR_{min} and IR_{med} can also address the structural rehabilitation along the weakest set of walls, reducing the difference of response respect to the direction of the earthquake.

The comparison of the FEM analysis with the Tabular Method in the present benchmark case, by the way of a large array of numerical launches, permitted to calibrate the tabular parameters. A proposal of new parameters based on the actual definition of structural regularity is here introduced and defined with numerical parameters. To study the effectiveness of this proposal these parameters are therefore used to develop a risk index, and the respective results are compared to the one obtained from the FEM model. This procedure consents to determine some weights or “important factors” that aims to tune the proposed model with the result of the real case of study.

Table 8 Regularity parameters of the case studies

p	Condition	Regularity	Mod_01	Mod_02	Mod_03	Mod_04	Mod_05	Mod_06	Mod_07	Mod_08	Mod_09	Mod_10
			SU1 (%)	SU2 (%)	SU3 (%)	SU4 (%)	SU1 + SU4 (%)	SU2 + SU3 (%)	SU1 + SU2 (%)	SU1 + SU2 + SU4 (%)	SU1 + SU2 + SU3 (%)	SU1 + SU2 + SU3 + SU4 (%)
1	A_i/A_0	In plan	14	28	30	12	20	19	10	9	22	16
2	L_1/L_2		247	393	311	326	200	308	382	350	322	389
3	K_x/K_y		48	46	47	17	41	69	54	45	51	45
4	H_i/H_{tot}	In elevation	40	100	29	83	40	29	40	40	21	21
5	L_i/L_0		49	100	26	100	41	26	39	39	16	16
6	ΔM		33	40	48	40	31	33	33	31	30	30
7	$\Delta K^{(-)}$		30	94	52	143	30	56	30	30	30	30
8	$\Delta K^{(+)}$		3	42	5	10	4	6	4	6	4	6

Table 9 Weights of the regularity parameters

Nr.	Condition	Regularity	w (–)
1	A_i/A_0	In plan	−0.96
2	L_1/L_2		−0.08
3	K_x/K_y		0.59
4	H_i/H_{tot}	In elevation	0.44
5	L_i/L_0		−0.14
6	ΔM		0.88
7	$\Delta K^{(-)}$		0.03
8	$\Delta K^{(+)}$		0.60

Further investigation on a wide population of masonry buildings of homogeneous characteristics will consent to extend the affordability of the proposed parameters for the tabular method, useful for quick evaluations in terms of distribution of resources.

Table 10 Risk indexes

Risk index	Mod_01	Mod_02	Mod_03	Mod_04	Mod_05	Mod_06	Mod_07	Mod_08	Mod_09	Mod_10
	SU1 (%)	SU2 (%)	SU3 (%)	SU4 (%)	SU1+SU4 (%)	SU2+SU3 (%)	SU1+SU2 (%)	SU1+SU2+SU4 (%)	SU1+SU2+SU3 (%)	SU1+SU2+SU3+SU4 (%)
$IR_{min,FEM}$	37.80	63.13	31.22	41.27	31.50	42.54	41.20	30.61	20.00	18.00
IR_{tab}	37.95	62.82	30.56	41.32	32.12	42.85	36.64	34.88	20.48	18.38

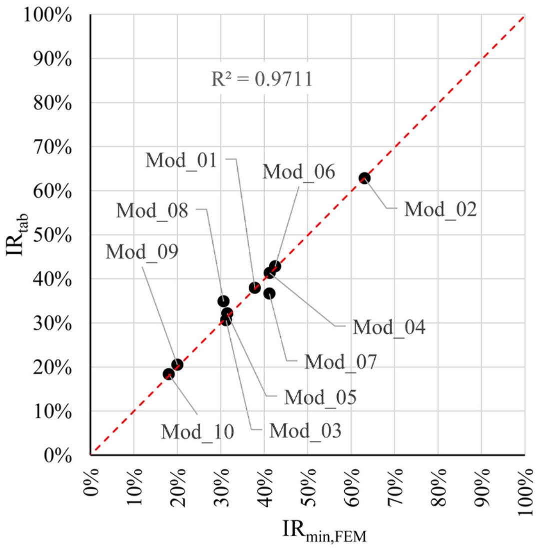


Fig. 10 Comparison between risk indexes (IR_{tab} vs $IR_{min,FEM}$)

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Author contribution All authors contributed to the study conception and design. Material preparation, data collection and analysis were performed by Fabio Doveri and Martina Ferrini. The original methodology and data approval was performed by Mario Lucio Puppio and Mauro Sassu. The first draft of the manuscript was written by Mario Lucio Puppio, Matteo Ariu and Mariangela Deligia and all authors commented on previous versions of the manuscript. All authors read and approved the final manuscript.

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Data availability The datasets generated during and/or analyzed during the current study are available from the corresponding author on reasonable request

Declarations

Conflict of interest The authors have no relevant financial or non-financial interests to disclose.

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