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PROPOSAL OF A GENERAL METHODOLOGICAL APPROACH FOR THE SEISMIC ASSESSMENT OF HISTORICAL MASONRY AGGREGATES

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Abstract

The vulnerability of masonry aggregates has been highlighted by recent strong earthquakes. Most of the Italian historical centres are characterized by adjacent masonry structures connected in aggregate that have undergone to structural and functional changes over time. Their seismic vulnerability shall be studied to avoid disastrous outcomes. Many studies are available in the technical literature to identify the most vulnerable structures in historic centres from a macroscopic perspective, furthermore the seismic behaviour of some typical structures was studied in detail using sophisticated numerical methods to provide a specific seismic vulnerability assessment. However nowadays there is not a general and standard procedure available, mostly methodological, based on well-known analysis methods, that can be followed to evaluate the seismic vulnerability of any building aggregate, and that allows to identify effective retrofitting interventions. Moreover, common national regulations do not provide a standard procedure for practitioners to follow in such cases. Thus, the purpose of this contribution is to show a protocol to be used in common design, that has been developed in a wider work, and it is based on a broad blend of analyses that can be performed with low-cost software. With different hypotheses and modelling assumptions, numerical analyses range from simple eigenvalue simulations to full 3D pushover computations. The suggested method has been presented in a wider work where it has been applied and benchmarked to the ex-monastery of Santa Maria della Pace in Piacenza, Italy.

Keywords: Masonry aggregates, Protocol for seismic assessment, Broad blend of advanced analyses.

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1 INTRODUCTION

Typical historical buildings of several countries in Southern Europe, which are earthquake prone areas (e.g. Croatia, Greece, Italy and Portugal), are made of masonry; they are often erected in continuity to one another and generated over the centuries by the progressive transformation of the urban tissue. Masonry buildings were typically built following simple rules of thumb, disregarding anti-seismic criteria; thus their vulnerability is expected to be relatively high. In light of the implementation of some seismic protection measures, there is the need to make available reliable numerical tools and protocols aimed at quantifying the horizontal acceleration they can withstand and identifying their most critical parts.

In terms of code of practice recommendations, for instance the Italian building code [1],[2] does not specify a standard procedure to be followed in assessing the seismic vulnerability of buildings embedded in urban aggregates. While the research in the field appears to be currently primarily focused on the investigation of specific structures [3]-[19].

The behaviour of a building aggregate is never standard and cannot be a priori predicted with sufficient accuracy; indeed, the structural behaviour is strongly influenced by the materials, the global and local regularity, the presence of rigid floors and the effectiveness of the connections. In addition, building aggregates are often characterized by the presence of different structural typologies.

A methodological approach that is sufficiently general to be applied in the broadest range of cases is still lacking in the technical literature. For the seismic vulnerability assessment of historical centres, several methods have been proposed, such as [20],[21]; they are useful for identifying the urban aggregate units that most likely need urgent retrofitting interventions, but specific additional analyses should be performed to identify more precisely the weaknesses of the structure and implement protection strategies. On the other hand, there are approaches, such as [22]-[25], which focus solely on the vulnerability for the activation of out-of-plane collapses, disregarding the possibility that the global behaviour of the structure is dominant.

Numerous investigations have been conducted for particular case studies, such as those for the “Pombaline” system in the downtown of Lisbon [26] or the “Casa Baraccata” in Italy [27]; these studies have focused on some non-standard building techniques. Several other advanced studies were conducted on common aggregates, but they are all specialized and primarily relevant for the examples examined, see for instance [28].

The role played by contiguous buildings in terms of stiffness, strength and dynamic behaviour is critical and difficult to be understood; indeed, studies on the interaction with neighbouring buildings are still at their early stages, and only highly specialised investigations had been presented, such as that performed for the Gabbia tower in Mantua (Italy) [29].

The general methods of analysis available nowadays for the seismic vulnerability assessment of existing buildings were described by Calvi et al. in [30], where benefits and drawbacks were underlined; however, a methodological approach capable of guiding practitioners step-by-step (namely a protocol) in a systematic evaluation on the level of reliability and importance of the results obtained in practical applications seems to be still missing.

This contribution is an extract of a wider work [31] and aims to present a general methodology that can be applied to any masonry compound inserted in a specific urban aggregate, based on well-known standard analyses, for a precise evaluation of the expected seismic vulnerability and the identification of the critical portions of the compound, in view of future retrofitting interventions. The method was discussed referring to the ex-monastery of Santa Maria della Pace in Piacenza, Italy, which is characterized by the presence of several different building typologies embedded in the compound.

2 THE METHODOLOGICAL APPROACH: NUMERICAL ANALYSES SUGGESTED AND PROTOCOL

The proposed protocol is summarised in the flowchart shown in Figure 1. It is meant to be applied to a generic case study and provides (i) numerical analyses to perform and (ii) expected output obtainable, guiding the user in the event that churches, vaults and towers are present.

In accordance with the type of analysis (linear FEM, non-linear FEM, kinematic, pushover FEM, pushover equivalent frame) and the intended outcomes of the analyses, the characteristics of the numerical models were defined.

Historical structures belonging to aggregates were typically built disregarding the effects of seismic events; additionally, they probably underwent to functional and structural changes over time. As a result, most connections between walls, roof and decks are typically ineffective. Thus, historical structures rarely exhibit global behaviour when loaded horizontally, instead the activation of local mechanisms and partial collapses is evident.

Therefore, the first analysis that shall be performed is an elastic eigenfrequency analysis on the finite element model of the entire building aggregate, which enables the identification of all the potential local mechanisms and the most probable ones (modes associated to a high participating mass), even if only approximately since the non-linear behaviour of masonry is disregarded [32]. The 3D model used for the analyses must be as accurate as possible to obtain reliable results. If the structure is complex, meshing the 3D model may require a significant amount of time; on the contrary, the material modelling is very straightforward because it is assumed linear elastic.

Once the most likely local mechanisms have been identified, the collapse load multiplier can be evaluated using the kinematic approach of limit analysis. This method can be used to investigate the out-of-plane behaviour of various macro-elements. For masonry, a no tension material hypothesis with infinite strength in compression, no deformability of macro-blocks, and no sliding is assumed; this may lead to an underestimation of the capacity of the structural elements, but the assumption is conservative [33]-[36].

If the building aggregate is characterized by an embedded church, additional possible local mechanisms among the 28 suggested by the Italian guidelines for cultural heritage [33] should also be evaluated. This procedure is straightforward and can be used without FE models, but, since it considers only approximately the actual geometry of the structure, it is always preferable to compare the results with those obtained from an elastic eigenfrequency analysis to ensure that all the possible local collapse mechanisms are identified. Moreover, the assumption of no-tension material for masonry may sometimes lead to an underestimation of the collapse accelerations [37].

If towers or bell towers are part of a building aggregate, the LV1 analysis, again suggested in [33], can be used as a first evaluation method for their seismic behaviour. This conservative analysis, by means of simple computations identify the PGA that triggers the collapse, considering only failures due to bending and compression; more refined analyses must follow.

Once the out-of-plane behaviour has been investigated, pushover analyses can be performed on the entire structure modelled using a suitable equivalent frame if the geometry of the structure is not too complex (i.e., the irregularities are not diffused). The modelling phase and the computations are quicker because the elements are 1D, however there are also well-known drawbacks as those discussed in [38]. Pushover analyses performed on equivalent frames account for the non-linear behaviour of masonry, allow the evaluation of the failure mode inside piers and spandrels and identify the collapse mechanism of the structure, which is always global. When evaluating the behaviour of complex structures characterized by arches and vaults, pushover analyses performed on equivalent frame models can be used to assess the in-plane behaviour of meaningful masonry perforated shear panels. The computations were developed following the SAM-II method implemented in the commercial software PRO_SAM by 2SI [39]-[41].

Finally, pushover analyses can be carried out using full 3D finite element models, that can represent either portions of the structure (partial models) or the entire building aggregate (full model). This method accounts for the actual geometry of the structure, and the results may be considered the most reliable. The drawback is that compared to the equivalent frame, the modelling phase and the computations require a longer time; additionally, the user must have a solid theoretical foundation as well as sufficient experience in advanced non-linear FE modelling and masonry behaviour understanding. Detailed crack patterns can be obtained using full 3D FE models; moreover, it is possible to track the displacements of different points on the structure, which is useful to study the triggering of local mechanisms. In this work, pushover analyses on 3D FE models were performed using the software Abaqus/CAE® [42].

Further details about the different models can be found in [31].

3 CASE STUDY: EX-MONASTERY OF SANTA MARIA DELLA PACE

The monastery was built in the XVI century by Benedictine nuns; the only portion of the original structure that has survived until now has not undergone significant structural changes (plan views, sections and pictures are available in [31]). The masonry structure is characterized by a cloister layout with two levels. Cross vaults cover the cloister and the corridors of the first floor. Cloister vaults, sometimes with lunettes, cover the majority of the rooms. Few rooms are covered by decks made of timber beams and joists. The pitched roof has a bearing structure made of timber beams and joists covered by tiles.

The structure can be classified as a building aggregate because, in addition to being surrounded by adjacent buildings on two sides, a church and a bell tower are included in one wing of the complex. The church has a single large nave that is covered by a barrel vault with reinforcing arches, while the bell tower is a slender hallow structure. Further details are shown in [31].

Starting with the available plan views and sections, a detailed geometrical model of the building aggregate was created, which was then refined through measurements done during in-place surveys. The 3D geometric model was built in Revit 2022® (Figure 2a), focusing solely on structural elements because architectural elements have no significant influence on the actual structural behaviour.

3.1 Numerical modelling

The 3D geometric model was imported from Revit 2022® to the software Abaqus/CAE®. To consider the stiffness and the mass of the structure more accurately, the decks and the roof were only considered in terms of applied load on the structure, because the rigid diaphragm hypothesis cannot hold since such elements are not stiff enough in their plane. This implies that walls should be considered not constrained at the different heights, resulting in more slender walls; this hypothesis is in any case on the safe side, and it should also be considered that the wall-to-floor connection is generally rather poor in a masonry historical structure.

Tetrahedral linear elements were used to discretize the model, as shown Figure 2b, additional information are provided in [31]. The soil boundary conditions were defined using fixed constraints, with rotations and displacements assumed to be zero; also the interaction with the adjacent buildings was also modelled by means of fixed constraints. This latter hypothesis considers the adjacent buildings to be infinitely stiff, which is a quite reasonable assumption in the case under consideration because the cluster of buildings is continuous for a long portion of the street, see [31].

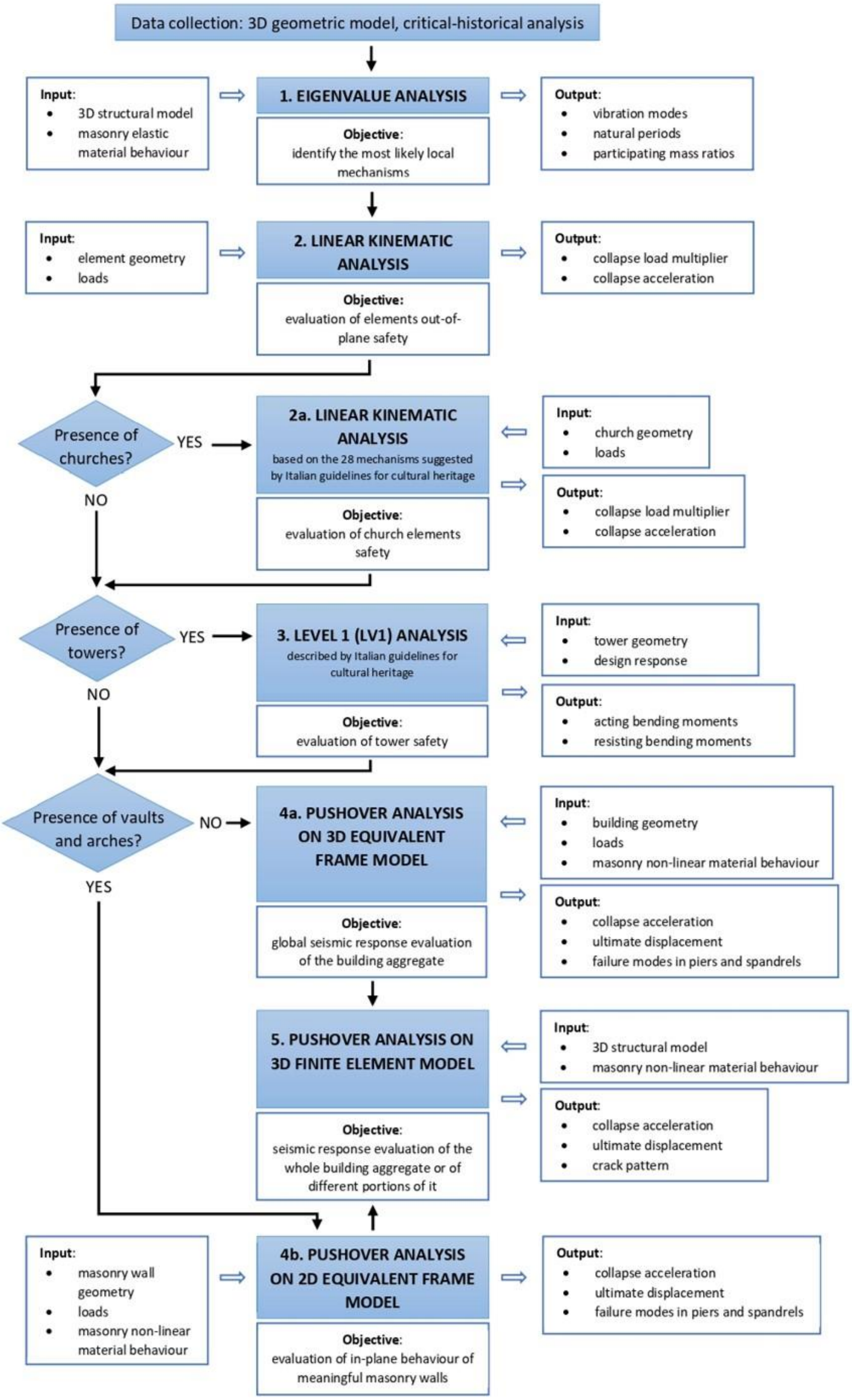
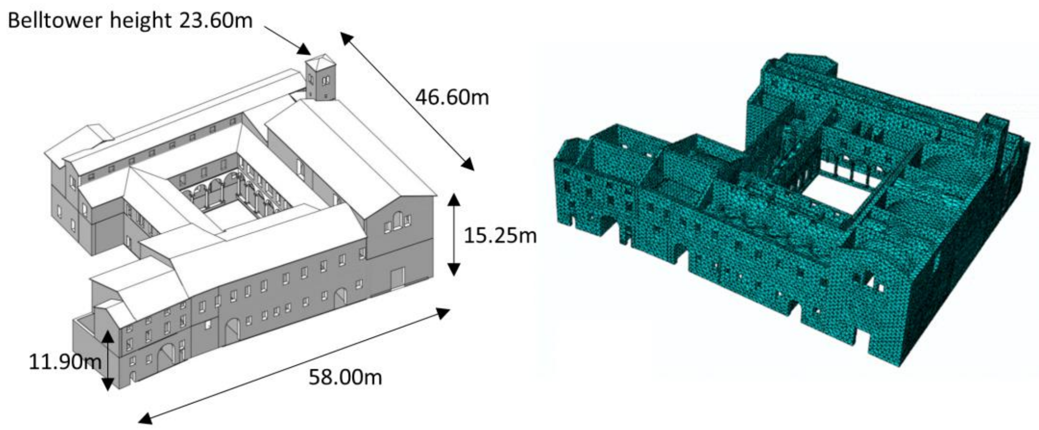


Figure 1: Flowchart of the protocol of numerical analyses for building aggregates [31].



a) b)

Figure 2: a) 3D geometrical model b) FE numerical model of the ex-monastery.

The structure is mostly made of masonry. Since there was few information available regarding the original masonry, according to [1] and [2], the mechanical characteristics were determined using the Masonry Quality Index (MQI) method developed by Borri and De Maria [43]. Masonry in the case under consideration is made of regular solid clay bricks, with an alternantion of stretchers and diatons and the lack of alignment for vertical joints, see [31]. According to [2], the level of knowledge attained for the materials must be considered when defining the mechanical properties of masonry; since only a visual inspection was carried out, the level of knowledge, in this case, was LC1 associated to a safety factor equal to 1.35, which is the lowest for the Italian code. Masonry elastic properties used for linear elastic analyses were defined (Young's modulus 1230 MPa, Poisson's ratio 0.25, density 18 kN/m^3). In order to perform non-linear analyses, the material non-linear behaviour had to be defined. In the case of masonry, the Concrete Damage Plasticity (CDP) model [44],[45] available in standard Abaqus/CAE® [46],[47] was assumed since is rather suitable, as demonstrated in [48]. The two primary failures that are reproducible are cracking in tension and crushing in compression. The parameters used for the calibration of the CDP model are listed in Table 1, they were chosen based on the results found in the technical literature [29],[49]-[51]. The compressive behaviour was described through the stress-strain diagram [52], while tensile behaviour through the fracture energy, see Table 1. The columns of the cloister, made of granite, were modelled with an elastic material (Young's modulus 5000 MPa, Poisson's ratio 0.25, density 27 kN/m^3) in the non-linear analyses as well because their strength is much higher compared to the masonry one.

Kc – Shape factor	0.667
e - Eccentricity	0.1
fb0/fc0 – Compressive yield ratio	1.16
ψ [°] – Dilatancy angle	10
μ - Viscosity	0.001

a)

f_m [MPa]	$\varepsilon_{inelastic}$ [-]
1.80	0.0000
1.90	0.0004
2.00	0.0013
1.90	0.0022
1.80	0.0026
0	0.0101

b)

f_t [MPa]	G_f [N/mm]
0.2	0.02

c)

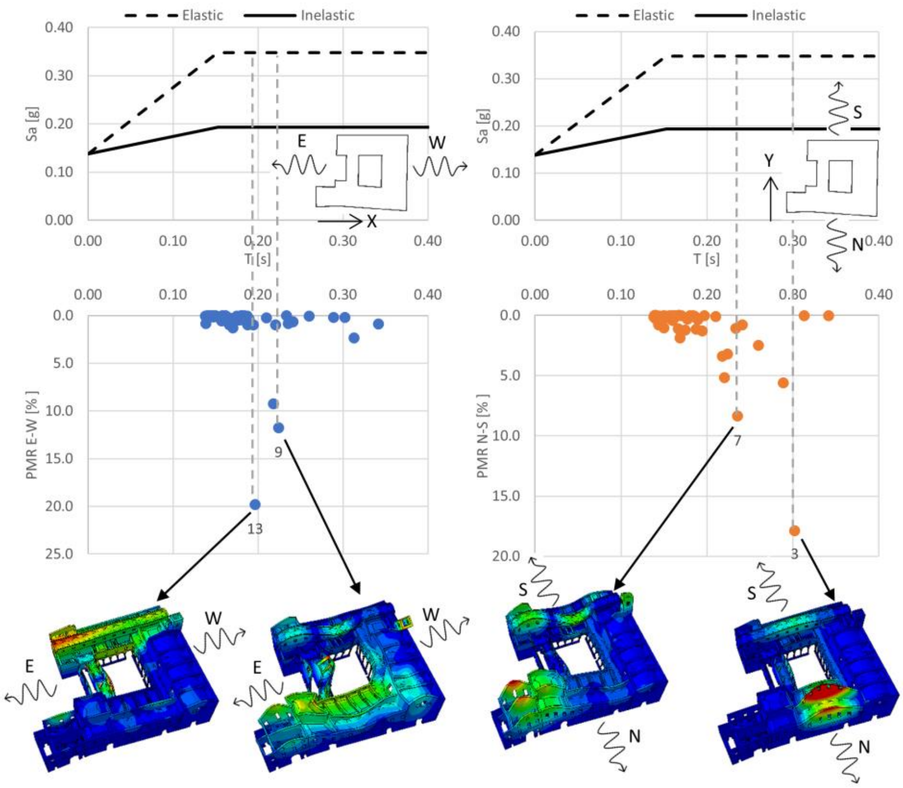
Table 1: a) CDP parameters, b) stress-strain curve in compression, c) fracture energy.

3.2 Numerical results

3.2.1. Modal analysis

Modal analysis was performed on the previously described FE model, with masonry modelled as an elastic material and the stiffness of the vaults considered 1/4 that of the masonry walls, in accordance with [33]. The analysis allows the identification of the structure's dynamic features: vibration modes, natural periods, and participating mass ratios. Figure 3 depicts the most relevant natural frequencies as well as the participating mass in North-South and East-West directions. Only local mechanisms were found because connections with floors and roof were assumed ineffective.

Figure 3 shows that significant modes exhibit periods that fall on the plateau of the design spectrum of accelerations, suggesting that all those portions of the structure involved in the active mode will be subjected to the highest acceleration, leading to the assumption that the activation of a mechanism is most likely. This is true at least while masonry still is in the elastic range, immediately following the application of the seismic action.



Mode 13 PMR E-W 19.80% | Mode 9 PMR E-W 11.77% | Mode 7 PMR N-S 8.37% | Mode 3 PMR N-S 17.87%

Figure 3: Eigenvalue analysis results of the first 50 modes [31].

3.2.2. Linear kinematic analysis

Modal analysis results indicate modal shapes with a high percentage of excited mass, suggesting the presence of possible active local collapse mechanisms. Limit analysis can thus be used to compute the collapse acceleration. Computations were performed here using the well-known excel spreadsheet "C.I.N.E.- Condizione d'Instabilità Negli Edifici" [34] provided by the Italian university labs of seismic engineering (Reluis).

Simple overturning, horizontal out-of-plane bending, vertical out-of-plane bending and overturning with crack on orthogonal walls are the available mechanisms. The collapse load multiplier α_0 is computed for each local collapse mechanism using the principle of virtual works, and then the spectral acceleration a_0^* is evaluated in accordance with [1] and [2]. The safety factor FS is defined as the ratio of the spectral acceleration required for the mechanism activation and the design spectral acceleration at the site. If the safety factor is less than 1, the structure is considered unsafe.

In Figure 4 the walls analysed are depicted (see [31] for further details) and the results are shown in terms of safety factors. According to the results of the analysis, the simple overturning for aggregates with walls of limited thickness is a critical issue. For this reason, an accurate evaluation of the effectiveness of the connection between orthogonal walls, floors and roof and the presence of tie rods is required. The confined horizontal flexure collapse mechanism can activate when connections between orthogonal walls are effective, and it is associated with greater values of collapse loads for walls of limited width, because the resistant arch effect is activated better. On the contrary, the resistant arch effect cannot give a significant contribution against the out-of-plane actions in long and thin walls. Furthermore, vertical flexure must be considered when there are thin and high walls. Once again, it is very crucial to accurately assess the effectiveness of the connections between orthogonal walls, floors and roof to determine whether the mechanism can be activated and, if so, which retrofitting interventions should be implemented. The corner overturning shall be considered for corners not constrained by adjacent buildings; connections between orthogonal walls and the roof type (e.g. thrusting type) have a great influence.

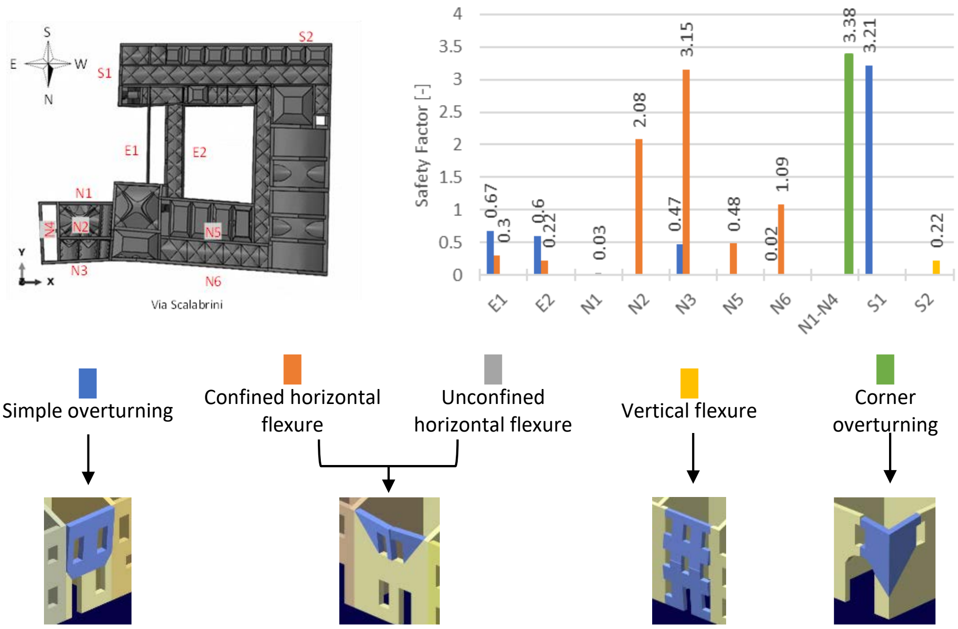


Figure 4: Linear kinematic analysis results for some selected walls.

For the portion of the building aggregate comprising the church, the local mechanisms that could be activated among the 28 suggested in [37], in this case, are the façade overturning, the tympanum rocking and the transversal out-of-plane failure of the church nave. Results in terms of safety factor are shown in Figure 5.

Once more, it is clear how crucially important assumptions about the effectiveness of connections between orthogonal walls are; if connections between the façade and lateral walls are not effective, the mechanism F2 could be activated, corresponding to a safety factor smaller than one, on the contrary, if connections are effective mechanism F1 could be activated corresponding to a safe condition. The horizontal flexure of the tympanum (T2) corresponds to a safety factor smaller than one, while simple overturning mechanism (T1) corresponds to a safe condition. Simple overturning of the nave lateral walls is not active because the roof is not of thrusting type. Even though the longitudinal walls are thin, they are constrained by tie rods inside the church; in addition, the wall is constrained on one side by cross vaults of the cloister outside the church, and on the other side by adjacent buildings. In this case, the configuration of the church within the building aggregate has beneficial effects, preventing the lateral overturning of the nave walls.

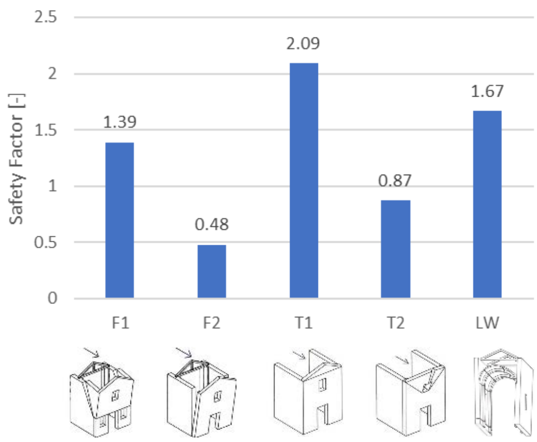


Figure 5: Linear kinematic analysis results for the church.

3.2.3. Bell tower LV1 analysis

The seismic vulnerability of the bell tower, embedded in the building aggregate object of study, was first assessed using the so-called LV1 analysis as defined in [33]. The bell tower is laterally connected to adjacent buildings of different heights, therefore two cases have been considered, which most likely provide a lower and upper bound of its actual behaviour. In the first case, the tower is assumed fixed at a height corresponding to the maximum one of the neighbouring walls, while in the second one the fixed base is in correspondence of their minimum height.

The seismic load is applied statically through an inverse linear distribution. The obtained results are summarized in Figure 6, where the acting bending moments (two cases) are compared to the resisting ones. The tower is modelled as a cantilever beam with perforated cross sections assuming a no tension material with a limited compressive strength. The critical section is located at the base, at 11.60m for case 1 and 15.25m for case 2; as evidenced by the regularity of the resisting bending moment shape, the small openings present on the tower do not significantly reduce the resisting moment of the sections. The diagrams clearly show that case 2 is the most unfavourable. In both cases the tower is safe because the safety factor is greater than one.

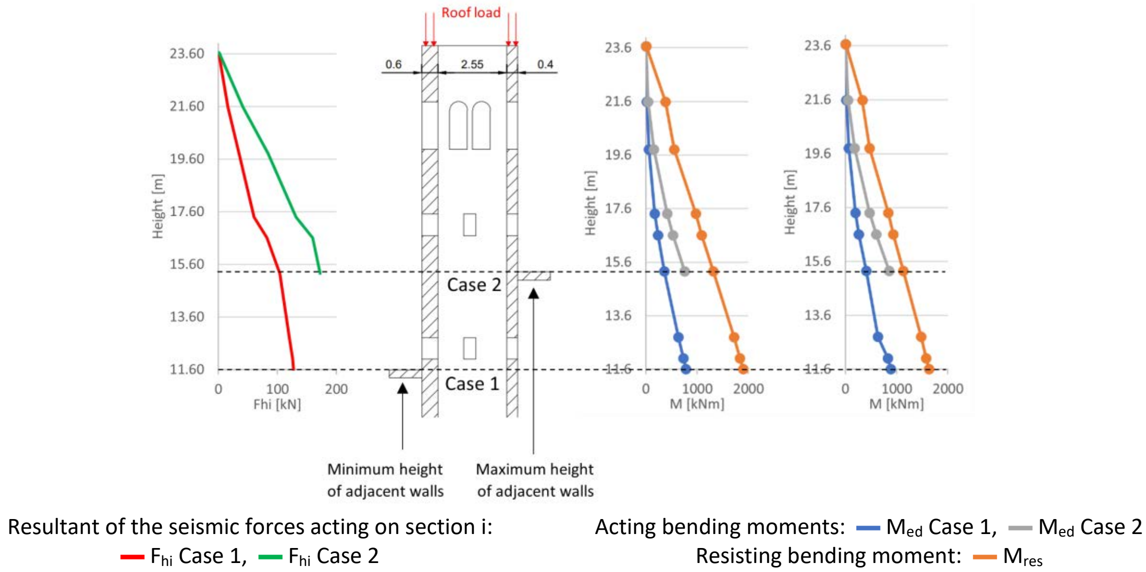


Figure 6: LV1 analysis results [31].

3.2.4. Pushover analysis on equivalent frames

Once the structure out-of-plane behaviour has been assessed in a simplified way, the in-plane behaviour of meaningful in-plane loaded walls can be studied using the equivalent frame method. Pushover analyses were performed with uniform and triangular force distributions in positive and negative directions; the analyses results are the capacity curves in terms of total shear at the base and displacement of a control point, deformed shapes with activation of the plastic hinges on the elements, and failure mechanism (see [31]).

In Figure 7 the walls analysed are shown and the results are depicted in terms of safety factor given by the ratio between the displacement capacity and the displacement demand. Almost all of the walls reached the SLV limit state for the same peak ground acceleration, resulting in a pier shear failure.

The displacement capacity can vary significantly between positive and negative directions. The geometry of the panel clearly influences the displacement capacity, in particular a crucial role is played by the dimension of the piers and the placement of the openings. Based on the analyses results, it is evident that the most vulnerable walls are those having big openings at the ground level, or those having multiple openings and thin piers. Walls characterized by a large number of openings are more flexible, resulting in a higher displacement demand and a lower safety factor. However, because of its conservative assumptions, the equivalent frame model generally provides conservative results.

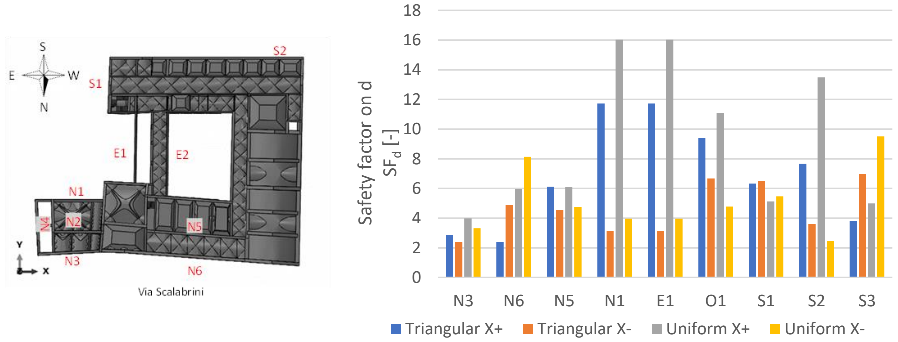


Figure 7: Pushover analysis on equivalent frame models results in terms of safety factor.

3.2.5. Pushover analysis on 3D FE models

The pushover analyses performed with 3D FE Abaqus models could be one of the most accurate analyses that practitioners try to perform. Three significant portions of the aggregate under study have been analysed (Figure 8). The southern portion is characterized by a basilica layout. Moreover, it is interesting because there are large rooms covered by cloister vaults at the ground level, bearing one of the highest walls of the first level (W1) in the middle span. The northern portion has a more regular layout, while the western portion is representative of the church embedded in the building aggregate. The portions under investigation were isolated from the rest of the context; studies found in the technical literature show that isolating portions of the aggregate leads to conservative results because the confinement given by the context is ignored.

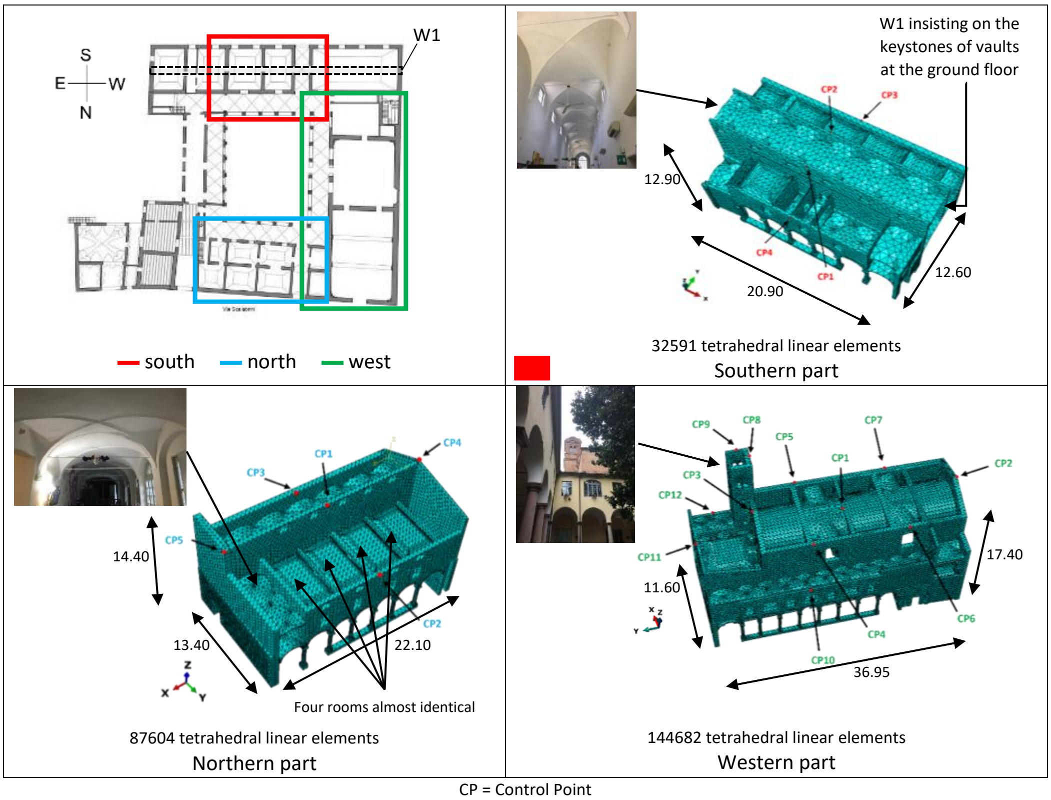


Figure 8: Portions of the structure analysed.

Pushover analyses results in terms of equivalent plastic strains for each portion investigated and capacity curves of significant control points are shown in [31]. Pushover analyses were performed loading the structure both for positive and negative North-South and East-West directions.

A summary of the analyses in terms of collapse accelerations of all the portions is shown in Table 2. The most vulnerable portion of the aggregate clearly is the western one, which includes the church. The nave longitudinal walls are not constrained by perpendicular walls that provide extra strength against out-of-plane bending. Indeed, the lowest maximum PGA was found for East-West horizontal loading direction (direction perpendicular to the longitudinal walls). Another vulnerable component is the bell tower, which shows significant displacements [31]. In addition, when the structure is subjected to a horizontal load along the South-North direction horizontal bending of the façade is activated [31]. This local mechanism was found to be critical using linear kinematic analysis as well, but the collapse acceleration found was lower, most likely due to the conservation assumption made in that case on the material and the interlocking and the different shape of the failure mechanism. The walls typical of the basilica layout of the southern portions are characterized by significant displacements (see [31]), implying that irregularity in elevation may lead to a higher seismic vulnerability. It has been found the activation a local mechanism due to confined horizontal flexure on the spine wall of the northern portion [31]. The activation of this mechanism was also discovered through linear kinematic analysis, but it was associated with a lower acceleration, once again it is a consequence of the conservative hypotheses made.

Collapse acceleration a_g[g]	Loading direction			
	E-W	W-E	N-S	S-N
Southern part	0.50	0.64	0.31	0.21
Western part	0.28	0.23	0.34	0.45
Northern part	0.63	0.60	0.63	0.62

Table 2: Collapse acceleration of the portions analysed.

4 CONCLUSIONS

Based on what was deeply presented in [31] and developed here, the following conclusive remarks can be drawn. In this paper, a general methodological approach to assess the seismic vulnerability of aggregates has been proposed, based on well-known analysis strategies that nowadays can be commonly used by practitioners with standard commercial codes. The procedure is a protocol that consists in performing various types of computations, from the simplest to the most refined, then in comparing the results and finally in drawing conclusions rating the reliability of the different methods used. The procedure allows to evaluate effective retrofitting interventions to reduce the seismic vulnerability of aggregates and it has been benchmarked on the ex -monastery of Santa Maria della Pace in Piacenza, Italy.

A linear eigenfrequency analysis has been performed on the structure, by means of the software Abaqus/CAE® and a full 3D FE model, for the identification of the most probable local failures. Then, limit analysis has been used to compute the collapse acceleration, using the spreadsheet C.I.N.E.. For the church, the 28 local mechanisms suggested by the Italian guidelines for the cultural heritage have been considered and a first assessment of the bell tower seismic vulnerability was done using the LV1 approach. After evaluating the structure out-of-plane response, pushover analyses were used to investigate the in-plane response of significant masonry walls, modelled with the equivalent frame, using the software PRO SAM. The out-of-plane behaviour was found to be critical, because masonry walls are verified in their plane. Finally, the seismic response of different portions of the structure was studied by means of 3D FE models and non-linear static analyses using the software Abaqus/CAE®.

When dealing with building aggregates attention must be paid on partial out-of-plane collapses and on the possible presence of churches and bell towers, which have a higher seismic vulnerability due to their geometry and plan layout. The presence of adjacent buildings may have beneficial effects, giving adequate confinement to the nave walls, and they may modify the cracks location that activate the collapse of towers. The seismic behaviour of portions exhibiting irregularities shall be deeply studied since their higher vulnerability. The effectiveness of the connections between walls, roof and floors is crucial for the activation of possible local mechanisms in aggregates. The local collapses can be identified first using linear kinematic analysis, and then verified with more accuracy through pushover analyses on FE models.

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