

Article

A Seismic Assessment Approach for Historical Masonry Aggregates

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Abstract

The seismic assessment of historical masonry aggregates is particularly challenging as their present configuration typically reflects a complex construction history involving geometrical irregularities, heterogeneous structural systems, and mechanical interaction among adjacent buildings. In this context, an accurate reconstruction of the three-dimensional geometry of the aggregate is a fundamental step, since differences in height, wall configuration, floor levels, openings, and structural continuity may directly influence the global dynamic behavior. This study investigates the San Filippo Neri historical masonry aggregate through an integrated geometrical and structural modelling strategy. The geometry of the aggregate is reconstructed from the available architectural and survey documentation, including plans, sections, elevations, and photographic information, with particular attention to wall thickness variations, openings, horizontal structures, roofs, and masonry vaults. The reconstructed geometry then serves as the basis for a three-dimensional finite element model of the entire aggregate. Global modal analysis is then performed to identify the portions of the block which exhibit a distinguishable dynamic behavior. Two selected portions are subsequently analyzed independently, and their modal properties are compared with those obtained from the full aggregate model. Good agreement is found in terms of dominant vibration periods and mode shapes, supporting their interpretation as approximately independent structural units. A representative regular building located within one of these portions, Palazzo Jenca, is then modeled through the equivalent-frame approach and subjected to nonlinear static analysis. The N2 pushover procedure is used to derive a structure-specific behavior factor, which is subsequently employed in the linear dynamic assessment of the complete aggregate. The results highlight the close relationship between geometrical configuration and dynamic interaction within historical building aggregates. They further show that geometrical reconstruction and global modal behavior can jointly provide a mechanically informed basis for defining structural units in seismic assessment.



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Keywords: historical masonry; building aggregates; geometrical reconstruction; structural units; modal analysis; equivalent frame; pushover analysis; seismic assessment

1. Introduction

Historical masonry aggregates represent one of the most complex classes of existing structures for seismic assessment [1–6]. Unlike isolated buildings, they are generally the result of progressive urban development processes extending over long periods and involving additions, partial demolitions, elevation changes, modifications of openings,

replacement of floors, and local strengthening interventions. Therefore, their present structural configuration often differs substantially from the original arrangement of the individual buildings [7–9]. As a consequence, the seismic response of historical aggregates is strongly affected not only by the mechanical properties of the masonry, but also by the interaction between adjacent structural portions. Differences in building height, wall alignment, floor levels, structural continuity, and construction sequence can significantly alter the distribution of stiffness and mass and, consequently, the global dynamic behavior.

The first critical step in the seismic assessment of such structures lies in reconstructing and interpreting their actual geometry. Available historical documentation is often incomplete, so the current configuration must instead be inferred by combining architectural drawings, structural surveys, sections, elevations, photographic information, and direct on-site observation. In complex masonry aggregates, geometrical reconstruction should not be regarded merely as a preliminary modelling operation, but rather as part of the structural interpretation of the asset.

A second major issue concerns the identification of the structural units to be considered in the analysis. Conventional subdivision criteria are commonly based on the historical development of the aggregate, geometrical discontinuities, construction techniques, differences in masonry typology, floor levels, and visible structural joints. However, these criteria do not necessarily demonstrate whether the selected portions behave independently under seismic loading. For strongly irregular aggregates, this issue is particularly important because the direct application of nonlinear analyses to the complete structure may be computationally demanding. Simplified approaches such as the equivalent-frame method are effective for the nonlinear analysis of unreinforced masonry buildings, but their applicability is generally more straightforward for regular structural configurations. Complex building aggregates may therefore require a preliminary investigation aimed at determining whether some portions can be meaningfully analyzed independently [10,11]. In this framework, global modal analysis can provide valuable information on the actual degree of dynamic interaction between adjacent parts of the aggregate. If specific portions develop recognizable vibration modes within the response of the complete structure, and if these modal characteristics remain substantially unchanged after numerical isolation, the corresponding portions may reasonably be treated as approximately independent structural units [12–14].

The present study investigates this approach through the San Filippo Neri historical aggregate in L'Aquila. The work emphasizes the complexity of historical building clusters and the limited specificity of code provisions when dealing with irregular aggregate configurations. The proposed procedure consists of three main stages. First, the three-dimensional geometry of the complete block is reconstructed from the available architectural and structural information, preserving the main morphological irregularities and structural discontinuities. Second, a global finite element (FE) model is developed and used to investigate whether the reconstructed geometrical configuration gives rise to dynamically distinguishable structural portions. Third, selected portions are analyzed independently and a representative regular building is subjected to nonlinear equivalent-frame analysis.

The main objective is therefore to evaluate whether geometrical reconstruction and global dynamic behavior can be combined to provide a mechanically informed basis for identifying structural units in historical masonry aggregates.

2. Historical Masonry Aggregate

2.1. Case Study

The investigated case study is the San Filippo Neri historical masonry aggregate in the city center of L'Aquila. The aggregate is bounded by Via Sallustio to the north, Piazza San Filippo to the west, Piazza Machilone to the east, and Via Tre Marie to the south. Its overall dimensions are approximately 52 m in the north–south direction and 32 m in the east–west direction (Figure 1).



(a)



(b)

Figure 1. (a) Location; (b) west view of the San Filippo Neri historical masonry aggregate.

L'Aquila is located in the Central Apennines, one of the most seismically active regions of Italy, characterized by a long history of destructive earthquakes associated with the extensional tectonic regime of the Apennine chain. Historical seismic events affecting the area include the earthquakes of 1349, 1461, and 1703, which caused extensive damage to the city's building stock. More recently, the earthquake of 6 April 2009 (Mw 6.3) severely affected the city and its historical built heritage, highlighting the high seismic vulnerability of traditional unreinforced masonry constructions. The observed damage was

influenced not only by the intensity of the seismic excitation but also by the construction characteristics of the buildings, including poor masonry quality, irregular structural configurations, differences in building heights, and interactions between adjacent structural units. In the San Filippo Neri aggregate, the 2009 earthquake caused widespread cracking in load-bearing walls, localized damage to horizontal structures, and partial roof collapses. Although most masonry panels exhibited relatively limited cracking, more severe damage was concentrated in specific portions of the aggregate, particularly along its southern side. These observations emphasize the importance of considering both the geometrical complexity and the structural interaction between adjacent buildings when assessing the seismic performance of historical masonry aggregates.

The current configuration of the block is the result of a complex construction history that is only partially documented. The development of the aggregate was therefore interpreted through the available historical information, façade analysis, and construction characteristics. The oldest portion is believed to be located along Via Tre Marie, whereas Palazzo Jenca represents a later construction phase that is architecturally more clearly identifiable. The aggregate is highly irregular both in plan and elevation. It includes buildings with different heights, floor levels, wall arrangements, horizontal structural systems, and construction techniques. The vertical configuration of the aggregate is characterized by two higher building portions, located along its northern and southern sides and comprising three storeys, connected by lower central buildings generally consisting of two storeys. The irregularity of the aggregate is further accentuated by the ground slope, which produces an elevation difference of approximately 4 m between opposite sides of the block. According to the geometrical survey, the ground-floor elevations range from approximately 710.10 to 715.60 m a.s.l., while the first-floor levels range from 714.10 to 718.70 m a.s.l. The second-floor elevations vary between 717.65 and 721.30 m a.s.l., and the third-floor levels between 720.85 and 724.65 m a.s.l. These differences reflect the heterogeneous development of the aggregate and result in discontinuities between the horizontal structural systems of adjacent buildings, which were explicitly considered in the three-dimensional numerical reconstruction. These characteristics make the direct identification of independent structural units uncertain when based exclusively on historical or geometric interpretation. The 2009 L'Aquila earthquake [15] damaged both vertical and horizontal structures, and parts of the roofs collapsed locally. Cracking spread throughout the bearing walls and exposed the poor quality of the multi-leaf masonry. Most wall panels showed only superficial cracks. The more severe cracks were limited to a few panels, mainly in the southern part of the block (Figure 2).



Figure 2. Wall panels with severe cracks.

2.2. Survey Information and Reconstruction Strategy

The geometrical reconstruction of the block constituted a fundamental stage of the study. A reliable structural model of historical masonry aggregates cannot be defined without first resolving the complex relationships among walls, openings, floors, roofs, and vaulted structures. The three-dimensional geometry was reconstructed by combining the available drawings, primarily consisting of plans, sections, and elevations, with the results of in-situ inspections aimed to identify the main load-bearing walls, wall thicknesses, openings, structural levels, differences in elevation among adjacent buildings, and the configuration of horizontal structural systems (Figures 3 and 4).

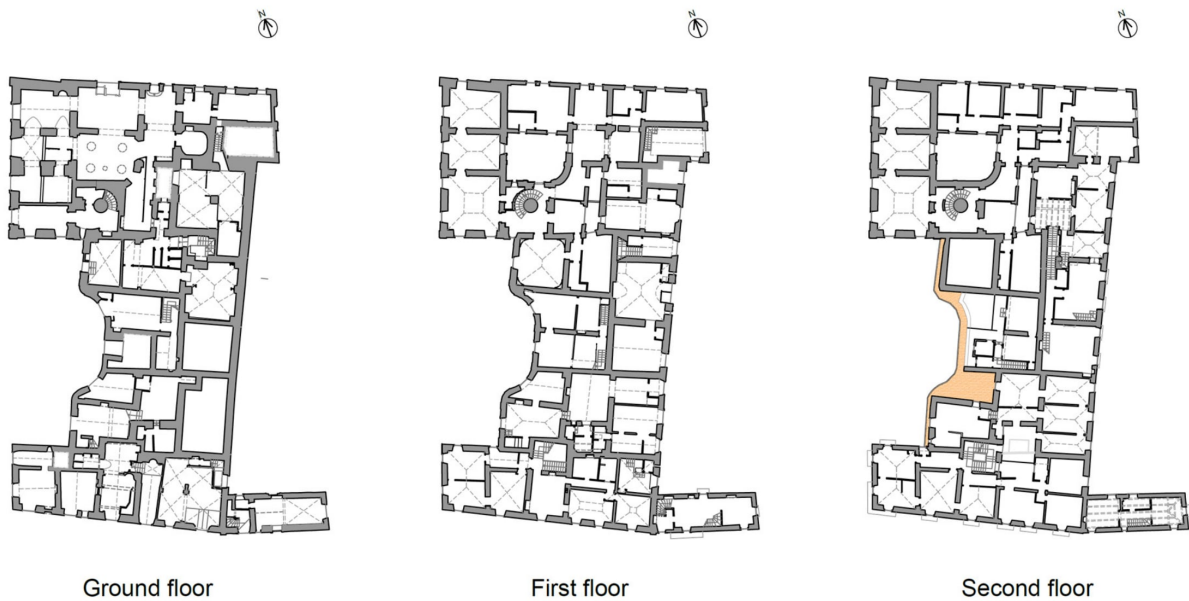


Figure 3. Ground, first and second floors.



Figure 4. Third floor and roof view.

The comparison among the different levels highlights the significant variation in the structural layout along the height of the aggregate. Walls are not always vertically aligned, room configurations change from floor to floor, and different portions of the aggregate

terminate at different elevations. Representative sections and elevations were therefore used to reconstruct the vertical organization of the complex (Figures 5–8).



Figure 5. South elevation.



Figure 6. West elevation.



Figure 7. North elevation.



Figure 8. East elevation.

The geometrical model was conceived to preserve those irregularities that could have a significant effect on the stiffness distribution and on the mechanical interaction between adjacent buildings. The available two-dimensional architectural drawings were transferred into a three-dimensional environment and used as the basis for the geometrical model. The modeling procedure relied on the superposition of floor plans and the definition of a consistent idealized layout of the walls suitable for the finite element structural modelling, which was identified in plan by the midplane lines of the walls. A particularly relevant step was the alignment of the architectural plans with the idealized structural representation (Figure 9).



Figure 9. Superposition of architectural floor plans and structural layout adopted for the geometrical reconstruction.

Changes in wall thickness, material properties, and floor intersections were introduced through horizontal and vertical subdivisions of the wall surfaces. Openings were also explicitly represented, since their size and position directly affect the stiffness and force-transfer mechanisms of masonry walls (Figure 10).

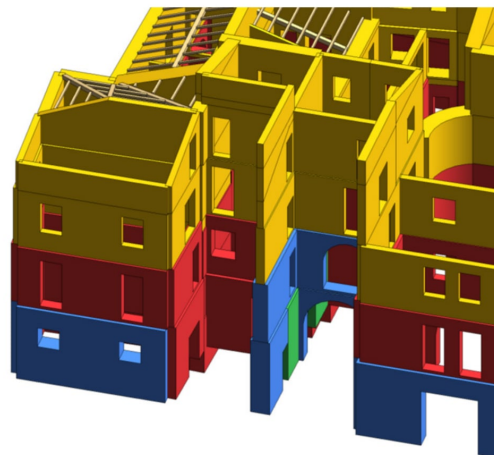


Figure 10. Representation of wall subdivisions, thickness variations, and openings within the three-dimensional model of a portion of the aggregate.

Therefore, the reconstruction process aimed not only to reproduce the visible architectural configuration but also to generate a geometrically coherent basis for subsequent structural analysis.

The horizontal structures are highly heterogeneous and include different construction typologies. Their representation was therefore adapted to the available information and to their structural significance. The timber roof was reconstructed according to the actual arrangement of the timber members using plans, sections, and photographic documentation (Figure 11).

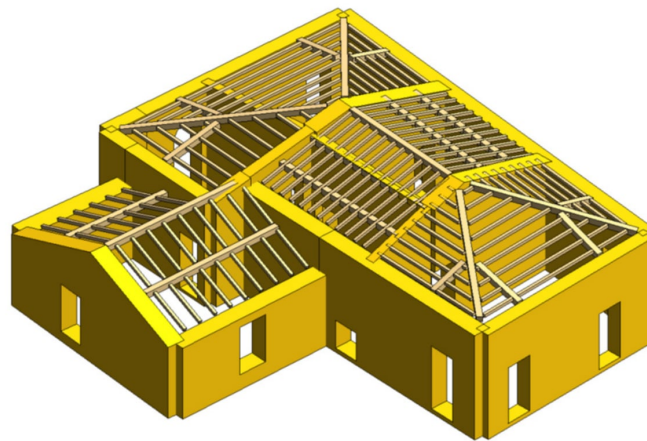


Figure 11. Reconstruction of the timber roof system.

The presence of different floor systems was also considered in the structural interpretation of the aggregate. Their position and orientation influence load transfer and in-plane stiffness and therefore contribute to the overall dynamic behavior.

The masonry vaults represented one of the most critical components of the geometrical reconstruction [16,17]. Their curved geometry could not be transferred directly through the adopted finite element modelling environment. Therefore, a dedicated reconstruction procedure was required. Vault surfaces were generated separately and manually discretized. The characteristic mesh size adopted for the vaults was approximately 30–40 cm, consistent with the discretization of the surrounding structural elements. Particular attention was devoted to the connection between vaults and supporting masonry walls, since geometrical incompatibilities at these interfaces could lead to discontinuities in the numerical model (Figure 12).

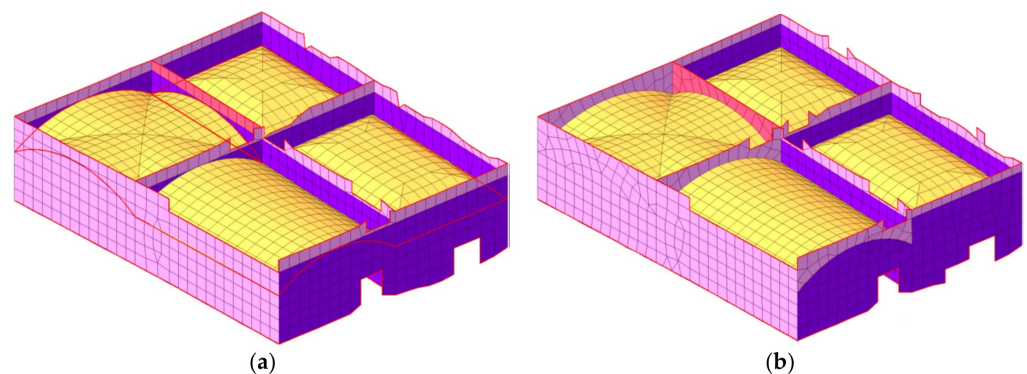


Figure 12. (a) Reconstruction of vaulted structures; (b) adjustment of the wall–vault interface.

The main difficulties encountered in modelling the masonry vaults were associated with the transfer of their complex geometries into the adopted FEM environment, rather than with their initial three-dimensional reconstruction. Although the vaulted surfaces could be

geometrically represented, their direct conversion into suitable shell finite elements was not supported by the adopted modelling workflow. Therefore, the vaults were reconstructed separately and manually discretized, adopting a characteristic element size of approximately 30–40 cm. Additional adjustments were required at the interfaces between the vaults and the supporting masonry walls to ensure mesh compatibility and avoid numerical discontinuities. These difficulties were therefore mainly related to the geometry-to-FEM conversion procedure and the need to obtain a consistent numerical discretization, rather than to an intrinsic inability of the FE software to represent vaulted structures [18].

Once the three-dimensional reconstruction was finished, the geometry was converted into a structural model suitable for finite element analysis. Midas Gen V1.2 2025 software [19] was used to generate the FE models and perform all the analyses. This transition required additional checks aimed at ensuring continuity and compatibility of the generated mesh. Local misalignments, missed connections, and unintended discontinuities around openings were corrected before the final model was transferred to the structural analysis environment (Figure 13).

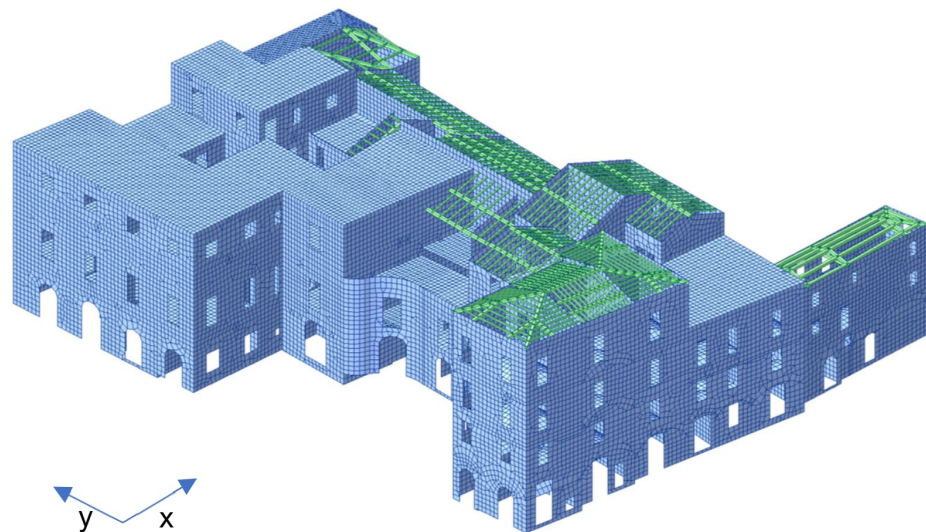


Figure 13. Final three-dimensional finite element model obtained from the reconstructed geometry.

The resulting model contains 75,062 elements in total, 74,275 of which are plate elements and 787 of which are beam elements. The final mesh of the model represents a compromise between architectural accuracy and structural modelling requirements, especially in terms of computational cost. The modeling process was intended to preserve the essential morphological features for reproducing the aggregate's global response while ensuring adequate numerical accuracy.

3. Material Characterization

The investigated aggregate is characterized by significant material heterogeneity, reflecting the various construction phases and building technologies used over time. Several masonry typologies were identified during the survey. The material properties were defined consistently with the masonry classification adopted in the geometrical reconstruction. The main mechanical parameters of the various materials used in the FE numerical model are reported in Table 1. In particular, the mechanical characterization of the four masonry typologies was defined in terms of the mean compressive strength (f_m), Young's modulus (E), and shear modulus (G). The reference values were derived in accordance with the Italian Building Code [20] and the associated Commentary [21], considering a Limited

Knowledge Level (KL1) and the corresponding confidence factor ($FC = 1.35$). The tensile strengths were assumed to be $0.1 f_m$.

For the FE analyses, the mechanical parameters were reduced using a safety factor equal to 4.05 for static analysis ($\gamma_m = 3.0$ and $FC = 1.35$), 2.70 for linear dynamic analysis ($\gamma_m = 2.0$ and $FC = 1.35$), and 1.35 for nonlinear analysis ($FC = 1.35$ only). A unit weight of 20 kN/m^3 was adopted for all four masonry typologies.

In the FE, the mechanical properties were assigned to the corresponding structural regions according to the material mapping derived from the survey. This allowed the global model to account for both geometrical irregularity and the spatial variation in stiffness and strength associated with the different masonry typologies.

Table 1. Material mechanical properties.

		f_m [N/mm ²]	E [N/mm ²]	G [N/mm ²]
Masonry Type 1	Static	0.395	243	81
	Linear dynamic	0.593	364	122
	Nonlinear analysis	1.185	729	243
Masonry Type 2	Static	0.222	193	53
	Linear dynamic	0.333	290	97
	Nonlinear analysis	0.667	580	159
Masonry Type 3	Static	0.740	731	183
	Linear dynamic	1.110	1096	274
	Nonlinear analysis	2.222	2193	548
Masonry Type 4	Static	0.593	370	123
	Linear dynamic	0.889	556	185
	Nonlinear analysis	1.779	1111	370

4. Global Finite Element Model

The complete three-dimensional FE model was used to investigate the global response of the aggregate. The primary objective of the model was to reproduce the overall stiffness and mass distribution and the interaction among the different structural portions rather than to simulate local nonlinear failure mechanisms. The model incorporates the main masonry walls, floor levels, horizontal structures, roofs, vaults, and geometric discontinuities identified during the reconstruction phase. The global FE model shown in previous Figure 13 was therefore adopted as the reference configuration for the subsequent modal analyses. Although the geometrical reconstruction provides a three-dimensional representation of the masonry volumes, the global FE model was developed primarily using shell elements, with the corresponding wall thicknesses assigned to the structural elements as a property. This modelling strategy was selected to reproduce the overall stiffness and mass distribution of the aggregate while maintaining a manageable computational effort. In principle, the same geometrical information could be used to develop a three-dimensional solid FE model through a dedicated volumetric discretization. However, such an approach would involve a substantially larger numerical model and was not considered necessary for the objectives of the present study, which mainly concerns the global dynamic response and the identification of structural portions with distinguishable modal behavior.

The particular configuration of the aggregate, which is characterized by two relatively high portions that are separated by a lower intermediate one, results in a specific dynamic behavior. The analyses showed that the two high portions exhibit a greater degree of freedom and develop recognizable modal responses, whereas the intermediate portion was constrained by the adjacent structures. The geometry of the aggregate was consequently identified as an

important factor in the definition of structural units. For the following analyses, the three main portions were identified as Portion 1, corresponding to the over-raised southern part of the aggregate, Portion 2, representing the lower central part constrained between the adjacent buildings, and Portion 3, corresponding to the northern part of the aggregate (Figure 14).

The geometrical reconstruction was not only used to generate the FE model, but also to preserve those morphological discontinuities that could influence dynamic interaction. In the proposed approach, geometrical reconstruction and dynamic identification are therefore directly interconnected: the reconstructed geometry provides the physical representation of the structural discontinuities, while modal analysis verifies whether these discontinuities actually result in distinguishable dynamic behavior.

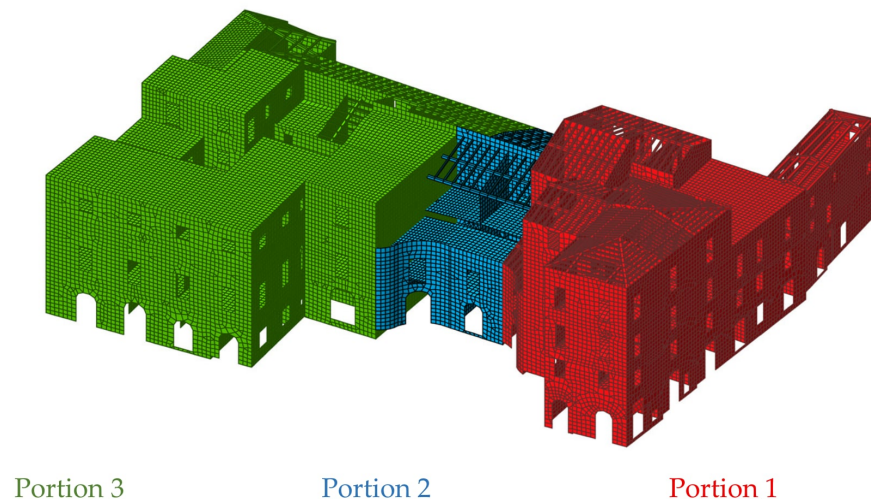


Figure 14. Subdivision of the aggregate in 3 portions.

4.1. Dynamic Behavior of the Full Aggregate

The modal analysis of the global model revealed that the aggregate does not behave as a dynamically homogeneous structure. The three main portions, previously identified based on the geometry, can be clearly distinguished also from the modal response. The two external portions, i.e., Portion 1 and Portion 3, exhibit comparatively large displacements and clearly recognizable translational modes. In contrast, the intermediate Portion 2 appears strongly constrained by the adjacent structures. For Portions 1 and 3, the analysis identifies dominant modes in both principal horizontal directions, developing a more clearly localized dynamic behavior, whereas Portion 2 does not show similarly distinct modal participation (Figure 15).

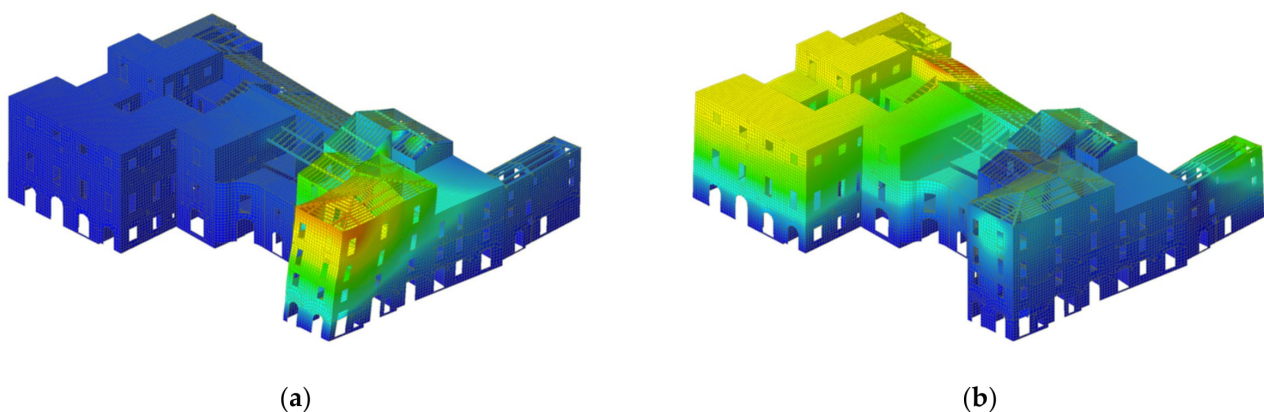


Figure 15. Representative global vibration modes: (a) Mode 3; (b) Mode 9.

The global geometry therefore appears to influence the effective degree of interaction between adjacent buildings. This observation supports the use of modal behavior as an additional criterion for structural-unit identification. However, localized vibration modes alone are not sufficient to demonstrate structural independence. For this reason, the selected portions were subsequently isolated and analyzed independently.

4.2. Isolated Portions

Portion 1 was extracted from the global model and analyzed independently. The dominant vibration mode shapes (Modes 3 and 4) obtained for the isolated structure were compared with those involving the same portion in the full model (Figure 16).

The isolated model exhibits dominant translational modes closely corresponding to those obtained within the complete aggregate. The reduction in model size is also substantial. The complete model contains 75,062 finite elements, whereas Portion 1 involves 29,794 elements, with a strong reduction in the computational cost.

The close correspondence between the dominant mode shapes and associated vibration periods (Table 2) indicates that the principal dynamic characteristics of Portion 1 are substantially preserved after isolation.

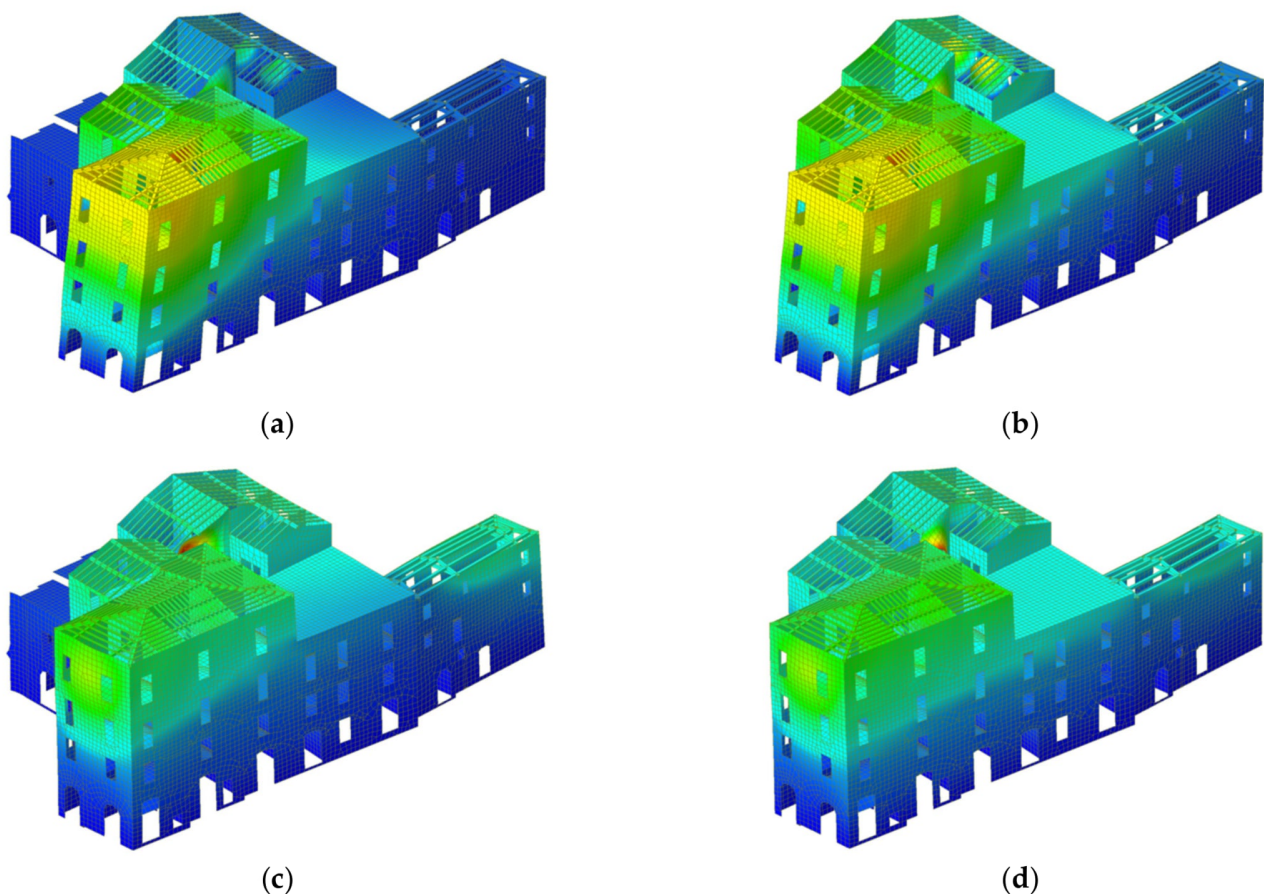
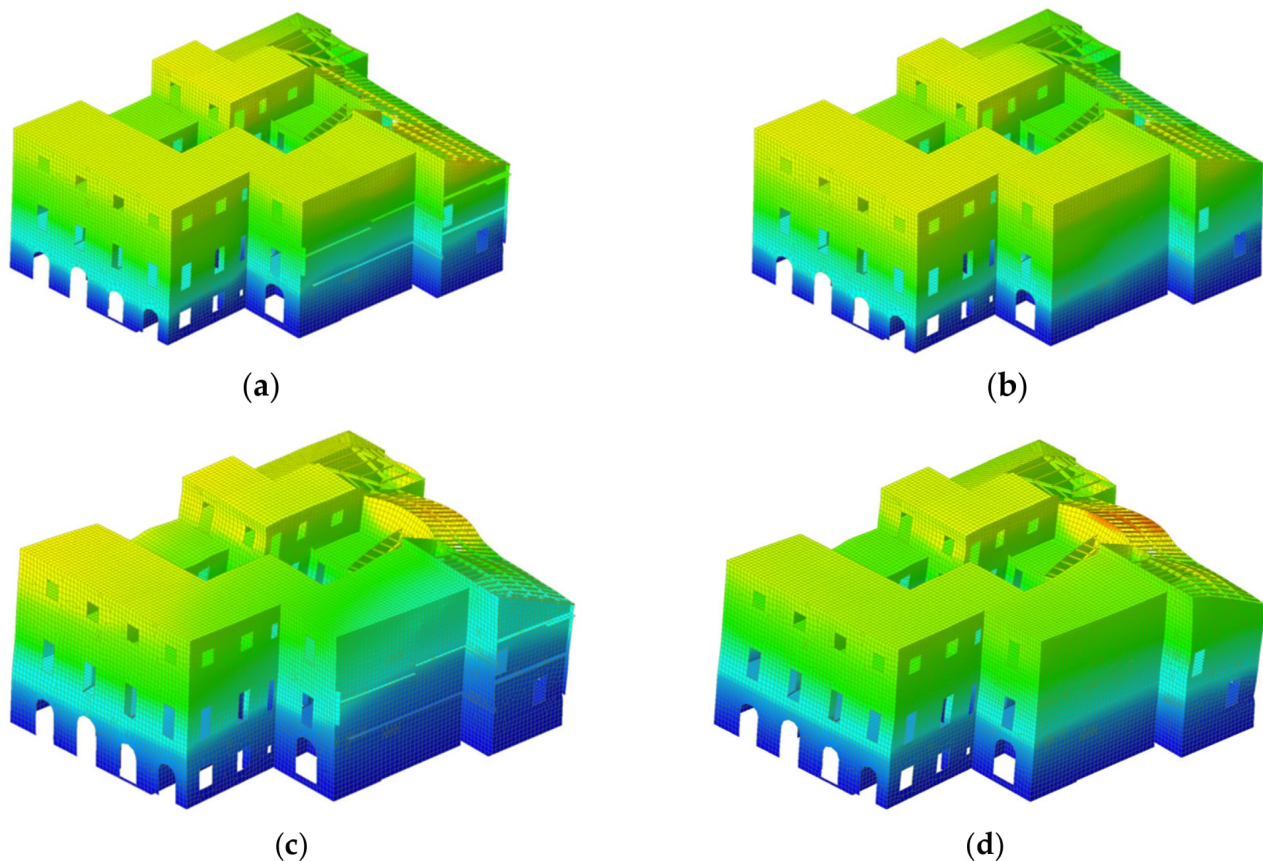


Figure 16. Comparison between vibration mode shapes of Portion 1: (a) global and (b) isolated Mode 3; (c) global; (d) isolated Mode 4.

The same procedure was applied to Portion 3. Figure 17 reports the comparison between global and isolated vibration mode shapes of Portion 3 considering Mode 2 and Mode 3 for the isolated model, and Mode 8 and Mode 9 for the global one.

Table 2. Vibration periods of dominant mode shapes for the global and isolated models and relative percentage difference.

	Global		Isolated		ΔT (%)
	Mode	Period (s)	Mode	Period (s)	
Portion 1	3	0.5561	3	0.5783	0.71
	4	0.4632	4	0.4665	
Portion 3	8	0.4064	2	0.4153	3.99
	9	0.3948	3	0.3991	

**Figure 17.** Comparison between vibration mode shapes of Portion 3: (a) global Mode 8 and (b) isolated Mode 2; (c) global Mode 9 and (d) isolated Mode 3.

Also in this case, the comparison shows a close correspondence between the dominant mode shapes and vibration periods. The number of elements is reduced from 75,062 in the global model to 40,992 in the isolated representation.

The results support the interpretation of Portions 1 and 3 as approximately independent structural units for the dominant dynamic response considered in the study.

Conversely, the same assumption cannot be made for Portion 2, whose geometry and location result in a stronger interaction with the surrounding structures.

To provide a quantitative basis for identifying approximately independent structural units, the percentage relative difference between the vibration periods of corresponding modes in the global and isolated models was evaluated as $\Delta T = (|T_{\text{isolated}} - T_{\text{global}}| / T_{\text{global}}) \times 100$. An indicative threshold of 5% was adopted as an operational criterion to assess the preservation of the dominant dynamic characteristics after numerical isolation. As reported in Table 2, the relative differences range from 0.71% to 3.99% for Portions 1 and 3, indicating a limited influence of the surrounding structures on the investigated vibration modes.

This quantitative agreement, together with the correspondence of the associated mode shapes shown in Figures 16 and 17, supports the assumption of approximate dynamic independence for the dominant modes considered. Nevertheless, this criterion does not establish complete structural independence, since interactions between adjacent buildings may still affect higher modes and the nonlinear seismic response. Therefore, the proposed subdivision should be regarded as a mechanically informed approximation rather than an indication of complete dynamic decoupling.

5. Equivalent-Frame Modelling of Palazzo Jenca

Palazzo Jenca, located within Portion 3, was selected for nonlinear analysis because of its architectural relevance and comparatively regular structural configuration. The in-plane behavior was studied using the equivalent-frame approach. Out-of-plane mechanism was not considered, since the building was provided in the past with appropriate strengthening systems to improve the out-of-plane behavior (i.e., tie-rods). The model was defined wall by wall, following the approach proposed in [22,23]. The equivalent-frame model was developed independently from the global shell FE model, using the reconstructed building geometry as a common reference. Each masonry wall was examined individually to identify the structural arrangement of piers, spandrels, and rigid nodes, considering the actual distribution of openings and wall dimensions. The resulting planar frames were subsequently assembled into a three-dimensional structural model. Therefore, the equivalent-frame representation was not obtained through an automatic conversion of a solid or shell FE model, but through a dedicated structural idealization of the building geometry. The final model consists of six planar wall frames assembled in three dimensions and includes 300 elements and 98 cross-sections (Figure 18).

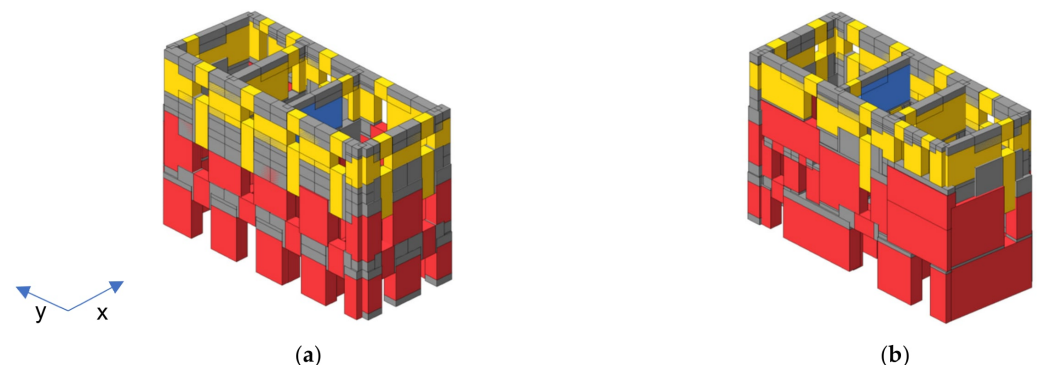


Figure 18. Palazzo Jenca equivalent frame model: (a) SW view, (b) NE view.

The equivalent-frame model provides a significantly simplified representation compared with the global plate model and is therefore suitable for nonlinear static analysis [24]. Nonlinear static analyses were performed along both principal horizontal directions X and Y (see Figure 18), considering positive and negative loading orientations. Both modal and mass-proportional load distributions were adopted, resulting in eight pushover configurations. The resulting capacity curves are shown in Figure 19.

The N2 procedure was used to determine the corresponding seismic demand and behavior factor. The behavior factors obtained from the eight analyses were relatively homogeneous (Table 3), and the minimum value of $q = 1.91$ was adopted.

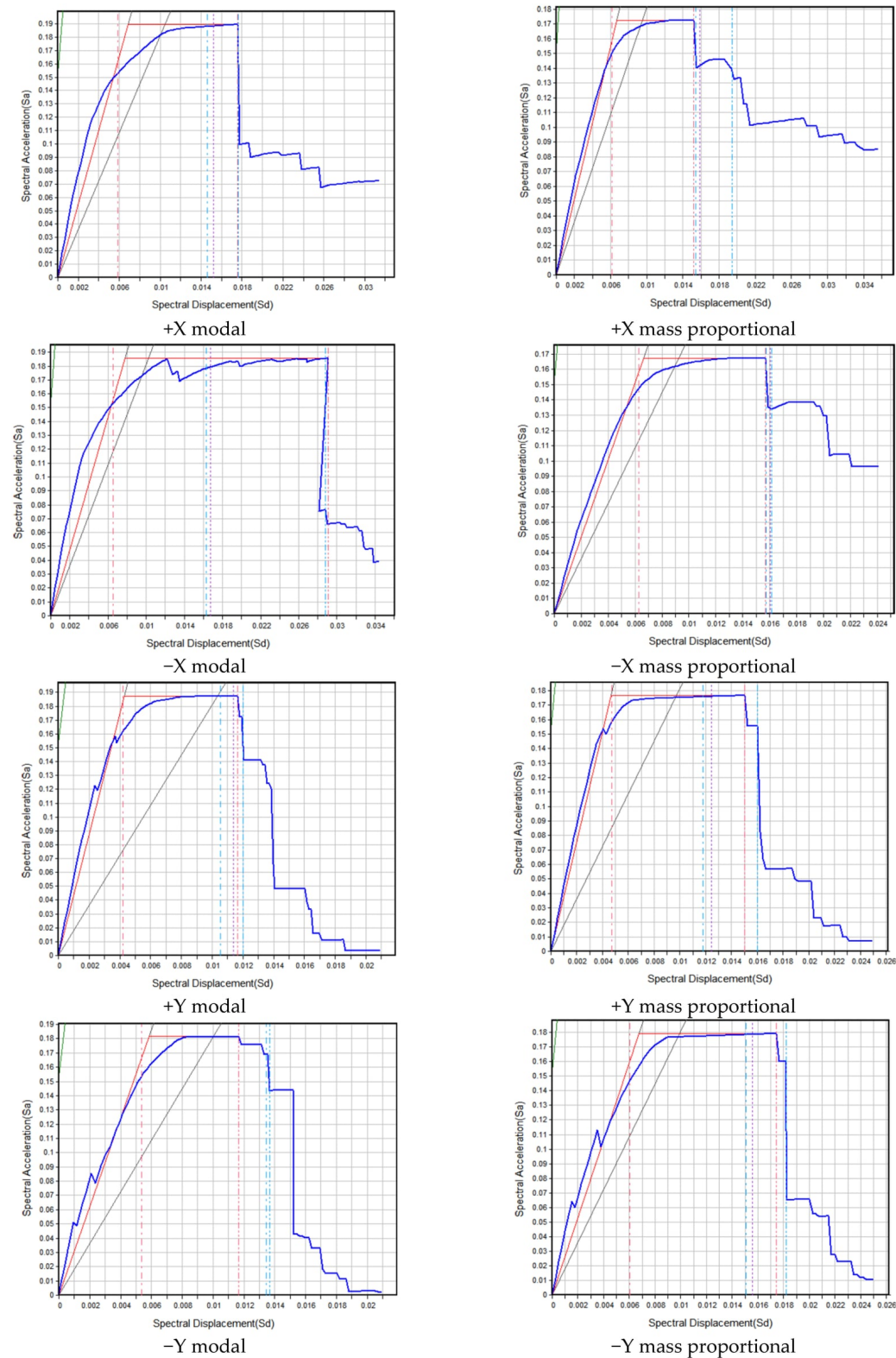


Figure 19. Capacity curves for modal and mass proportional lateral load distributions.

Table 3. Behavior factors for the considered load profiles (m = modal, mp = mass proportional).

Load Profile	+X (m)	−X (m)	+Y (m)	−Y (m)	+X (mp)	−X (mp)	+Y (mp)	−Y (mp)
q	1.91	1.95	1.94	2.00	2.10	2.17	2.05	2.02

This value was compared with the values given by the two most recent versions of the Italian building code. According to the current code [20,21], the simplified behavior factor q resulted equal to 1.89. The previous provisions for existing masonry buildings [25,26] give instead a value of $q = 2.25$. The value from the nonlinear analysis agrees very closely with the NTC 2018 estimate (a difference of about 1%). The 2009 value is about 18% higher. This difference comes mainly from how the two codes treat irregularity. The 2009 formulation reduces q only for irregularity in elevation. The NTC 2018 formulation penalizes irregularity in plan and in elevation separately. For the analyzed building, the combined reduction reproduces the redistribution capacity found in the pushover analysis well.

This result supports the more conservative approach of the current code for irregular URM buildings. It also shows that the earlier provisions may overestimate the dissipative capacity of buildings with significant irregularity in plan.

6. Linear Dynamic Assessment of the Full Aggregate

The behavior factor identified from the nonlinear analysis of Palazzo Jenca was used in the linear dynamic analysis of the full aggregate FE model. Figure 20 reports the horizontal and vertical response spectra adopted for the global linear dynamic analysis. The severe damage limit state design spectra were defined according to the Italian Standards considering the following input data: geographic coordinates (13.39467° E, 42.34986° N), soil category B, topography category T1, horizontal ground acceleration $a_g = 0.261$ g at bedrock, scale factor 0.65.

The resulting stress distributions were evaluated throughout the full aggregate, considering the seismic load combination defined according to [20]. As examples, Figures 21 and 22 shows the minimum and maximum principal stresses for one of the load combinations with X direction as the dominant seismic one. The stress distributions obtained from the global linear dynamic analysis provide further insight into the seismic response of the aggregate. While the minimum principal stresses generally remain within the admissible compressive range, the maximum principal stresses indicate localized regions in which the tensile strength of masonry is exceeded, particularly within Portion 1.

However, the principal-stress criterion alone provides a conservative assessment for masonry, as it does not account for the beneficial effect of axial compression on the shear resistance of masonry panels. A more detailed assessment of the most critical wall (i.e., south façade of portion 1) considering the interaction between axial and shear stresses showed that the piers subjected to compressive axial forces remained within the resistance domain, whereas most spandrel panels, characterized by negligible or tensile axial stresses, did not satisfy the adopted resistance criterion.

These results highlight the role of axial compression in enhancing the shear resistance of masonry and show that the global stress analysis is particularly effective for identifying critical regions, although local verification of individual masonry panels is required for a more reliable assessment of their actual safety conditions.

A comparison between the numerical results and the damage observed following the 2009 L'Aquila earthquake provides further insight into the reliability of the adopted modelling approach. As documented by the post-earthquake survey (Figure 2), the most severe cracking was mainly concentrated in the southern part of the aggregate, corresponding to Portion 1. This damage distribution is qualitatively consistent with the results of the global linear dynamic analysis, which identifies localized exceedances of masonry tensile strength predominantly within the same portion (Figures 21 and 22). Furthermore, the detailed assessment of the southern façade of Portion 1 highlights the vulnerability of several spandrel panels, whereas the masonry piers subjected to compressive axial forces

generally remain within their resistance domain. These findings support the ability of the numerical model to identify the portions of the aggregate most susceptible to seismic damage. However, the comparison represents a qualitative consistency check rather than a quantitative validation, since the linear elastic formulation does not explicitly simulate cracking, progressive stiffness degradation, or the development of nonlinear failure mechanisms. Nevertheless, the correspondence between the observed damage concentration and the numerically identified critical regions provides supporting evidence for the proposed assessment approach.

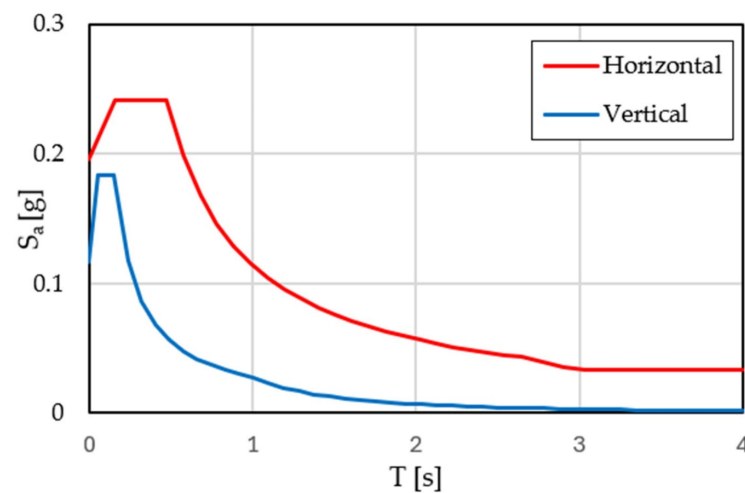


Figure 20. Horizontal and vertical response spectra adopted for the global linear dynamic analysis.

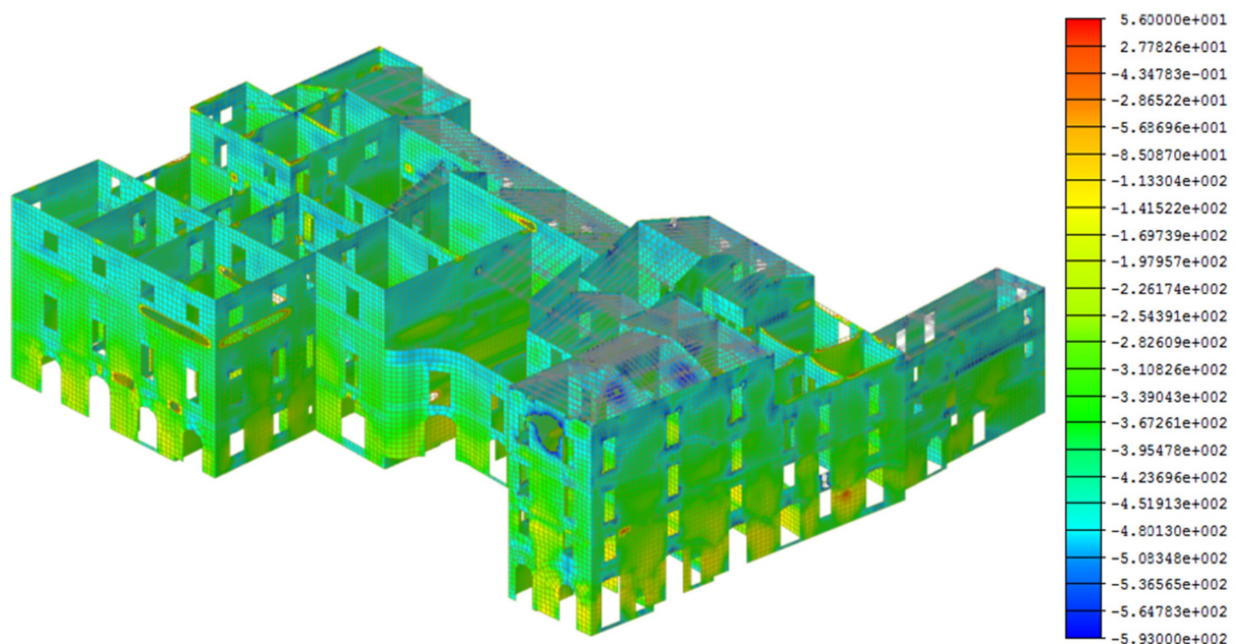


Figure 21. Minimum principal stress obtained considering one of the load combinations with X direction as the dominant seismic one (kN/m^2). The non-colored areas have stresses higher than the maximum tensile/compressive strength.

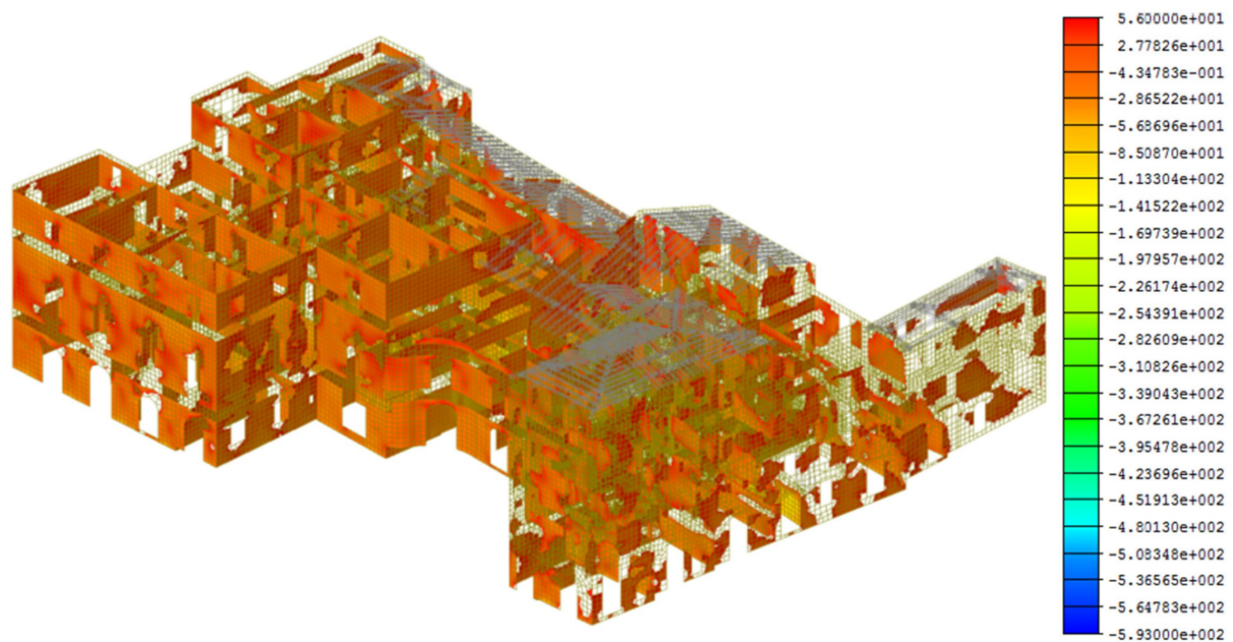


Figure 22. Maximum principal stresses obtained considering one of the load combinations with X direction as the dominant seismic one (kN/m^2). The non-colored areas have stresses higher than the maximum tensile/compressive strength.

7. Discussion

The results highlight the importance of considering geometrical reconstruction, structural interaction, and dynamic behavior as interconnected aspects of the seismic assessment of historical masonry aggregates.

The first relevant outcome concerns the role of geometry. The three-dimensional reconstruction shows that the aggregate is characterized by significant variations in building height, structural layout, and mutual constraint. These morphological characteristics are directly reflected in the dynamic response. Portions 1 and 3, which are geometrically higher and less restrained, develop recognizable vibration modes within the complete aggregate. In contrast, Portion 2 shows stronger interaction with its neighboring structures. The study therefore confirms that the overall geometry of the aggregate should not be regarded solely as a modelling input. It represents a structural parameter that may control whether individual portions can develop a distinguishable dynamic response.

A second important result concerns the validation of the structural subdivision. The dominant vibration mode shapes and vibration periods of Portions 1 and 3 remain substantially consistent after numerical isolation. This suggests that comparison between global and isolated modal behavior can provide a useful check of an assumed structural-unit subdivision. However, a possible limitation concerns the representation of the mechanical connections between adjacent buildings. In the present global FEM model, continuity was assumed at the interfaces between the different structural portions, without explicitly modelling construction joints or connections with reduced stiffness. Although this assumption allows the overall interaction within the aggregate to be investigated, it may not fully reproduce the actual mechanical behavior of interfaces characterized by partial connectivity or discontinuities. Introducing interface elements with variable stiffness would allow the influence of different connection conditions on the modal properties and dynamic interaction to be assessed. Such an investigation would require more detailed information on the construction characteristics of the interfaces and a dedicated parametric study. Therefore, the approximate dynamic independence identified for Portions 1 and 3 should be interpreted within the assumptions of the adopted continuous model, while the

sensitivity of these results to different interface conditions remains an important topic for future research.

A third aspect concerns computational efficiency. The isolated models involve substantially fewer finite elements than the complete aggregate. However, model reduction should not be considered an objective in itself. It is acceptable only when the resulting structural unit preserves the dominant response observed in the global model. The nonlinear assessment of Palazzo Jenca demonstrates a possible application of this strategy. Once a sufficiently regular and dynamically representative portion has been identified, the equivalent-frame method can be used to investigate nonlinear response with reduced computational cost.

Despite the encouraging results obtained for the investigated case study, some limitations of the proposed approach should be acknowledged. Although the integrated methodology combining three-dimensional geometrical reconstruction, global finite element modelling, modal characterization, and detailed structural assessment can be extended to other historical masonry aggregates, the specific conclusions obtained for the San Filippo Neri aggregate cannot be directly generalized. In particular, the dynamic interaction between adjacent buildings depends on several case-specific factors, including geometrical irregularities, differences in building heights, construction techniques, masonry properties, and the degree of mechanical connection between adjacent structural units. Consequently, the identification of approximately independent portions should be verified individually for each aggregate rather than assumed *a priori*.

Moreover, the adopted global linear elastic model does not explicitly reproduce nonlinear masonry damage mechanisms, progressive stiffness degradation, or the potentially complex behavior of interfaces between adjacent buildings. However, performing nonlinear time-history analyses on the complete three-dimensional FEM model would involve considerable computational effort, particularly considering its geometrical complexity and the large number of finite elements. In this context, the identification of approximately independent structural portions represents an important advantage of the proposed approach. The reduced dimensions of the isolated models could make nonlinear dynamic analyses more computationally affordable, while preserving the main dynamic characteristics identified through the global modal analysis. Nevertheless, their application would require appropriate nonlinear constitutive models for masonry, careful consideration of boundary conditions and structural interactions, and further verification of the representativeness of the isolated models under nonlinear response conditions.

These above-mentioned simplifications may influence the predicted stress distributions and the assessment of structural interaction. Therefore, the proposed methodology should primarily be regarded as a framework for identifying the global seismic response and the most vulnerable structural portions, rather than as a comprehensive tool for predicting local collapse mechanisms. Further applications to aggregates with different morphological and construction characteristics, together with advanced nonlinear analyses and more detailed modelling of structural interfaces, would be beneficial to assess the robustness and general applicability of the proposed approach.

8. Conclusions

This study investigated an integrated procedure for the seismic assessment of a historical masonry aggregate combining geometrical reconstruction, global modal analysis, and nonlinear modelling of smaller structural units.

The three-dimensional geometry of the San Filippo Neri aggregate was reconstructed from available plans, sections, elevations, and photographic information, preserving the main structural irregularities, wall configurations, horizontal systems, roofs, and vaulted

components. The reconstructed geometry proved to be fundamental for interpreting the structural interaction within the aggregate. Differences in building height and mutual constraint were reflected directly in the global modal response.

Modal analysis of the complete aggregate identified two portions exhibiting distinguishable dynamic behavior. When Portions 1 and 3 were analyzed independently, their dominant vibration periods and mode shapes remained closely comparable with those obtained from the full model. The results therefore indicate that geometrical interpretation and dynamic consistency should be considered jointly when defining structural units in historical masonry aggregates.

The equivalent-frame analysis of Palazzo Jenca provided a structure-specific behavior factor of $q = 1.91$. This behavior factor was used in the global linear dynamic assessment, leading to results consistent with those obtained from the nonlinear procedure for the investigated case.

Overall, the proposed strategy suggests that accurate geometrical reconstruction can provide more than the geometrical basis for FE modelling: when combined with global dynamic analysis, it can also support the mechanical interpretation of structural interaction and the identification of suitable units for seismic assessment.

Further applications to different aggregate morphologies and construction typologies are required to verify the general applicability of the approach and to establish quantitative criteria for dynamic structural-unit identification.

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