Derivation of Fragility Curves of Masonry Buildings in a Row Aggregate Located in Mirandola (MO)

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Abstract. The assessment of the seismic vulnerability of masonry buildings in aggregate typical of historic centers is a highly complex and widely debated topic. A review of the current state of the art reveals several critical and open issues. The aim of this work is to contribute to the modeling of the interaction between structural units (SUs) and the so called "aggregate effect". Given the complexity of the theme, this paper focuses on analyzing an aggregate with a regular plan configuration, allowing for the investigation of the in-plane (IP) response of both the entire aggregate and individual SUs, without the influence of potential torsional effect associated with plan irregularity. The aggregate selected for the analysis is a row aggregate located in Mirandola (MO), an area that was affected by the seismic swarm of May 2012. The wall panels of the SUs are made of solid bricks with variable thickness, while the floors, predominantly wooden, are arranged parallel or perpendicular to the street side. A 3D numerical model was performed according to the equivalent frame modeling strategy with the Tremuri software. Special attention was given to evaluating the historical evolution of the aggregate in order to accurately model the boundary conditions of each SU. Two different modeling criteria are proposed, depending on the interaction between the SUs. Through nonlinear dynamic analyses (NLDAs), fragility curves for IP vulnerability were derived for both the entire aggregate and some SUs in their isolated configurations.

Keywords: Buildings in aggregate, Historic centers, Equivalent-frame model, In-plane response, Fragility curves.

1 Introduction

Assessing the seismic vulnerability of masonry buildings in aggregate presents a complex challenge due to the interaction between Structural Units (SUs) and the variability introduced by historical urban development. In historic centers, buildings in aggregate result from a stratified construction process over time, characterized by extensions, vertical additions, and structural modifications [1].

These transformations have led to significant heterogeneity in construction properties and interaction mechanisms among the units. This evolutionary process has produced variations in materials, differences in wall thicknesses, irregular geometric configurations, and, in many cases, ineffective connections between adjacent SUs.

The seismic response of such aggregates is strongly influenced by several factors, including the quality and type of materials used, the arrangement of floor diaphragms, the presence of reinforcements, and, most importantly, the degree of connection between adjacent units. The interaction between SUs can have both beneficial and detrimental effects. On the one hand, structural continuity between units can enhance the overall strength of the aggregate, reducing local collapse mechanisms and improving energy dissipation capacity. On the other hand, irregularities in connections, differences in stiffness between SUs, or misalignments between floors and load-bearing walls can amplify instability phenomena and increase overall seismic vulnerability.

Despite the significance of these aspects, the scientific literature does not yet provide a unified framework regarding the effects of aggregation and the mutual influence between SUs [2]. The complexity of assessing the seismic vulnerability of masonry aggregates has led to a growing interest in experimental and numerical studies. Notably, the AIMS project [3] part of the Horizon 2020 initiative, involved shake-table tests on a two-unit stone masonry aggregate, highlighting the significant uncertainties in modeling such structures. Seismic vulnerability assessment methodologies for aggregates vary considerably and may rely on empirical, analytical, or numerical approaches, each with specific advantages and limitations. The lack of specific regulations and standardized criteria for evaluating historic masonry aggregates further complicates the development of effective risk mitigation strategies. Addressing this challenge requires methods capable of capturing the unique characteristics of aggregates by integrating historical knowledge, experimental investigations, and advanced numerical modeling.

In this context, simulation software based on the Equivalent Frame (EF) model, such as Tremuri [4], provides a valuable tool for assessing the in-plane (IP) seismic behavior of masonry aggregates. This methodology strikes an effective balance between accuracy and computational efficiency, allowing for a detailed representation of the IP response of both the entire aggregate and individual SUs while maintaining manageable computational demands.

Nonlinear Dynamic Analyses (NLDAs) are particularly advantageous in this setting, as they enable a more realistic assessment of force redistribution within the aggregate and a more precise evaluation of the interaction effects between SUs under seismic loading. This approach overcomes the limitations of nonlinear static analyses (NLSAs), which often fail to adequately capture the complex dynamic response of aggregates with irregular configurations and variable connections between SUs.

The selected case study represents a linear aggregate with a relatively regular layout. This configuration minimizes the influence of torsional effects, allowing the study to focus primarily on IP response analysis. A key aspect of the investigation involves defining the boundary conditions for each SU, accurately reflecting the historical evolution of the aggregate. This enables a more reliable identification of critical vulnerability zones and provides insight into the mechanical interactions between adjacent units.

The fragility curves derived from numerical analyses will be a fundamental outcome of this study, offering detailed insights into the IP vulnerability of both the entire aggregate and individual SUs. The results will provide a significant contribution to understanding the seismic behavior of linear masonry aggregates, forming a solid basis for future parametric analyses aimed at investigating the key factors influencing vulnerability.

2 Case study

The case study analyzed concerns a building aggregate located in the historic center of Mirandola (MO), dating back to the 19th century and consisting of eighteen SUs spread over an area of approximately 1166 square meters. The specific portion of the aggregate studied includes seven SUs, as shown in *Fig. 1*.

Fig. 1 Floor plan and elevations of the analyzed portion of the aggregate. The SUs analyzed both in the isolated configuration and within the aggregate are highlighted in dark grey. The walls without openings are highlighted in red.

The heights of the units range between two and three floors, as visible in the elevation. A detailed analysis allowed for the study of the historical evolution of the SUs and their interactions, identifying which units are structurally independent, interlocked, or adjacent. Units A, D, E, and G were classified as independent, while

B is adjacent to unit A and SUs C and F are interlocked. The hypothesized historical evolution of the aggregate is represented in *Fig. 2*.

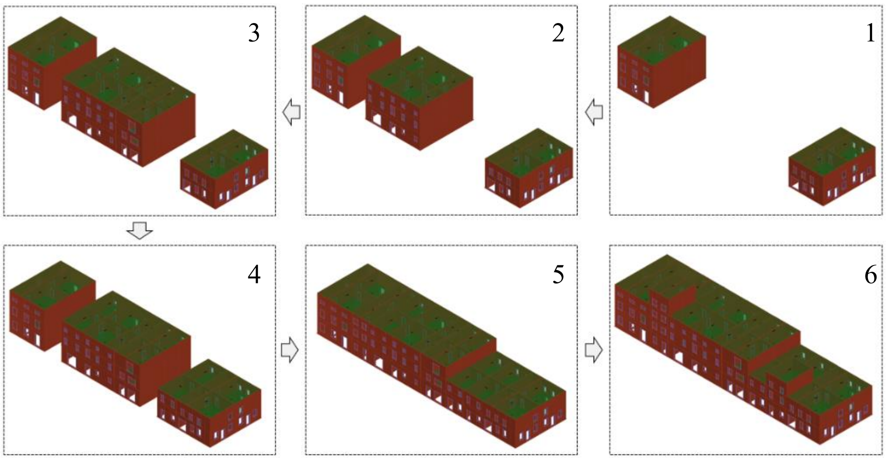

Fig. 2 Representation of the historical evolutionary phases of the buildings in aggregate.

The building was modeled using a single type of masonry, consistent with the one defined in the explanatory Circular of the NTC2018 [5], "Full brick masonry with lime mortar." The masonry parameters, as indicated in the same Circular, were used for modeling and are reported in *Tab. 1*, where: f_m is the compressive strength, f_b is the shear strength, f_{v0} is the shear strength of the masonry in the absence of compression, E is Young's modulus, G is the shear modulus, and finally, w is the specific weight of the masonry. The shear constitutive law of the masonry has been assumed as the minimum between two criteria: the one defined by Mohr-Coulomb and the one by Mann and Müller.

Tab. 1 Mechanical parameters of the considered masonry.

f_m [N/cm^2]	f_b [N/cm^2]	f_{v0} [N/mm^2]	E [N/mm^2]	G [N/mm^2]	w [kN/m^3]
260	7.6	13	1500	500	18

Additionally, the floors in the SUs are double timber floors, equipped with beams with a base of 10 cm, a height of 30 cm, and a spacing of 50 cm, and floorboards with a total thickness of 6 cm, as indicated in *Tab. 2*.

Tab. 2 Mechanical properties of the floor.

s [cm]	E_x [N/mm^2]	E_y [N/mm^2]	G [kN/m^2]
6	24000	12000	750

3 Modeling criteria for the in-plane response

The buildings in aggregate under study were modeled using the Equivalent Frame (EF) approach, as implemented in the Tremuri software [4]. The choice to model the IP response of the aggregate using the EF approach was guided by its intuitive nature and ease of implementation compared to the finite element model (FEM), which requires detailed discretization of the structure. This process can be particularly complex and time-consuming, especially for existing or irregular buildings. An important advantage of EF modeling using macro-elements is its computational efficiency, allowing for numerous Non-Linear Dynamic Analyses (NLDAs) to be performed in relatively short times.

The decision to use the Tremuri software is based on its phenomenological constitutive models, which allow for an accurate representation of the hysteretic response under various damage mechanisms. Tremuri has also been validated in numerous studies for performing many NLDAs [6], [7], [8]. This method considers masonry walls composed of vertical elements (piers) and horizontal elements (spandrels), connected by rigid areas (nodes) that are not subjected to damage. In the models, the non-linear response of the panels is described by a constitutive law based on a phenomenological approach and a piecewise-linear beam model (NLBEAM) proposed in [9]. The NLBEAM is characterized by a constitutive law describing the non-linear response up to very severe damage levels (DL, from 1 to 5) through the definition of a relationship between the drift value $\delta_{E,i}$ and the corresponding fraction of residual shear strength $\beta_{E,i}$ upon reaching the i-th DL, differentiated for bending and shear behavior for piers, and for spandrels. For further details on the parameters governing this response, please refer to [10], [11]. *Tab. 3* provides these values for the aggregate under study.

Tab. 3 Thresholds used for piers and spandrels parameters.

	PIERS								
	drift θ [%]			*residual strength β*		*hysteretic response*			
	DL3	DL4	DL5	DL3	DL4	c_1	c_2	c_3	c_4
SHEAR	0.47	0.73	0.94	0.6	0.2	0.8	0.8	0	/
FLEXURAL	0.6	0.9	1.2	1	0.85	0.9	0.8	0.6	0.5
	SPANDRELS								
	drift θ [%]			*residual strength β*		*hysteretic response*			
	DL3	DL4	DL5	DL3	DL4	c_1	c_2	c_3	c_4
	0*	1	1.5	0.6	0.6	0.2	0	0.3	0.8

* defined starting from drift corresponding to the yielding point of the element and assuming a ductility factor equal to 4.

Based on the analysis of the historical evolution of the aggregate, some SUs were selected and analyzed both in their aggregate configuration and as isolated units (*Fig. 3*).

In modeling the entire aggregate, particular attention was given to correctly representing the boundary conditions of each SU, as this can significantly affect the modal behavior and overall seismic response of the structure.

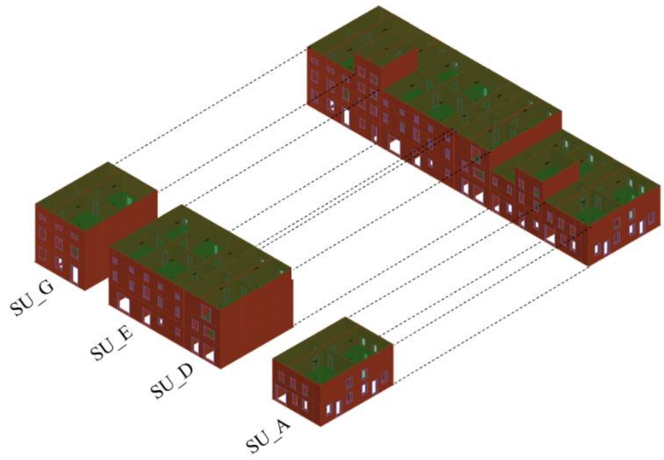

Fig. 3 3Muri model of the four analyzed SUs in both aggregated and isolated configurations.

The modeling of the interaction between adjacent SUs was differentiated depending on whether the adjacent SUs were structurally independent, with adjacent walls, or structurally dependent. In the latter case, the SUs were developed in different periods, filling the free spaces between them, or when a SU was built against an existing one, sharing the wall of the unit it leans on. This distinction is crucial for correctly representing the behavior of the SUs and their interaction within the aggregate. For this reason, a thorough analysis of the evolutionary phases of the aggregates under examination was conducted to understand the factors that led to the current structural configuration (*Fig. 2*).

In the literature, there are many studies examining the interaction between SUs [12], [13], [14], [15]. In cases where the SUs were found to be structurally independent, the approach proposed in [11] was chosen to simulate the possible transverse sliding between the SUs and their potential opening while avoiding element interpenetration (*Fig. 4 I.a* and *I.b*). Therefore, when the SUs are treated as independent, the modeling aimed to accurately represent the unilateral constraints at the joints, allowing detachment between the SUs while preventing interpenetration and including potential pounding effects. To achieve this, the SUs were modeled separately by introducing a finite-length gap equal to half the thickness of the two adjacent walls of the SUs. They were then connected using elastic truss elements and orthotropic membranes to regulate their interaction. The elastic truss elements, with a sectional area of 0.00164 m² and an elastic modulus E of 210000 MPa, were assigned null tensile behavior to allow for detachment while reproducing the stiffness of the masonry in contact. These elements were calibrated to represent the stiffness of the masonry involved in the interaction, considering the interstory height, wall thickness, and an interaction depth of approximately 1 meter per side. Additionally, orthotropic membranes were introduced with a thickness of 0.05 m, an elastic modulus of E = 39420 MPa, and a shear modulus of G = 13112 MPa. The shear stiffness of these membranes was derived from parametric analyses to realistically simulate transverse sliding between the SUs. A sensitivity analysis was conducted to fine-tune the shear stiffness of the membrane, ensuring that the relative sliding between the SUs remained limited, as observed experimentally. In reality, such sliding is generally restricted due to friction and interlocking effects. While pounding can lead to joint openings or localized crushing damage, significant transverse displacements between adjacent SUs are rarely observed. In the case where the SUs are structurally dependent, only the elastic truss elements with the aforementioned properties were introduced, in such a way as to allow separation between the SUs exclusively in the direction of the aggregate's development (*Fig. 4 II.a* and *II.b*).

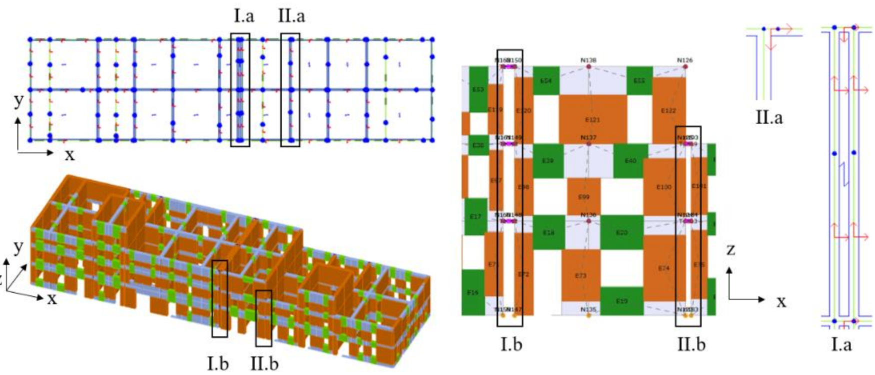

Fig. 4 Equivalent frame model of the aggregate. I.a and I.b: Plan and elevation details, respectively, between SU_D and SU_E, which are adjacent. II.a and II.b: Plan and elevation details, respectively, between SU_C and SU_D, where SU_C is leaning against SU_D.

To model this case study as realistically as possible, in addition to carefully defining the boundary conditions of the various SUs, all elastic connecting beams were appropriately calibrated to account for imperfect interlocking between walls and the resulting reduction in stiffness associated with the flange effect.

4 Numerical analyses and derivation of the fragility curves

To evaluate the aggregate effect concerning the IP response of the four SUs under study, and consequently derive their fragility curves, preliminary analyses were conducted. These included NLSAs and modal analyses to understand the dynamic behavior of both the aggregate and the individual SUs. The modal analyses allowed for a comparison of the modal shapes, periods, and participating masses of each SU. Two extreme cases have been considered: one where the SU is isolated and another where the SU is part of an aggregate but perfectly connected to adjacent SUs. The modal shapes corresponding to both configurations are shown in *Fig. 5*, and the corresponding vibration periods are shown in *Tab. 4*.

Based on the results of these analyses, the Rayleigh damping coefficients necessary for performing NLDAs were also derived (see *Tab. 5*). The value of initial damping is equal to 3 as usual for masonry building.

US_E was found to be the stiffest unit; therefore, it is expected to perform worse in the aggregate configuration, as it attracts more inertial forces in the dynamic response. Units A and G exhibit a second torsional mode, and the third bending mode in the Y direction of unit A also has a torsional component, likely due to the fact that one of its two perimeter walls in the Y direction has openings, while the other is solid (see *Fig. 1*).

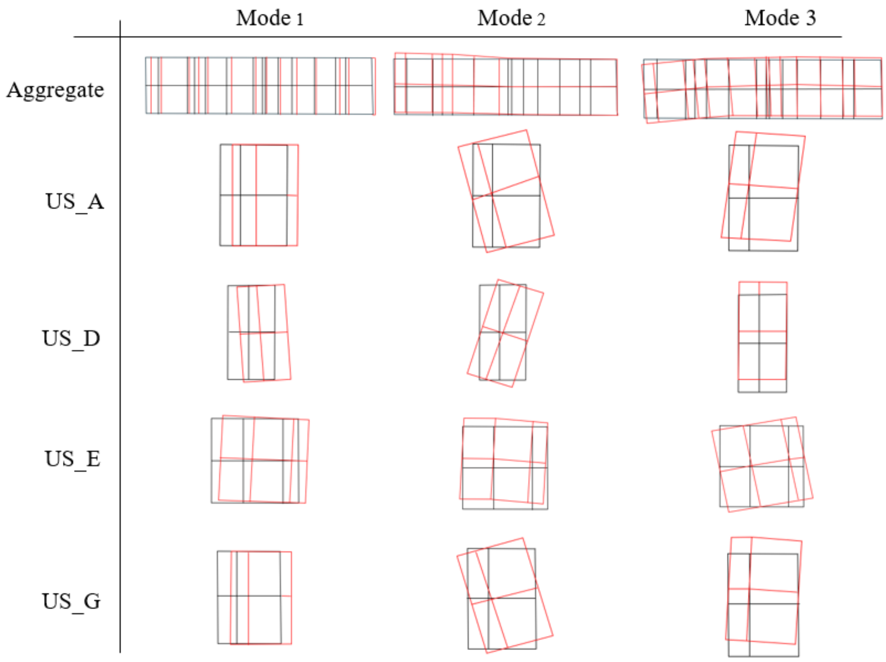

Fig. 5 Modal shapes of the SUs in the aggregate and isolated configurations.

Tab. 4 Vibration periods of the isolated SU and the entire aggregate.

	$T_{AGGREGATE}$ [S]	T_{SU_A} [S]	T_{SU_D} [S]	T_{SU_E} [S]	T_{SU_G} [S]
MODE 1	0.209	0.151	0.289	0.169	0.210
MODE 2	0.184	0.138	0.216	0.148	0.163
MODE 3	0.149	0.109	0.150	0.134	0.148

Tab. 5 Rayleigh coefficient α (mass proportional damping) and β (stiffness proportional damping) for the isolated and aggregated configurations.

	AGGREGATE	SU_A	SU_D	SU_E	SU_G
α	0.6010	0.8320	0.4350	0.7440	0.5980
β	0.0013	0.00096	0.00184	0.0011	0.0013

The actual behavior of the connections between the SUs, including the potential for detachment or partial interaction, is instead captured through non-linear analyses. The fragility curves for each structural configuration of every SU were obtained by conducting NLDAs using two sets of 135 accelerograms, each comprising two horizontal components, for stiff soils representative of the Italian seismic hazard. These accelerograms were selected within the 2019-2021 research agreement between the Civil Protection Department (DPC) and the Network of University Laboratories for Earthquake Engineering (ReLUIS) as part of the WP4 "Seismic Risk Maps - MARS" work package. The fragility curves were developed in terms of five damage levels (DLs) compatible with the damage classification provided by the EMS98 macroseismic scale [16]. The procedure used to obtain the fragility curves is based on monitoring the severity and spread of damage in the building through the cumulative rate of walls that reached a given DL [17]. The adopted approach considers both the contribution of piers and spandrels in defining the damage. The average value (μ) and the standard deviation (σ) of the Peak Ground Acceleration (PGA) of the input motions causing the achievements of each damage level was then calculated. These values were used to calibrate a lognormal distribution of the PGA causing the attainment of DLi. The probability of exceeding the different damage levels, pDLi, under a certain PGA can be consequently calculated as the probability that such PGA "exceeds" the calibrated lognormal distribution. The fragility curves, so defined, obtained for each SU are shown in *Fig. 4*.

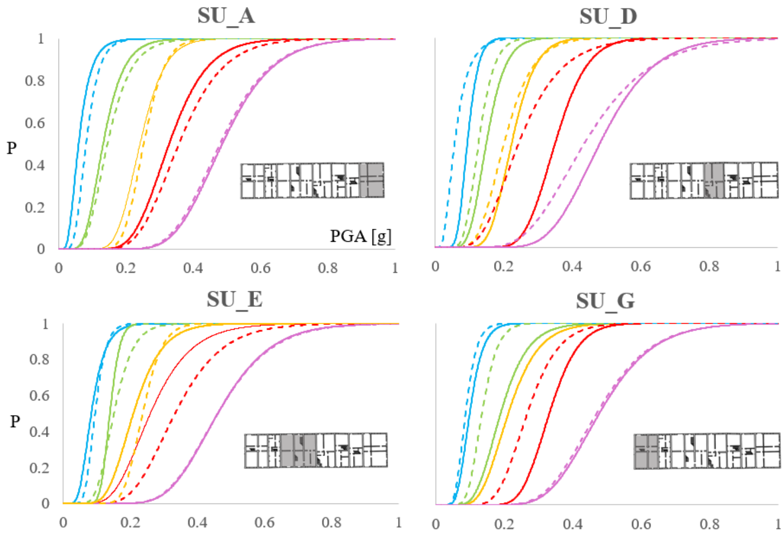

Fig. 6 For each analyzed SU, a comparison between the fragility curves obtained for the SU in isolated configuration and in aggregate configuration.

Tab. 6 summarizes the mean and standard deviation values, as well as the number of analyses falling within each DL for each SU in both structural configurations. What emerges from the fragility curves shown in *Fig. 6* is a highly case-specific behavior, highlighting the complexity of assessing the seismic vulnerability of aggregates. The units that benefit from the aggregate effect are the enclosed unit US_D and the end unit US_G, whereas the enclosed unit US_E and the end unit US_A exhibit lower vulnerability when in an isolated configuration.

To better understand the results emerging from the fragility curves, further investigations were conducted. In addition to modal analyses, uniform NLSAs were performed to assess the stiffness of the various SUs and verify the consistency of the different analyses. The NLSAs were carried out for each SU, considering both the isolated and aggregate configurations.

Tab. 6 Mean (μ) and standard deviation (σ) values of fragility curves for each DL for each SU analysed.

			DL1	DL2	DL3	DL4	DL5
SU_A	Isolated	μ	0.08	0.15	0.25	0.36	0.48
		σ	0.37	0.38	0.20	0.29	0.25
		n° analyses	50	74	10	18	82
	Aggregated	μ	0.06	0.13	0.24	0.33	0.49
		σ	0.48	0.36	0.24	0.27	0.24

		n° analyses	36	90	10	23	81
SU_D	ISOLATED	μ	0.06	0.13	0.20	0.24	0.44
		σ	0.53	0.28	0.32	0.39	0.34
		n° analyses	41	66	9	21	102
	AGGREGATED	μ	0.09	0.15	0.22	0.35	0.48
		σ	0.27	0.29	0.24	0.21	0.26
		n° analyses	40	59	14	21	81
SU_E	ISOLATED	μ	0.09	0.15	0.24	0.33	0.27
		σ	0.27	0.30	0.20	0.33	027
		n° analyses	38	57	11	14	92
	AGGREGATED	μ	0.08	0.14	0.21	0.26	0.46
		σ	0.39	0.17	0.32	0.36	0.27
		n° analyses	54	37	26	13	96
SU_G	ISOLATED	μ	0.09	0.13	0.21	0.27	0.47
		σ	0.29	0.26	0.33	0.27	0.27
		n° analyses	38	54	16	21	92
	AGGREGATED	μ	0.10	0.19	0.25	0.33	0.48
		σ	0.31	0.33	0.60	0.20	0.26
		n° analyses	70	49	4	15	85

Fig. 7 presents the pushover curves for the end units US_A and US_G, while *Fig. 8* shows the same curves for the enclosed units US_D and US_E. The pushover curves display, on the vertical axis, the base shear normalized by the mass of each SU, allowing for a better relative comparison of the resistance of the different units. The results indicate that the end unit US_A, in the Y direction, exhibits a limited elastic range in the aggregated configuration compared to the isolated one. Conversely, the end unit US_G is significantly less ductile in the isolated configuration, making it more vulnerable compared to the aggregated state. These observations are consistent with the findings from the fragility curves obtained through NLDAs (see *Fig. 6*). Regarding the enclosed unit US_E, the difference in response between the two configurations is more pronounced in the X direction than in the Y direction. This is consistent with the fact that, in the X direction, corresponding to the development axis of the aggregate, this unit is more affected by the aggregate effect.

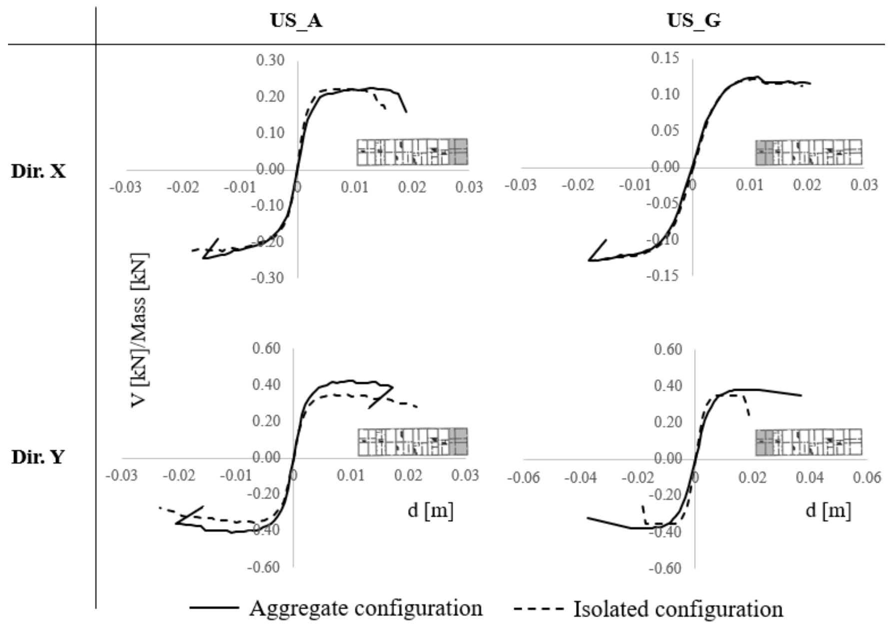

Fig. 7 Uniform pushover in X and Y direction for the end units SU_A and SU_G in the isolated and aggregate configuration.

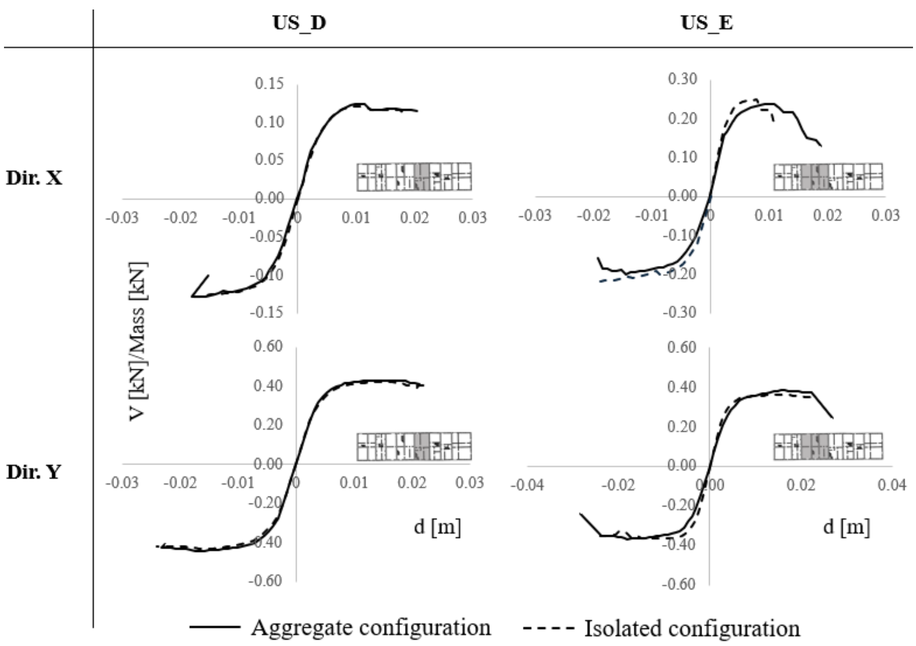

Fig. 8 Uniform pushover in X and Y direction for the enclosed units SU D and SU_E in the isolated and aggregate configuration.

5 Conclusions

The assessment of the seismic vulnerability of masonry buildings in aggregate is a complex challenge due to the interaction mechanisms between SUs. This study analyzed the IP seismic response of an aggregate through numerical modeling, considering both the entire aggregate and individual SUs in their isolated configurations. Using the EF approach with Tremuri software, modal analyses, NLSAs and NLDAs were performed to evaluate the effects of the aggregate on seismic response. The modeling was developed with the objective of accurately representing the structural behavior of the aggregate, with particular attention to the definition of the connections between different SUs. Specifically, imperfect interlocking between walls was considered by introducing calibrated connecting beams to reduce the stiffness associated with the flange effect.

Modal analyses provided an initial evaluation of the stiffness distribution, showing how the presence of the aggregate modifies modal periods and mode shapes. Subsequent pushover analyses further explored these interactions, allowing for a comparison of the stiffness and strength of different SUs in both isolated and aggregate conditions.

One of the key aspects of this study is the derivation of fragility curves through NLDAs, enabling a probabilistic assessment of seismic vulnerability across different structural configurations. These curves reveal a highly case-specific response, confirming the difficulty of defining general trends for masonry aggregates. Some SUs benefit from the aggregate effect, while others become more vulnerable in the aggregate configuration.

The results highlight the complexity of the aggregate phenomenon and the challenge of drawing general conclusions. Although the position of SUs within the aggregate is a relevant factor, it is not sufficient to unambiguously determine the beneficial or detrimental effects of aggregation. The seismic behavior of individual units is strongly influenced by their relative stiffness in comparison to adjacent units. The results of NLDAs are consistent with those of simpler analyses (modal and NLSAs), reinforcing the importance of an integrated evaluation to fully understand structural interactions. This complexity underscores the necessity of a detailed assessment of the connections between SUs for an accurate estimation of seismic vulnerability in historic masonry aggregates. In the future, further research should address the modeling uncertainties highlighted in this study, particularly those related to boundary condition representation and the variability of interaction mechanisms between adjacent SUs. In this regard, additional parametric analyses are recommended to calibrate connection elements and interface properties, so as to realistically simulate detachment, friction, and interlocking phenomena. Future sensitivity analyses should also explore alternative geometric configurations, varying both the position of individual SUs within the aggregate and the relative stiffness between the units. This would allow for a more systematic evaluation of how these parameters influence seismic vulnerability, helping to identify recurring patterns or critical thresholds beyond which the aggregate effect shifts from a protective constraint to a damage-amplifying factor. Another relevant aspect is the integration of out-of-plane (OOP) response, whose influence on SU vulnerability remains largely underexplored. The interaction between in-plane (IP) and OOP mechanisms represents a complex challenge that extends beyond aggregates to masonry structures in general. In the future, it would be useful to develop tools capable of identifying which mechanism activates first, in order to better understand the nature of structural interaction and its implications for seismic performance. Finally, parametric investigations based on simplified configurations could support the development of generalized predictive criteria.

6 Acknowledgements

The study presented in the article was developed within the research activities of the ReLUIS-DPC 2024-2026 research programs (WP4 and WP5), funded by the Department of Civil Protection.

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