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SEISMIC VULNERABILITY ASSESSMENT OF COMPLEX HISTORICAL MASONRY AGGREGATES WITH INTERACTIONS BETWEEN STRUCTURAL UNITS

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Abstract

Large scale complex historical buildings, such as palaces or historical city centres, are usually composed of aggregates that were built or connected in different phases. Consequently, under seismic action, there will be an interaction between the buildings that needs to be taken into account when performing their seismic vulnerability assessment. As seen by past seismic events, historical masonry aggregates present a high vulnerability to seismic action.

In this paper, different types of connections will be modeled between adjacent bodies and their effects on the global seismic response will be studied. The National Palace of Sintra, Portugal, an excellent example of large-scale irregular complex rubble stone masonry historic buildings, will be considered as a case study. The Palace is composed of several units built during different time epochs, starting from the XIII Century until the XVII. A collection of the different cases of connections between the units presented in the Palace will be organized and delivered here, according to the presence of different heights and/or floor levels, walls that are connected in same direction or perpendicular to each other, and other irregular cases that were found.

Using some units of the Palace as an example, the modeling approach considered for each type of interaction between the units by means of EFM will be described. The results of the global seismic assessment carried out with nonlinear static analyses will be presented and discussed. A comparison regarding the models with and without considering the connections of the adjacent buildings will also be delivered. The results presented here will be useful for the seismic rehabilitation of the historical buildings composed of aggregates. Engineers in common design practice will be able to model the interactions between units following this approach that uses the EFM, a low time consuming and computational effort numerical method, in the design of strengthening interventions of complex masonry aggregates.

Keywords: Historical aggregates, interactions between buildings, equivalent frame modeling, nonlinear static analyses.

1. General Considerations

The evaluation of seismic vulnerability in existing masonry buildings is a key step in devising effective mitigation strategies and enhancing the resilience of urban infrastructure. This importance is underscored by the significant damage observed during past seismic events ([1], [2], [3], and [4]). Unreinforced masonry (URM) structures, which are widespread in both historical and urban settings, present a variety of materials, construction methods, and irregular geometries that complicate their seismic performance. This complexity is further heightened by their frequent inclusion in building aggregates, where structural connections can vary in type and effectiveness. Factors such as differing construction techniques from various time periods, inconsistent materials, irregular roof and floor

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heights, and suboptimal renovations often weaken the overall structural behavior [5], thereby increasing the seismic vulnerability of these aggregates [6], as shown in Fig. 1.

Figure 1. Local mechanism and in-plane damage in existing aggregates in Amatrice, Italy, after the 2016 Central Italy earthquake (Photo: A. Penna/G. Guerrini).

Understanding how the units within an aggregate interact presents a significant challenge, particularly when historical documentation is limited and on-site surveys are constrained. Identifying individual units within an aggregate can be difficult. Given these aggregate effects, it is essential to assess the seismic vulnerability of the entire aggregate rather than evaluating isolated units. These complexities highlight the need for a robust methodological framework for evaluating seismic vulnerability in masonry aggregates. Recent studies, such as those by [7], [8], and [9], have begun developing models and methods to better understand and calibrate interactions within historical aggregates.

This study examines the interactions between units in irregular aggregates, focusing on the role of connections between adjacent units and their impact on seismic behavior. The research uses the National Palace of Sintra as a case study, a complex aggregate structure built over multiple periods, representative of many historical palaces and ancient urban centers. Numerical models are developed for both standalone units and those integrated within the aggregate. A previously developed aggregate model featuring monolithic connections ([10]) has been further refined and adapted for this study. The findings provide detailed insights into the in-plane seismic response of irregular URM walls and the interactions between neighboring units, analyzed through pushover curves.

2. Case Study

The National Palace of Sintra (NPS), situated in the village of Sintra on the outskirts of Lisbon, Portugal, comprises a series of royal residences that form a complex of buildings developed through successive construction phases, additions, and adaptations over several centuries. The exact foundation date of the oldest section of the Palace remains unknown. It is believed that the initial structures were built around the 10th or 11th century during the Moorish rule of Sintra, corresponding to the foundations of the northern part of the Palace. However, the primary construction and renovation phases occurred during the reign of King João I (1357–1433), with significant additions made later during the reign of King Manuel I (1469–1521). It was during the period of this last mentioned king, that by his order the first official drawings of the Palace were made. Fig. 2 presents the North façade of the Palace in 1507 and a current image of said façade. It is possible to observe that the three main central units remain similar in time with minor differences, comprising (from left to right) the kitchen with two large chimneys, the

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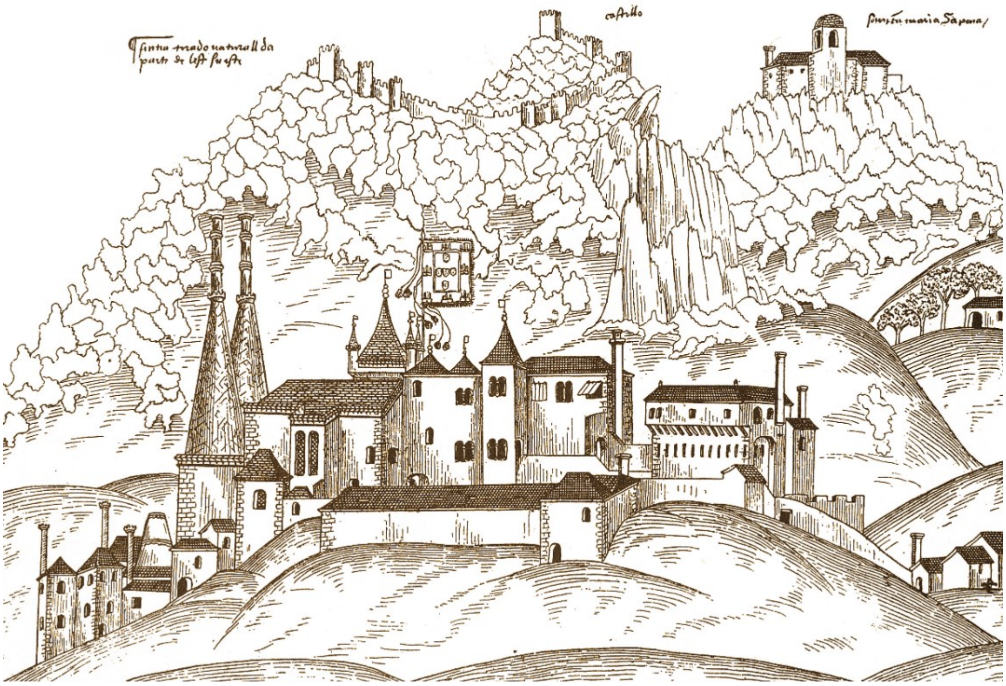
cross-shaped chapel (from here on identified as Unit 2), and, on the right, adjacent to the chapel, a large building with four rooftops and a protrusion in its center (from here on identified as Unit 1). The National Palace of Sintra exemplifies the aggregate building process, illustrating how such constructions evolve naturally through the growth of isolated units, with additional structural elements incorporated over time.



a)

b)

Figure 2. North Façade of the National Palace of Sintra: a) 1507 drawing by Duarte D'Armas, and b) current photo captured by the drone of IST.

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Identifying the main construction periods of the Palace required an initial extensive investigation into the Palace's history, supported by the historian team from Parques de Sintra - Monte da Lua (PSML). This research was complemented by a comprehensive experimental campaign, with sample removal and analysis, flat-jack tests, ambient vibration tests and ground-penetrating radar tests, and a detailed geometric survey using laser scanning and drone technology [10]. However, detecting the connections between sections built in different periods presents significant challenges due to the limited historical records on the construction phases and subsequent structural modifications. Consequently, some uncertainty remains regarding the precise nature of these connections. The identified connections between these sections were categorized into in-plane and in-height connections. The in-plane connections were further subdivided into (i) connections formed by the thickening of existing walls or (ii) connections created by the intersection of walls. The in-height connections, on the other hand, were classified based on the misaligned floor levels between adjacent buildings, where the storey height difference was either (i) smaller or (ii) greater than the thickness of the dividing wall. To analyze the interactions within the aggregates, four units of the oldest part of the NPS complex were selected (as identified in Fig. 3), corresponding to a former residential area, previously identified as Unit 1, and its adjacent buildings.

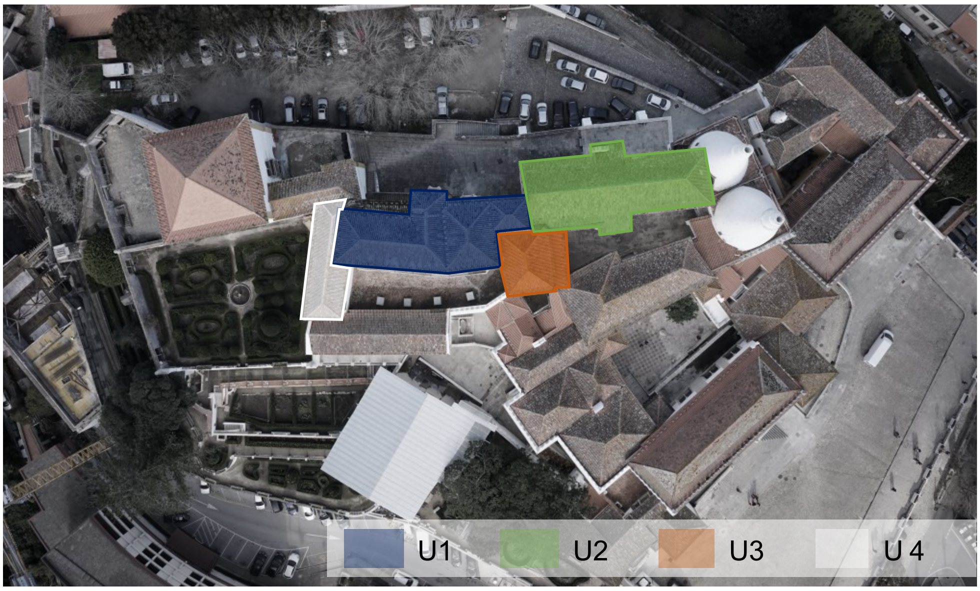

Figure 3. Case study units (Photo: IST drone)

Acquiring accurate geometry using traditional methods is highly challenging for buildings as complex, expansive, and irregular as the National Palace of Sintra. Consequently, the geometrical data were collected using laser scanning and drone surveys, as described in [11]. Unit 1 comprises three to four levels, including an intermediate floor in the central area, with floor heights varying between 2.4 m and 5.8 m. Unit 2, which serves as the chapel, on the other hand, stands at a height of 11 m and features an internal balcony positioned near its connection with Units 1 and 3. Unit 3 serves as a passage between Unit 1 and Unit 2, presenting three floors. Unit 4, also a residential area, was built more recently and presents three floors. The external walls of these buildings range in thickness from 0.65 to 0.88 m. Unit 1 is built atop an uneven and elevated bedrock, resulting in ground floor levels that differ significantly across the unit. Most of the ground floor walls are constructed directly against the bedrock. Moreover, Units 2, 3, and 4 are built at a lower ground floor level than Unit 1.

Ground Penetration Radar (GPR) revealed that Unit 2's walls are constructed with two stone leaves, with the outer leaf measuring 0.30 m thick and containing stones approximately 30–40 cm in size. At the top floor of Unit 1, the GPR identified two types of masonry: one more structured and the other less organized, with significant voids that may explain the visible cracks in the walls. All floors are flexible

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timber constructions, typically supported by main beams connected to the façades. In some areas, the timber boards are overlaid with tiles.

Additionally, previous interventions introduced tie-rods into Unit 1 and Unit 2. In Unit 1, two tie-rods are located on the upper floor of the main room, to prevent out-of-plane displacements of the two primary façades. Similarly, Unit 2 contains two rows of tie-rods linking its main façades, preventing out-of-plane movements. One alignment of tie-rods overlaps with the masonry arch that separates the choir from the chancel on the eastern side of the transept, while another alignment overlaps with the arch separating the chancel from the altar—a later addition to the structure. Each alignment includes two tie-rods placed at different heights: one integrated into a roof truss and another at the ceiling level, visible from the building's interior.

The case study, particularly Units 1 and 2, has been previously examined by [12] and [13]. However, in these studies, their analyses have always considered the connections to adjacent units as monolithic.

3. Numerical Study of the Interactions

3.1. Equivalent Frame Modeling

The Equivalent Frame Method (EFM) was used to model the aggregates under the assumption of monolithic connections. The 3Muri software [14] was employed for constructing the models, while modifications of the geometry and refinements of the mesh were carried out in Tremuri ([15], [16]). This approach was particularly suitable for modeling complex aggregates like the National Palace of Sintra, as it significantly reduces time and computational demands. Fig. 4 illustrates the plan and 3D view of the numerical model of the four structural units in the aggregate, as obtained from the 3Muri software prior to mesh refinements.

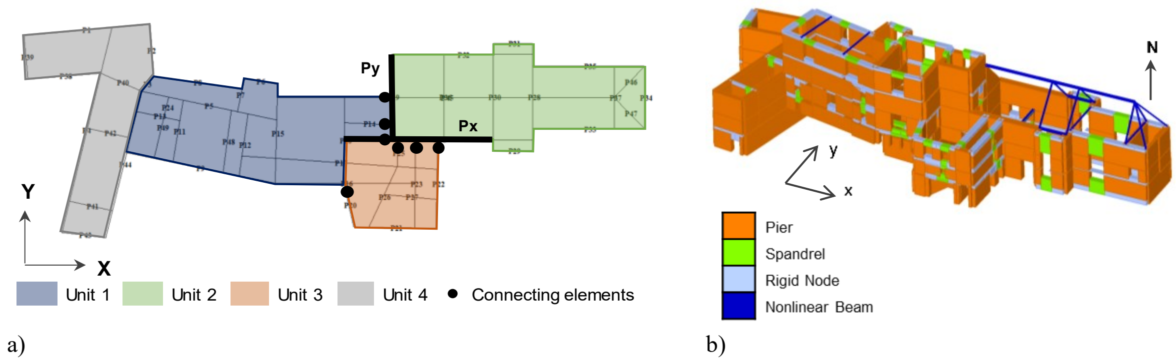

Figure 4. a) Plan view of the numerical model with the identification of the connecting elements; b) 3D view of automatically generated EF mesh.

Three different levels of interaction between the structural units were considered: negligible interaction, in which structural units were modeled as isolated, partial, and perfect connections. For the latter, the adjacent units were directly linked by sharing nodes at their intersections, eliminating the need for additional elements. On the other side, the integration of the individual units into an aggregate with partial connections was modeled using spandrel elements. The spandrels are designed to resist only compression and shear forces, with no capacity to withstand tensile stresses. Each spandrel was assigned a length equal to half the thickness of the shared transverse wall, simulating the embedded portion of the walls within the existing structure. For the case-study units, only connections between walls oriented perpendicularly were modeled. Floor misalignments were considered when modeling full or partial connections by dividing the pier at the height of each floor. Given the numerous irregularities in the case-study aggregate, the analysis concentrated on the response of Unit 1 to different types of interactions with the adjacent units, considering only partial connections on one side of Unit 1 (located in Fig. 4), specifically with Units 2 and 3. The connections with Unit 4 on the opposite side were simplified and modeled as perfect.

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The materials were characterized through the extensive experimental campaign outlined in [10]. A numerical model, assuming perfect connections between four Units, was employed to calibrate the mechanical properties of the masonry. For small vibrations, such as those recorded during ambient vibration tests, the connections between the units were assumed to behave monolithically. The masonry properties used in the numerical model are listed in Table 1.

Young's modulus (E) and shear modulus (G) were calibrated using the dynamic characterization results obtained from ambient vibration tests conducted in Units 1 and 2. Observations of the masonry in exposed inspection areas revealed disorganized, irregular rubble stone in Unit 1 and roughly dressed rubble masonry in Unit 2. Based on the calibrated values of E and G, other material properties, such as tensile strength (f_t), compressive strength (f_p), and weight (w), were proportionally derived from reference values in the Italian Standard [17] for the identified masonry types. The double flat-jack test results were found to be inconsistent and unreliable for determining compressive strength and Young's modulus, so only the stress levels from single flat-jack tests were used to calibrate the masses in the numerical model.

Additionally, the top floor of Unit 1 was determined to be in poor condition, exhibiting significant cracks. Consequently, the masonry properties for this level were reduced by half. For the unit located adjacent to the west side of Unit 1 (not included in this study), the same calibration methodology was applied to a separate model. No experimental tests were performed on Unit 3, and its material properties were assumed to match those of Unit 2, given its good state of preservation and similar construction period. The material properties for Unit 4 were calibrated using ambient vibration tests conducted on another unit adjacent to Unit 4, built during the same era.

Table 1. Masonry properties (adapted from [10])

	***Young's modulus, E* (GPa)**	***Shear modulus, G* (GPa)**	***Tensile strength, f_t* (MPa)**	***Compressive strength, f_c* (MPa)**	***Weight, w* (kN/m³)**
			Disorganized irregular stone masonry		
MIT (2019) [17]	0.69 – 1.05	0.23 – 0.35	0.03 – 0.048	1.0 – 2.0	19
Unit 1	0.80	0.26	0.036	1.24	18
Unit 4	1.20	0.40	0.063	2.43	18
			Roughly dressed rubble masonry with varying leaf thickness		
MIT (2019) [17]	1.02 – 1.44	0.34 – 0.48	0.0525 – 0.0765	2.0	20
Unit 2 / Unit 3	1.44	0.48	0.08	3.00	19

3.2. Numerical Results

3.2.1. Nonlinear Static Analysis

Nonlinear static analyses were carried out in the X direction (longitudinal) and Y direction (transverse) for a uniform loading, proportional to the mass. Fig. 5 presents the pushover curves for the three types of connections in terms of the base shear coefficient of Unit 1 versus the average top-floor horizontal

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displacement of the same structural unit. The base shear coefficient was determined by calculating the ratio of the total horizontal force to the total vertical force at the base of the first floor without a supporting wall built against the bedrock. Each curve is displayed up to the ultimate displacement of Unit 1, defined as the point at which either a lateral strength reduction of at least 20% or the formation of a collapse mechanism is observed.

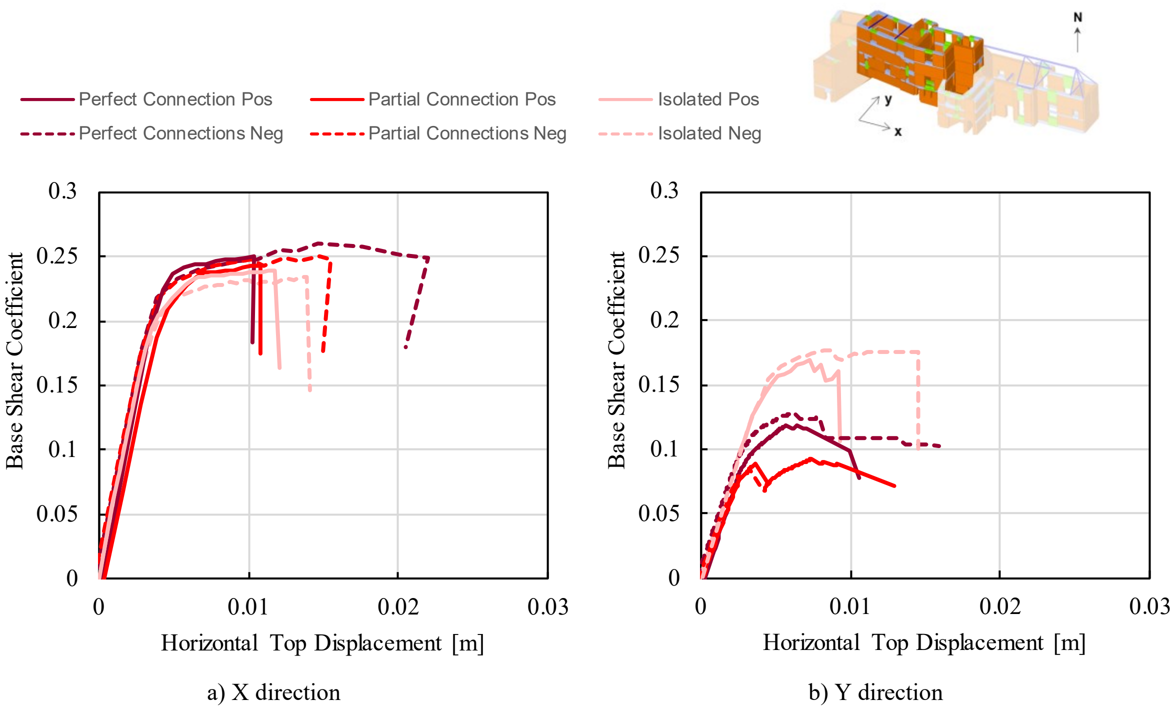

Figure 5. Pushover curves of Unit 1: a) X direction, b) Y direction, considering a uniform loading.

The pushover curves indicate that the aggregate effect does not significantly influence the behavior of Unit 1 in the longitudinal direction of the aggregate (X direction). Units 1 and 2 exhibit similar stiffness, while Units 3 and 4 have a minimal impact on the overall stiffness of Unit 1. Consequently, when subjected to loading in the X direction, the aggregate moves cohesively. In fact, regardless of the connection type, Unit 1's collapse is always governed by a soft-story mechanism in the same façade wall. This mechanism is characterized by a concentration of damage on a single floor preceding failure, resulting in a sudden loss of lateral strength. However, although the same collapse mechanism occurred for the three connection types and the global behavior is not influenced in this direction by the type of connection, not all walls presented the same behavior. For example, Fig. 6 shows the damage pattern that changes with the type of connection in a wall in the X direction shared between Unit 1 and Unit 2, named Px as identified in Fig. 4. The isolated model presents higher concentrated damage at the lower levels, while for the model with perfect connections, the damage is shared between the two buildings on the upper floors. In addition, for partial connections, the connecting elements present tensile stresses, allowing the opening of a gap between the two structural units.

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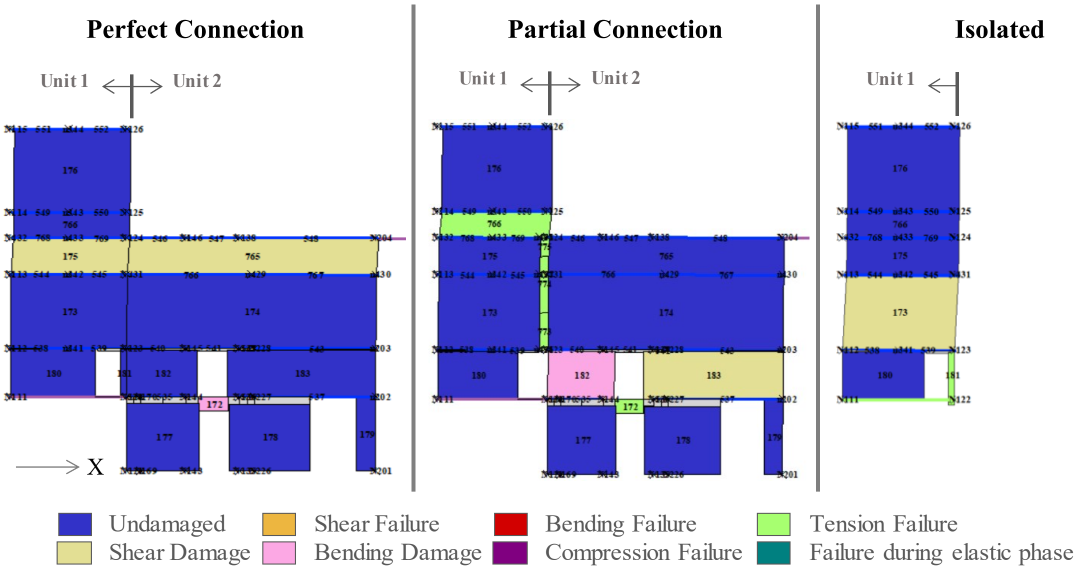
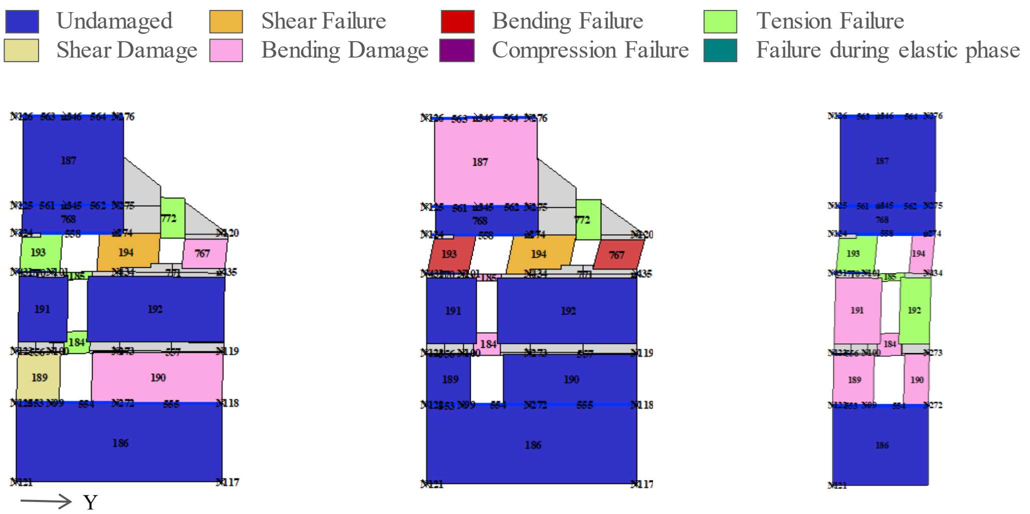


Figure 6. Damage pattern of a wall Px for a top-of-wall horizontal displacement of 0.01 m.

Unlike the X direction, the Y direction reveals a significant aggregate effect on Unit 1's behavior. This influence is not manifested in stiffness changes, largely dictated by the position, geometry, and height of adjacent units in the aggregate. Instead, it affects the behavior of the walls, leading to variations in collapse mechanisms and lateral strengths. Among the three models, the isolated configuration demonstrates the highest strength, followed by the models with perfect and partial connections. Unit 4 restrained Unit 1 within the aggregate, introducing slight torsion. In fact, in the aggregate, the wall shared between those two units presented low force values and remained in the elastic phase when the model started losing resistance. In addition, the different connections introduced different mechanisms of collapse, as presented in Fig. 7. For perfect connections, the damage on wall Py (identified in Fig. 4) is localized in two floors, and the collapse starts to occur on the upper floor with damage (as shown in Figure 7a); while, with partial connections, the wall presented lower strength due to the soft-story collapse localized on an upper floor (Figure 7b). On the other side, for the isolated model, the damage is distributed in height between three floors (Figure 7c).



a) b) c)

Figure 7. Damage pattern of wall in Y direction shared between Unit 1 and Unit 2, for a top wall horizontal displacement of 0.02 m.

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In terms of displacement capacity, the results varied depending on the direction and connection type. In the positive X direction, displacement capacity remained similar regardless of the connection type, whereas in the negative X direction, it increased with the level of connection. In the Y direction, the model with perfect connections exhibited a slightly higher displacement capacity compared to the isolated model. The partial connection model demonstrated the highest displacement capacity in the positive Y direction, while in the negative Y direction, the collapse of Unit 1 was not reached due to the premature failure of wall Py.

4. Conclusions

The study explores the modeling of complex building aggregates using the Equivalent Frame Method (EFM), using the National Palace of Sintra in Portugal as a case study. This historic structure, originating in the 10th century, has undergone extensive modifications and expansions over the centuries, both horizontally and vertically. The investigation focuses on a specific section of the aggregate, comprising four structural units, which were analyzed both as isolated and as part of the aggregate. Particular attention was given to the type of connections between three of these units, considering two configurations: a monolithic connection representing perfect continuity, with shared nodes between the adjacent units, and a partial connection characterized by spandrel elements made of rubble stone masonry without tensile strength. The impact of varying story heights on the behavior of adjacent piers was also assessed by segmenting the pier next to the connection.

Nonlinear static analyses were performed to evaluate the in-plane response of these connections. The pushover curves reveal notable differences in the behavior of the unit only in the Y (transverse) direction when modeled as isolated or part of the aggregate. Modeling the connections as perfect, as would be done when designing a retrofit solution, allows for taking advantage of the lateral strength of Unit 1, resulting in a strength increase compared to models with partial connections. However, the lateral strength of the aggregate models remains lower than that of the isolated model, which offers a conservative approach, while the deformation capacity is slightly greater. These varying levels of connection influence the collapse mechanisms of the aggregate, thereby affecting its lateral strength and ultimate displacement. In the X (longitudinal) direction, the global pushover curves remain largely unaffected by the type of connection; however, differences were observed in the damage patterns of individual walls. Despite minimal global influence in this direction, this highlights the localized impact of connection variations within the aggregate.

While the results of such a complex case study did not allow for generalized conclusions about whether the aggregate effect was beneficial or detrimental, it was evident that incorporating spandrels resulted in a more realistic representation of damage patterns. Further research is suggested to test alternative connection modeling techniques and to expand the analysis to other units within the aggregate. Additional insights would require conducting nonlinear dynamic analyses to generalize the aggregate effect.

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DOI: https://doi.org/10.5592/CO/3CroCEE.2025.146

References

[1] Guerreiro, L., Azevedo, J., Proença, J., Bento, R., Lopes, M. (2000): Damage in ancient churches during the 9th of July 1998 Azores earthquake, *XII World Conference on Earthquake Engineering*, Auckland, New Zealand.

[2] D'Ayala, D.F., Paganoni, S. (2011): Assessment and analysis of damage in L'Aquila historic city centre after 6th April 2009, *Bulletin of Earthquake Engineering,* 9(1),81–104, doi: 10.1007/s10518-010-9224-4

[3] Penna, A., Morandi, P., Rota, M., Manzini, C.F., da Porto, F., Magenes, G. (2014): Performance of masonry buildings during the Emilia 2012 earthquake, *Bulletin of Earthquake Engineering*, 12, 2255–2273, doi: 10.1007/s10518-013-9496-6

[4] Sorrentino, L., Cattari, S., Da Porto, F., Magenes, G., Penna, A. (2019): Seismic behaviour of ordinary masonry buildings during the 2016 central Italy earthquakes, *Bulletin of Earthquake Engineering*, 17(10), 5583-5607, doi: 10.1007/s10518-018-0370-4

[5] Carocci, C. F. (2012): Small centres damaged by 2009 L'Aquila earthquake: on site analyses of historical masonry aggegates, *Bulletin of Earthquake Engineering*, 10, 45-71, doi: 10.1007/s10518-011-9284-0

[6] Cardinali, V., Cristofaro, M.T., Ferrini, M., Nudo, R., Paoletti, B., Tanganelli, M. (2021): A Multiscale Approach for the Seismic Vulnerability Assessment of Historical Centres in Masonry Building Aggregates: Cognitive Approach and Interdisciplinary Perspectives, *International Journal of Architectural Heritage*, 16(6), 839–64, doi: 10.1080/15583058.2021.1992536

[7] Angiolilli, M., Pinasco, S., Cattari, S., Lagomarsino, S. (2023): On the vulnerability features of historical masonry buildings in aggregate, *Procedia Structural Integrity*, 44, 2074–2081, doi: 10.1016/j.prostr.2023.01.265

[8] Gagliardo, R., Godio, M., Portioli, F.P.A., Landolfo, R. (2023): Seismic analysis of failure mechanisms in adjacent interacting stone masonry buildings via rigid block modeling, *Bulletin of Earthquake Engineering*, 22(12), 6195-6224, doi: 10.1007/s10518-023-01659-1

[9] Tomić, I., Beyer, K. (2023): Shake-table test on a historical masonry aggregate: prediction and postdiction using an equivalent-frame model, *Bulletin of Earthquake Engineering*, 22, 6225-6258, doi: 10.1007/s10518-023-01765-0

[10] Ponte, M., Bento, R., Vaz Silva, D. (2019): A multi-disciplinary approach to the seismic assessment of the National Palace of Sintra, *International Journal of Architectural Heritage*, 757-778, doi: 10.1080/15583058.2019.1648587

[11] Godinho, M., Machete, R., Ponte, M., Falcão, A.P., Gonçalves, A.B., Bento, R. (2019): BIM as a resource in heritage management: An application for the National Palace of Sintra, Portugal, *Journal of Cultural Heritage,* 43, 153-162, doi: 10.1016/j.culher.2019.11.010

[12] Malcata, M., Ponte, M., Tiberti, S., Bento, R., Milani, G. (2020): Failure analysis of a Portuguese cultural heritage masterpiece: Bonet building in Sintra, *Engineering Failure Analysis*, 115, 104636, doi: 10.1016/j.engfailanal.2020.104636

[13] Ponte, M., Bento, R., Costa, A., Quelhas, B., Guedes, J.M., Ilharco, T., Lopes, V. (2021): Reduction of earthquake risk of the National Palace of Sintra in Portugal: The palatine chapel, *International Journal of Disaster Risk Reduction*, 60, 102172, doi: 10.1016/j.ijdrr.2021.102172

[14] S.T.A. DATA.: *3Muri* [Computer Program]. Italy, Version: 11.5, 2018, Available at https://www.servizi.stadata.com/download/?lang=en&sw=3muri+project

[15] Lagomarsino, S., Penna, A., Galasco, A., Cattari, S. (2013): TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings, *Eng. Struct.,* 56, 1787–1799, doi: 10.1016/j.engstruct.2013.08.002

[16] Penna, A., Lagomarsino, S., Galasco, A. (2014): A nonlinear macro-element model for the seismic analysis of masonry buildings, *Earthquake Engineering & Structural Dynamics*, 43(2), 159-179, doi: 10.1002/eqe.2335

[17] MIT. (2018): DM 17/01/2018. Norme Tecniche per le Costruzioni. In *Gazzetta Ufficiale della Repubblica Italiana*. Rome, Italy.