Structures 57 (2023) 105177

Contents lists available at ScienceDirect

Structures

journal homepage: www.elsevier.com/locate/structures

General methodological approach for the seismic assessment of masonry aggregates

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A R T I C L E I N F O

Keywords:
Masonry
Historical building aggregates
Seismic assessment protocols
Advanced numerical analyses

A B S T R A C T

Strong earthquakes have recently shown the vulnerability of masonry structures. In Italy, most of the historical centres are characterized by adjacent masonry structures connected in aggregate that have been subjected to structural and functional changes during time. Their structural behaviour shall be studied to avoid catastrophic outcomes after seismic events. In the technical literature, many studies are available to identify the most vulnerable structures in historical centres from a macroscopic point of view, or the seismic behaviour of some typical structures was studied in detail with sophisticated numerical methods providing 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 structural behaviour under seismic actions of any building aggregate, and that allows to identify effective retrofitting interventions. Moreover, national regulations typically do not provide a standard procedure that can be followed by practitioners for such kind of problems. Thus, the purpose of this paper is to suggest a protocol to be used in common design, based on a broad blend of analyses that can be carried out with commercial software. It relies in a variety of numerical analyses, spanning from simple eigenvalue simulations to full 3D pushover computations, assuming different hypotheses for the material behaviour. The suggested method has been applied and benchmarked to an ex-monastery in northern Italy and several structural considerations to provide a sufficient insight useful for practitioners are provided.

1. Introduction

In Europe, the vulnerability assessment of existing and historical masonry buildings plays a crucial role, since most of the built stock is made by such material. Furthermore, several countries in Southern Europe, where a good amount of the architectural heritage is concentrated (e.g. Croatia, Greece, Italy and Portugal) are earthquake prone areas. In such countries, historical city centres are characterized by buildings clustered in aggregate, often erected in continuity with each other, and generated over the centuries by the progressive transformation of the urban tissue, in which elevation floors were added to existing structures and plan extensions were made by adding structural units. It is possible to define a building aggregate as the composition of those structures, having different features in terms of geometry, materials and dynamic behaviour, which are adjacent or have walls in common (e.g. residential buildings having different characteristics, church and bell tower, church and residential buildings, tower and residential buildings). Typically, they were erected following only rules of thumb, without any anti-seismic criterion. Their vulnerability is expected to be rather high and, considering the implementation of some seismic protection measures, there is the need to put at disposal reliable numerical tools and protocols aimed at quantifying the horizontal acceleration they can withstand, at the same time identifying the most critical parts.

Currently, the research in the field appears quite jeopardized and mainly aimed at the investigation of specific case studies [1–26]. In relation with the recommendations provided by codes of practice, it is interesting to notice that, for instance, the Italian building code does not provide a standard procedure to be followed for the seismic assessment of buildings embedded in urban aggregates. It only highlights some critical aspects that should be accounted for during the analysis, because they are expected to influence the structural response to a great extent. In particular, in [27] and [28] it is specified that a possible interaction coming from structural contiguity with adjacent buildings shall be considered with care. Moreover, the effects of thrusts on common walls of the structural units due to the presence of floors at different heights,

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https://doi.org/10.1016/j.istruc.2023.105177
Received 27 February 2023; Received in revised form 12 July 2023; Accepted 30 August 2023

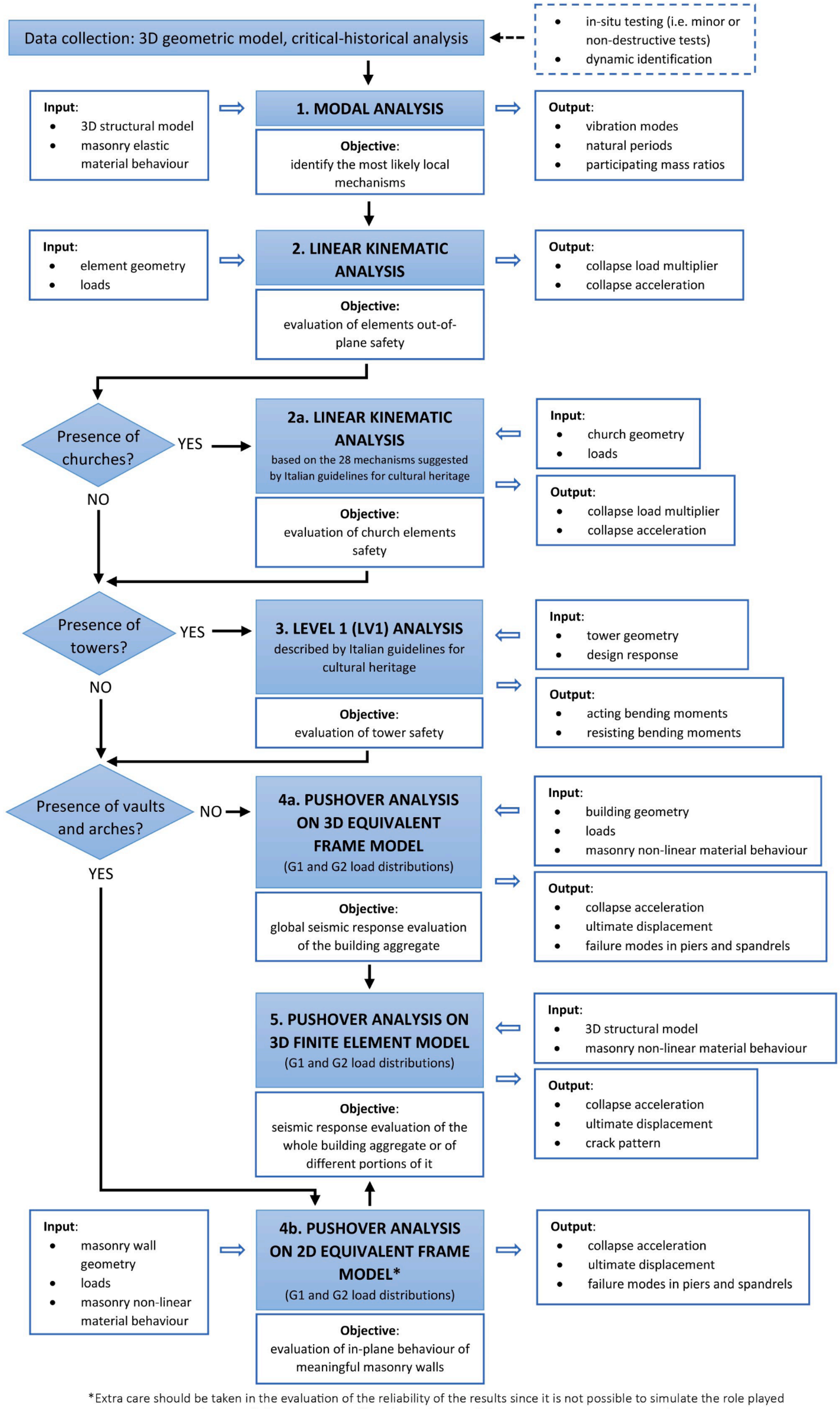


Fig. 1. Flow chart of the protocol to use in the application of the numerical analyses for building aggregates.

local mechanisms activated by the height irregularity and setbacks of the structural units, should be considered with particular care.

It should be also pointed out that the behaviour of a certain aggregate always depends on the materials used, the general and local regularity of the structure, the presence of rigid floors and, most important, the effectiveness of the connections. Moreover, different structural typologies can be found in the same construction because of the changes made during time. For these reasons, the behaviour of a building aggregate is never standard and cannot be a priori known with sufficient accuracy. Consequently, surveys aimed at reaching a sufficient level of knowledge are always critical and mandatory for a correct calibration of any kind of numerical model to use.

The recent specialized literature shows that a methodological approach general enough to be applied in the widest variety of cases is still missing. From a literature survey, it seems that several efforts come from Portuguese and Italian researchers but in many cases the approach adopted is a very advanced full 3D non-linear FEM/DEM modelling, which appears beyond the common usability by practitioners if applied without any other supporting simulation, see e.g. [23,25]. More than one method was proposed for the seismic vulnerability evaluation of historical centres, as the ones developed by Lagomarsino and Giovinazzi and known as "macro-seismic method" and "mechanical method" [29]. Another approach for the evaluation of the seismic behaviour of masonry building aggregates was developed by Formisano et al., [30]. All previous methods are useful to increase awareness on those units of an urban aggregate that most probably require urgent retrofitting interventions, but they do not give information about the possible activation of local or partial collapses, such as the seismic vulnerability and risk assessments carried out in Portugal for the cities of Coimbra [31] and Faro [32]. Thus, specific additional analyses should be carried out to identify more precisely the causes of weakness and implement protection strategies. On the other side, there are approaches that put exclusive emphasis on the vulnerability for the activation of out-of-plane collapses [33–36], but again post-earthquake surveys show that in some cases local mechanisms are not a concern, because the global behaviour is predominant. Many investigations have been carried out for specific case studies, as for instance in the downtown of Lisbon, which is characterized by building aggregates erected following the so-called "Pombaline" system, a sort of assemblage of wood-masonry structural elements, where wooden beams form a cage (Gaiola) for the walls. Ramos and Lourenço proposed a finite element FE analysis based on a commercial general purpose software, which allows both having a certain insight into the most vulnerable buildings of the compound and identifying which kind of strengthening interventions could be adopted [37]. In Italy, a similar construction system has been developed in the XIXth century and is called "Casa Baraccata". Its seismic behaviour has been deeply studied, again with standard methods, being nowadays relatively known [38]. The previous investigations focus on quite peculiar building technologies that cannot be considered the standard, and if we move on the existing research carried out for common aggregates, it is certainly true that there are several other advanced studies available, but all specific and relevant mainly for the examples investigated, see for instance [39].

Another important aspect to consider in the seismic assessment of large building aggregates is the influence of geological conditions. Indeed, the different units belonging to a compound may lay on soils with mechanical properties very heterogeneous, a feature which may amplify ground motion in different ways and cause different damage scenarios, as demonstrated in [40,41].

The complexity of the topic does not allow to draw easily general conclusions, because the role played by contiguous buildings in terms of stiffness, strength and dynamic behaviour turns out to be crucial and rather difficult to understand. Speaking about the interaction with neighbouring buildings, it can be affirmed that the research is still at an embryonic stage and only very specialized investigations have been presented, as for instance that performed in [42], where the seismic behaviour of the Gabbia tower in Mantua (Italy) was considered, with the aim of evaluating the minimum extension of the buildings in aggregate to be modelled in the FE analyses for a realistic prediction of the behaviour under the application of the seismic load.

The general methods of analysis available nowadays for the seismic vulnerability assessment of existing buildings were studied probably for the first time by Calvi et al., [43], but only their advantages and disadvantages were pointed out, whereas a methodological approach able to guide the practitioners step-by-step (in other words a protocol) in a systematic evaluation on the level of reliability and importance of the results obtained in practical applications seems still missing.

Having this target in mind, the present paper is aimed at presenting a general methodology to follow, that can be applied to any masonry compound inserted in a specific urban aggregate, based on well-known standard analyses, for an evaluation – in the most precise way – of the expected seismic vulnerability and the identification of the critical portions of the compound, in light of a retrofitting intervention to implement. It is worth noting that the suggested protocol is characterized by a wide blend of different numerical approaches. Unavoidably, some of them exhibit a certain numerical complexity which cannot be eliminated for the successful application of the protocol. In any case, standard laptops and workstations, nowadays available in common practice, can be used for the safety assessment obtained by the aforementioned protocol and such limitation is not a problem.

The method is discussed with reference to a complex case study, namely the ex-monastery of Santa Maria della Pace in Piacenza, Italy. The choice of the studied building aggregate was done precisely for its general features, in which many different building typologies are present. For this reason, it appears suitable for the application in practice of the proposed operative methodology aimed at the evaluation of the structural behaviour under seismic loads of the single typologies in relation to the building aggregate context. In particular, a wide blend of numerical analyses is proposed, taking care about -for instance- particular interacting scenarios, both in terms of context extension and characteristics of masonry behaviour. The outcome of the study allowed to suggest the methodological approach to follow for an in-depth evaluation of the expected response of the aggregate.

2. The methodological approach: Numerical analyses suggested and protocol

The study of the building aggregate involves the use of different numerical models, both in relation to the dimension of the models themselves (because they can refer either to portions or to the whole building aggregate) and in relation to the type of constitutive behaviour assumed for the materials.

The protocol proposed is summarised in flow-chart depicted in Fig. 1. It is intended for its application on a generic case study and provides (i) numerical analyses to perform and (ii) expected output obtainable, guiding the user in case of presence of churches, vaults and towers.

The characteristics of the numerical models were defined in relation to the type of analysis (linear FEM, non-linear FEM, kinematic, pushover FEM, pushover equivalent frame) and to the objectives of the analyses.

In the modelling phase, it has to be considered that historical structures belonging to aggregates were generally built without considering the effects of seismic events; furthermore most likely they were subjected to functional and structural changes. For these reasons the connections between walls, roof and decks are typically ineffective. Thus, when loaded horizontally they rarely exhibit global behaviour, with a clear activation of local mechanisms and partial collapses. The first analysis that shall be carried out is therefore an elastic eigenfrequency analysis on the finite element model of the entire building aggregate, that allows one to identify – albeit in an approximate way – all the possible local mechanisms and the most probable ones. Once the most probable local mechanisms – associated to a sufficient excited mass – are identified, it is possible to evaluate the collapse load multiplier using the kinematic approach of limit analysis. With this method the out-of-plane behaviour of different macro-elements can be studied. If in the building aggregate there is a church embedded, additional possible local mechanisms among the 28 proposed by the Italian guidelines for cultural heritage [44] should be also evaluated. If towers or bell towers belong to a building aggregate, their seismic behaviour can be evaluated in a simplified way through the LV1 analysis, again proposed by the Italian guidelines for cultural heritage. Once the out-of-plane behaviour has been studied, it is possible to evaluate the in-plane behaviour of meaningful masonry perforated shear panels by means of pushover analyses carried out on equivalent frame models. Moreover, if the geometry of the structure is not too complex (i.e., the irregularities are not diffused), it is also possible to perform pushover analyses on the whole structure modelled using a suitable equivalent frame. Finally, pushover analyses can be performed through full 3D finite element models, which can represent portions of the structure (partial models) or the entire building aggregate (full model).

In the following, the flowchart in Fig. 1 is thoroughly explained step-by-step, underlying the steps' aims and highlighting advantages and drawbacks of each numerical model and of each type of analysis. Before performing any type of analysis, the first step is the data collection; indeed, for existing structures, the critical historical analysis is fundamental and mandatory to reach a good knowledge on geometry, materials, structural changes made over time, existing damage and construction details. In this first step, in-situ testing should preferably and when possible accompany the calculations, to gathering more accurate information about materials and structural response. Minor or non-destructive techniques could be used for in-situ testing according to the degradation level of each structure. The calibration of the models and of the boundary conditions between adjacent buildings could be improved through dynamic identification.

Moreover, such step is crucial to identify pre-existing damage and crack patterns on the structure caused by past events, such as earthquakes. Generally, there are two different approaches to deal with existing damage in historical buildings: (i) assign weakened mechanical properties to the masonry material [45,46] or (ii) detach the nodes in correspondence of the open cracks in order to describe -at least geometrically- the existence of a physical discontinuity inside the masonry structural element [47]. Both approaches have proved to be effective in the evaluation of the seismic behaviour of historical structures, but they are characterized by a certain level of approximation. Indeed, it is impossible for approach (i) to give quantitative indications on the state of damage and area involved by the crack propagation. For approach (ii), the description of the crack is more accurate if a suitable advanced survey is carried out (which cannot be limited to laser scanner or UAVs, because the information inside the walls is lost), but the pre-existent state of stress especially near the crack tip prone to propagation remains unknown. Unfortunately, to cope with such a task would require very demanding numerical simulations (e.g. birth and death finite elements or analyses by phases), which obviously cannot be proposed in standard design.

2.1. Modal analysis

By means of a modal analysis it is possible to identify those modes with high participating mass. Generally for the type of structures under consideration, the participating mass ratio for each mode is very low, meaning that the response of the structure is characterized by local mechanisms. Even if the analysis is performed on an elastic model of the full building aggregate, in which the non-linear behaviour of masonry is not accounted for, still it can provide valuable information on the expected and possible failure mechanisms triggered; the elastic hypothesis for the material is still realistic at least immediately after the application of the seismic excitation, when masonry can be reasonably considered uncracked [48]. In order to obtain reliable results, the 3D model used in the analyses shall be as accurate as possible; if the structure is complex, the 3D model could require a significant amount of time to be meshed. On the contrary, the modelling of the material is very simple, since only specific weight, elastic modulus and Poisson's ratio are required. The advantages are that the analysis is fast and allows to detect all the most probable local mechanisms, that are not a priori known, and consequently save time in the following phases.

2.2. Linear kinematic analysis

Generally, historical masonry buildings are uncapable of developing a global seismic response, mainly because of the bad connections between orthogonal walls, floors and roof; consequently, it is important to preliminary identify characteristic vulnerable elements. Based on the seismic structural behaviour of similar buildings, it is possible to assume which local collapse mechanisms are the most probable. Local collapse mechanisms can be then studied through a kinematic limit analysis. Such procedure is based on the choice of a collapse mechanism and the evaluation of the horizontal action that activates the mechanism by means of the principle of virtual powers.

The analysis considers the structure made of rigid blocks. When rigid blocks are correctly identified, also observing the visible damage on the building, this type of analysis gives in an easy way a reliable estimation of the ultimate resources of the structure. The outcomes of the analysis are strongly influenced by the hypothesis made on the blocks involved in the mechanism and on the connections between walls, roof and floors. It is assumed for masonry a no tension material hypothesis with infinite strength in compression, no deformability of macro-blocks and absence of sliding. Crushing in compression and cracking in tension of masonry are not considered; this could lead to underestimate the capacity of the structural elements [44,49–51].

Computations were carried out by means of the well-known Excel spreadsheet "C.I.N.E.- Condizione d'Instabilità Negli Edifici" [49] provided by the Italian network of seismic engineering university labs (Reluis). The available mechanisms are the simple overturning, the horizontal out-of-plane bending, the vertical out-of-plane bending and the overturning with crack on orthogonal walls. For each local collapse mechanism, the collapse load multiplier α_0 is computed by means of the principle of virtual works, and then the spectral acceleration a_0^* is evaluated according to [27] and [28].

As already pointed out, in masonry building aggregates churches and bell towers may be present. The Italian guidelines for cultural heritage [44] suggest 28 possible local mechanisms, which have been observed with a certain frequency during past earthquakes. The linear kinematic analysis is easier when the mechanisms that can be activated are a-priori known. Even if this procedure is simple and could be used without FE models, it accounts for only in an approximate way the actual geometry of the structure and the actual boundary conditions. Thus, it is always better to compare the results with those collected with an elastic eigenfrequency analysis to be more confident that all the possible local collapse mechanisms are identified. Moreover, the assumption of no tension material for masonry sometimes could be responsible for an underestimation of load carrying capacity, leading to predict collapse accelerations lower than the actual ones [52].

2.3. Level 1 (LV1) analysis

In built aggregates, towers and bell towers are strongly embedded in adjacent buildings. Cracking patterns could be either characterized by damages in the plane of the walls or by rotations restrained by the external constrains, constituted by adjacent buildings or the church. A first estimation of the seismic vulnerability of such towers may be done through a simplified analysis, as the so-called Level 1 LV1 analysis proposed by Italian guidelines for cultural heritage [44]. This analysis is conservative and provides the PGA that triggers the collapse by means of simple computations. The LV1 analysis may be used as a first evaluation method, as it considers only failures due to bending and compression; more refined analyses must follow after.

The behaviour of a tower depends on the slenderness, the degree of connection among walls, masonry quality, the presence of adjacent structures and on the damages caused by the bell vibrations or foundation problems. The tower is regarded as a cantilever, fixed at the base and subjected to self-weight and a system of static horizontal forces, with an inverse linear distribution, mimicking the seismic action; failure may occur crushing in a generic section along the height for mixed bending and compression. The assessment under compression and bending of a slender tower is done comparing the acting bending moment with the ultimate resisting one, evaluated assuming null tensile strength and limited compressive strength for masonry. Such check should be carried along the two principal inertia directions of the section and at different heights, since it is not possible to identify a priori the

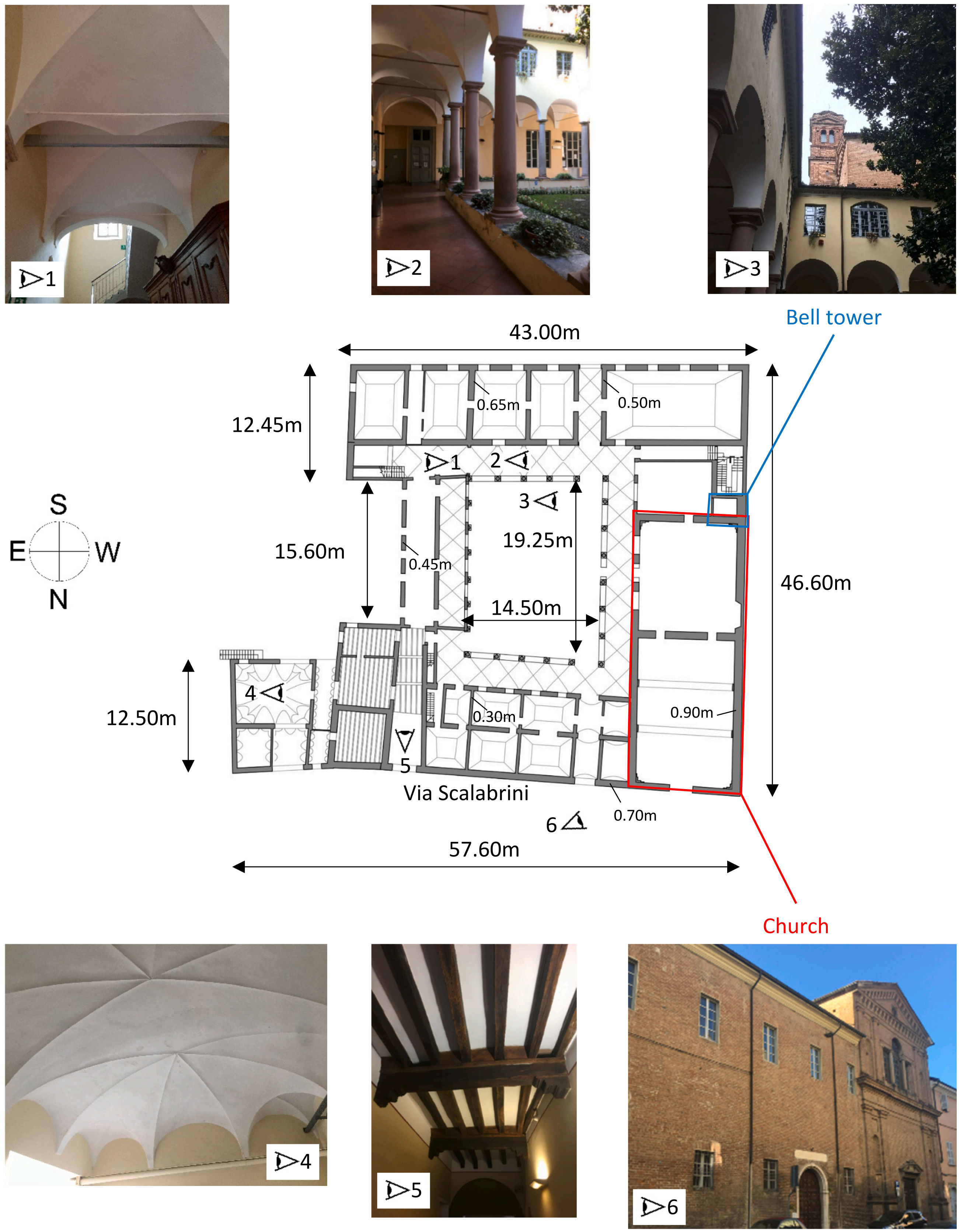


Fig. 2. Plan view and photos on the ground floor.

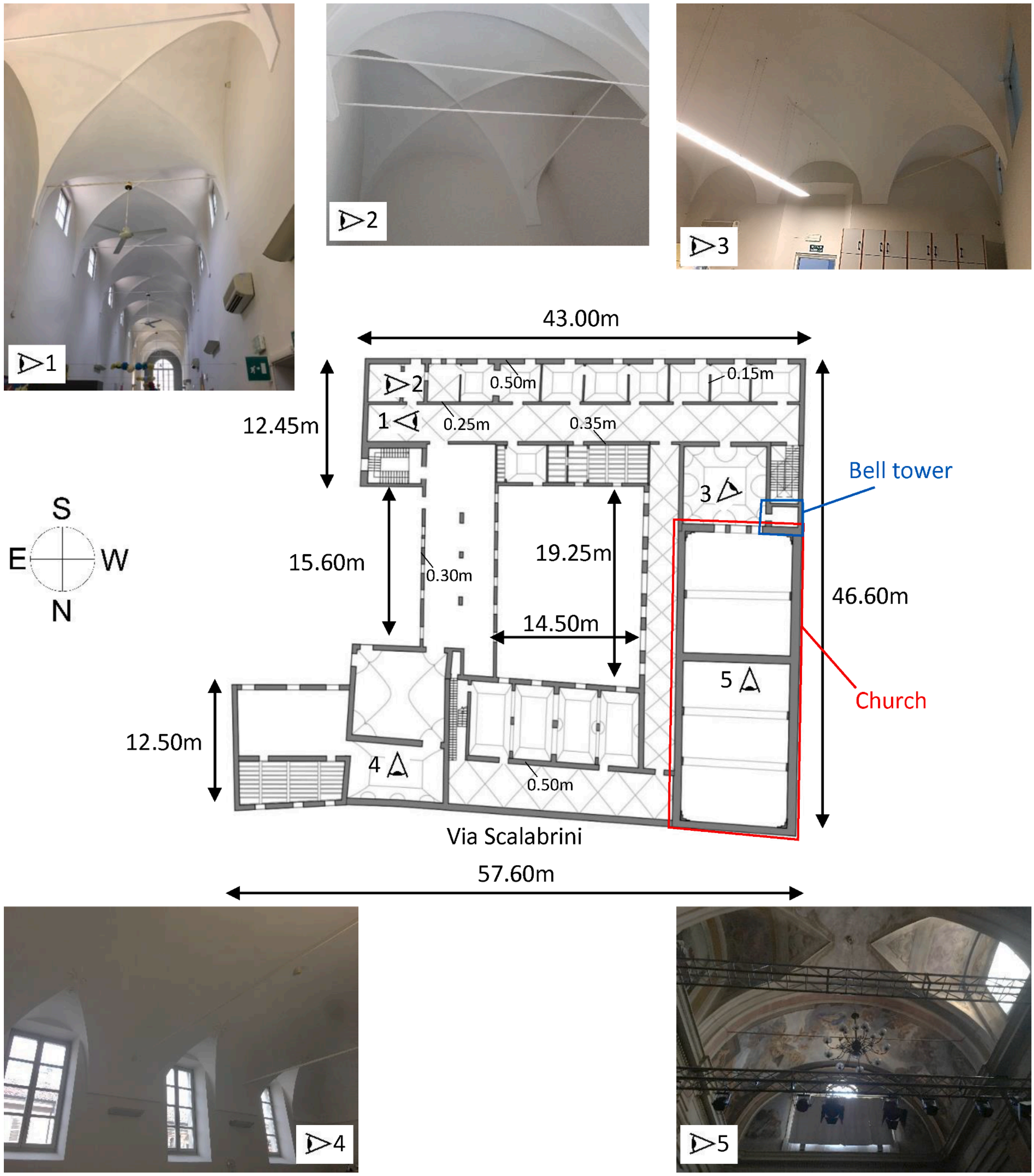


Fig. 3. Plan view and photos on the first floor.

most critical section of the tower, because of the possible presence of openings and tapers in the walls. Thus, the structure is subdivided in n portions along the height with uniform geometric features, and the safety assessment is carried out at each change of the transversal section.

2.4. Pushover analysis on equivalent frame models

Nowadays, pushover analysis represents probably the most accurate tool used by practitioners to evaluate the behaviour of a structure subjected to seismic actions. If the geometry of the structure is not complex (absence of arches and vaults), it is possible to model it by means of the so-called equivalent frame, which is a discretization of assembled piers, spandrels and rigid beams. The elements used are 1D, thus the modelling phase and the computations are faster. A pushover analysis carried out on equivalent frames takes into account the non-linear behaviour of masonry, allows the evaluation of the failure mode inside piers and spandrels and identifies the collapse mechanism of the structure, which is always global. Local failures cannot be detected and the crack pattern developing inside single elements is lost.

When it is necessary to evaluate the global behaviour of complex historical structures, for instance when arches and vaults are present, the results coming from pushover analyses performed on 3D equivalent frame models of the entire structure should be considered with extra-care, since the seismic response cannot be simulated with care using 1D elements. In addition, the floors are frequently deformable and the collapse is governed by local failure mechanisms hardly detectable by

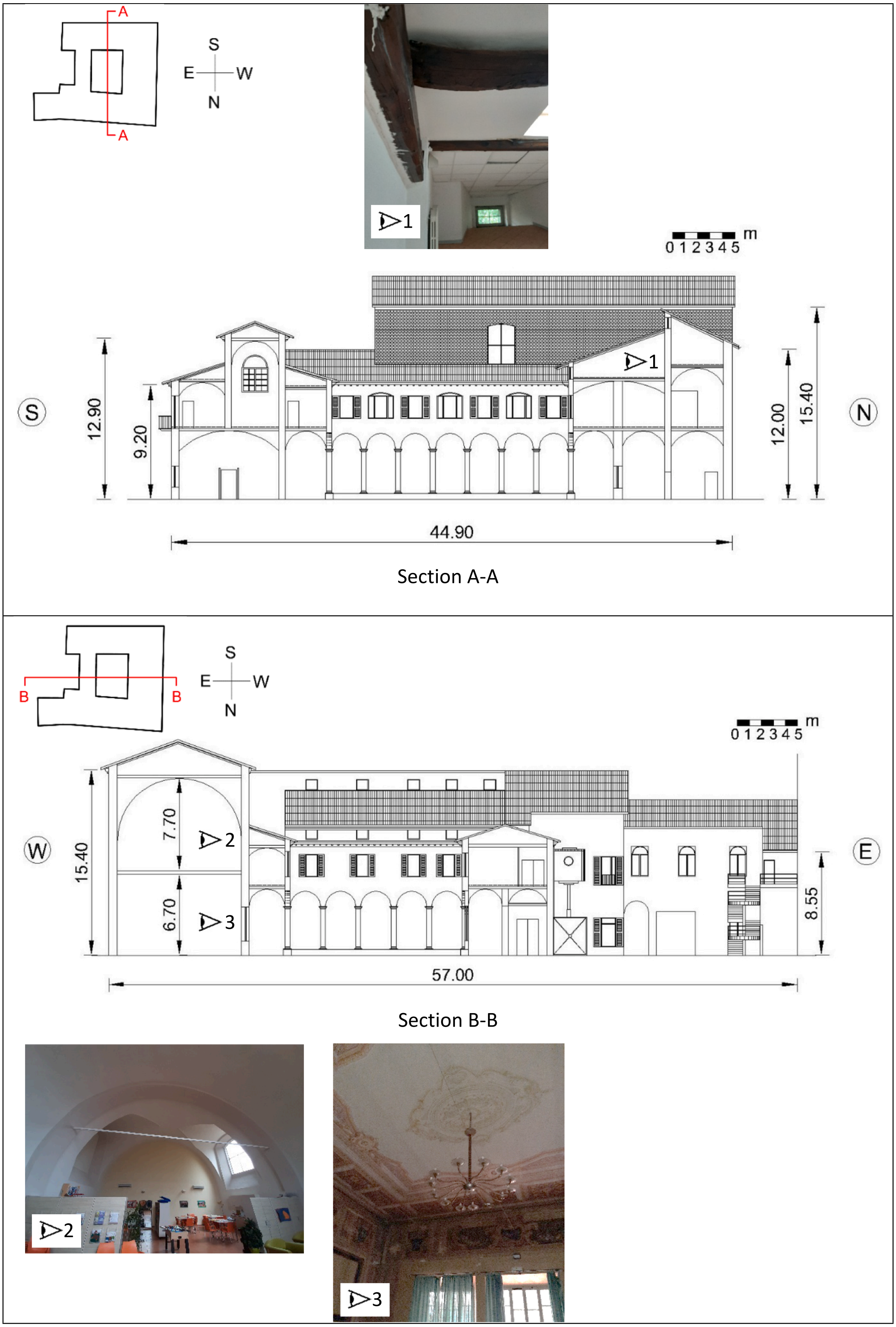


Fig. 4. Sections A-A and B-B of the structure.

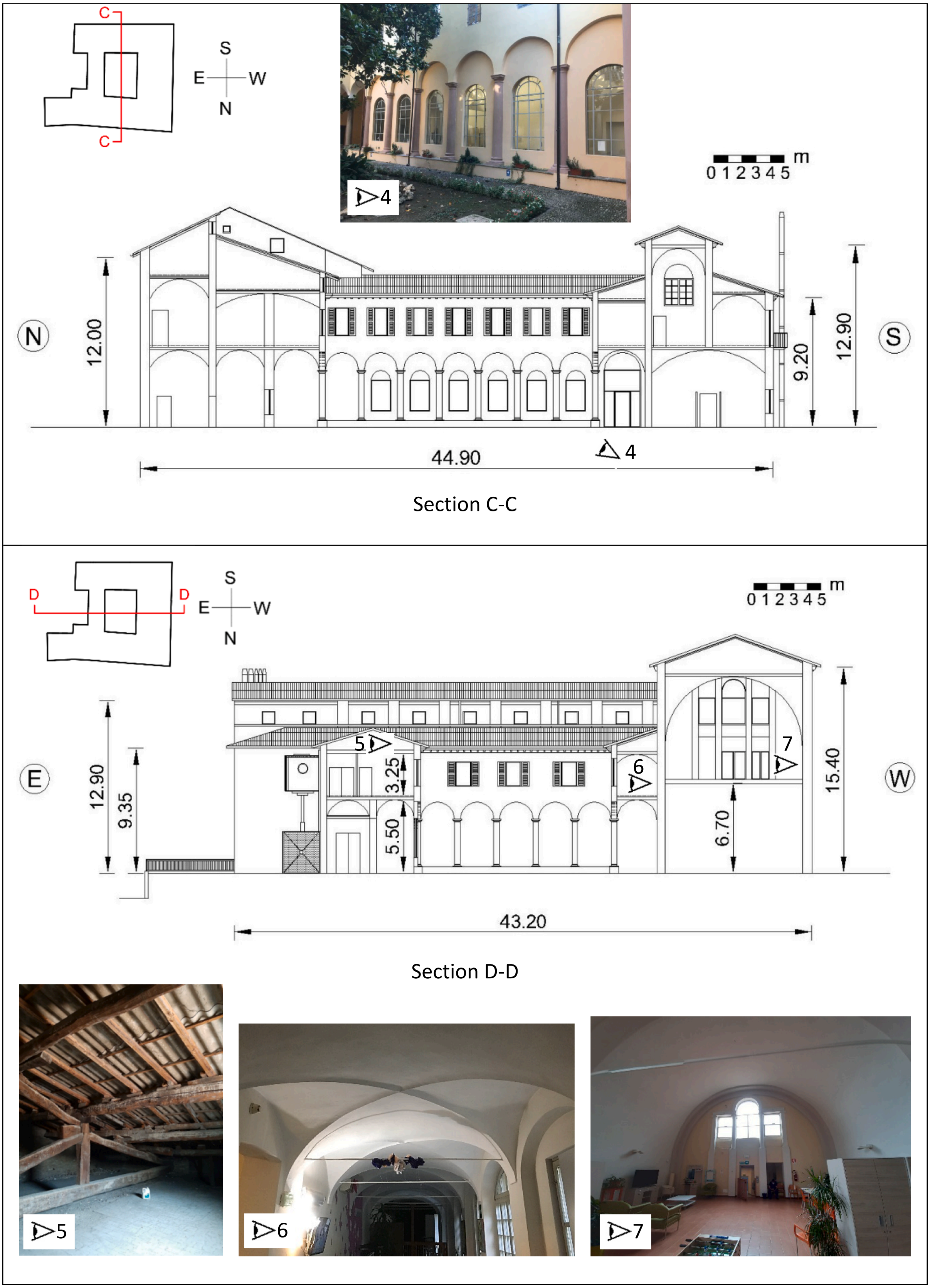


Fig. 5. Sections C-C and D-D of the structure.

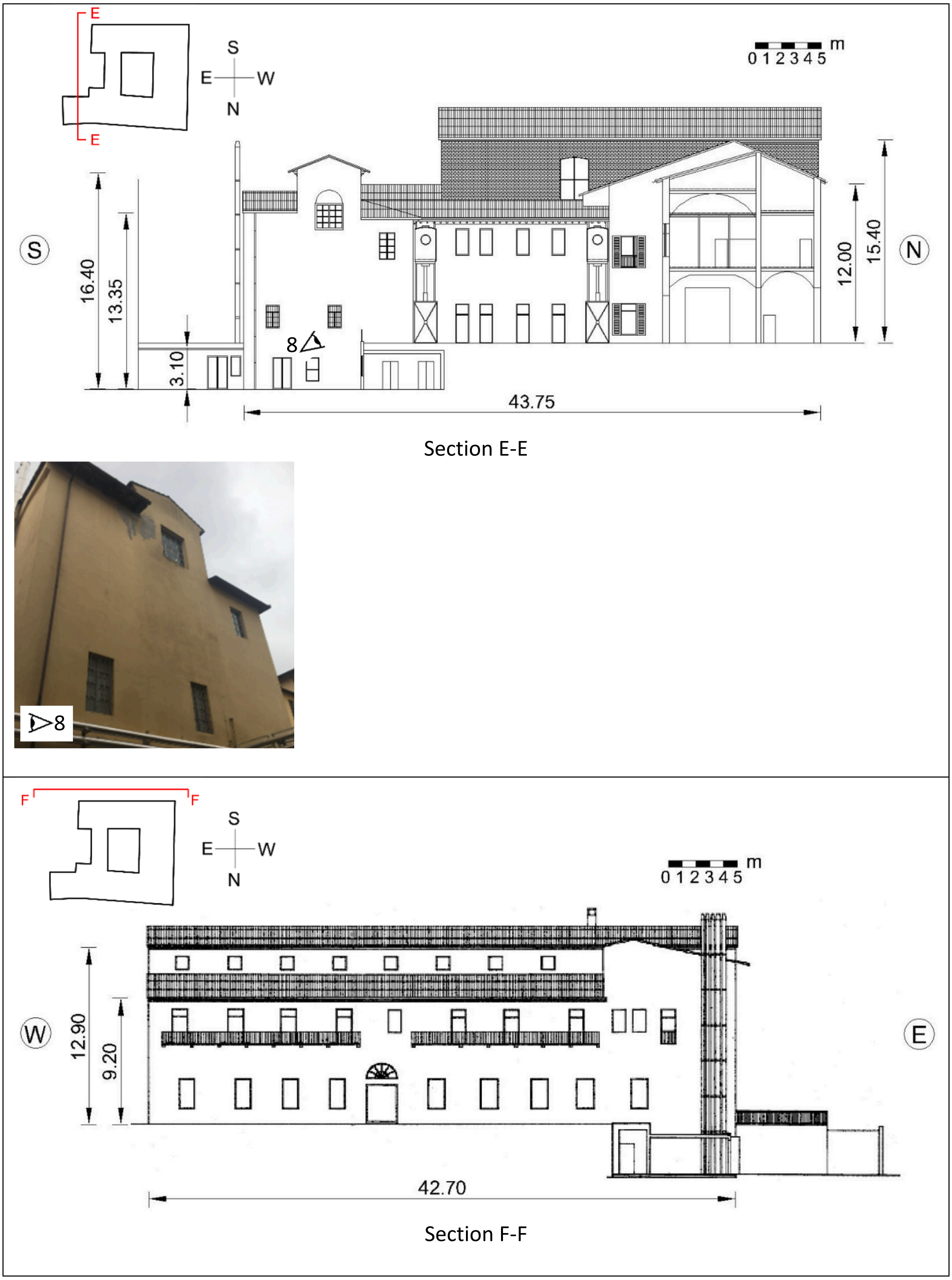


Fig. 6. Sections E-E and F-F of the structure.

this model. This notwithstanding, the in-plane behaviour of significant portions of the building can be predicted with good accuracy. From a pushover analysis performed on single masonry panels, it is also possible to evaluate the ultimate displacement corresponding to the Life Safety limit state (SLV) within a displacement-based assessment. Such analysis could be also useful to identify effective retrofitting interventions with limited computational burden.

In the present paper, pushover analyses were carried out applying the well-known SAM-II method (Simplified Analysis of Masonry buildings), developed by Magenes and Calvi in 1996 for in-plane loaded

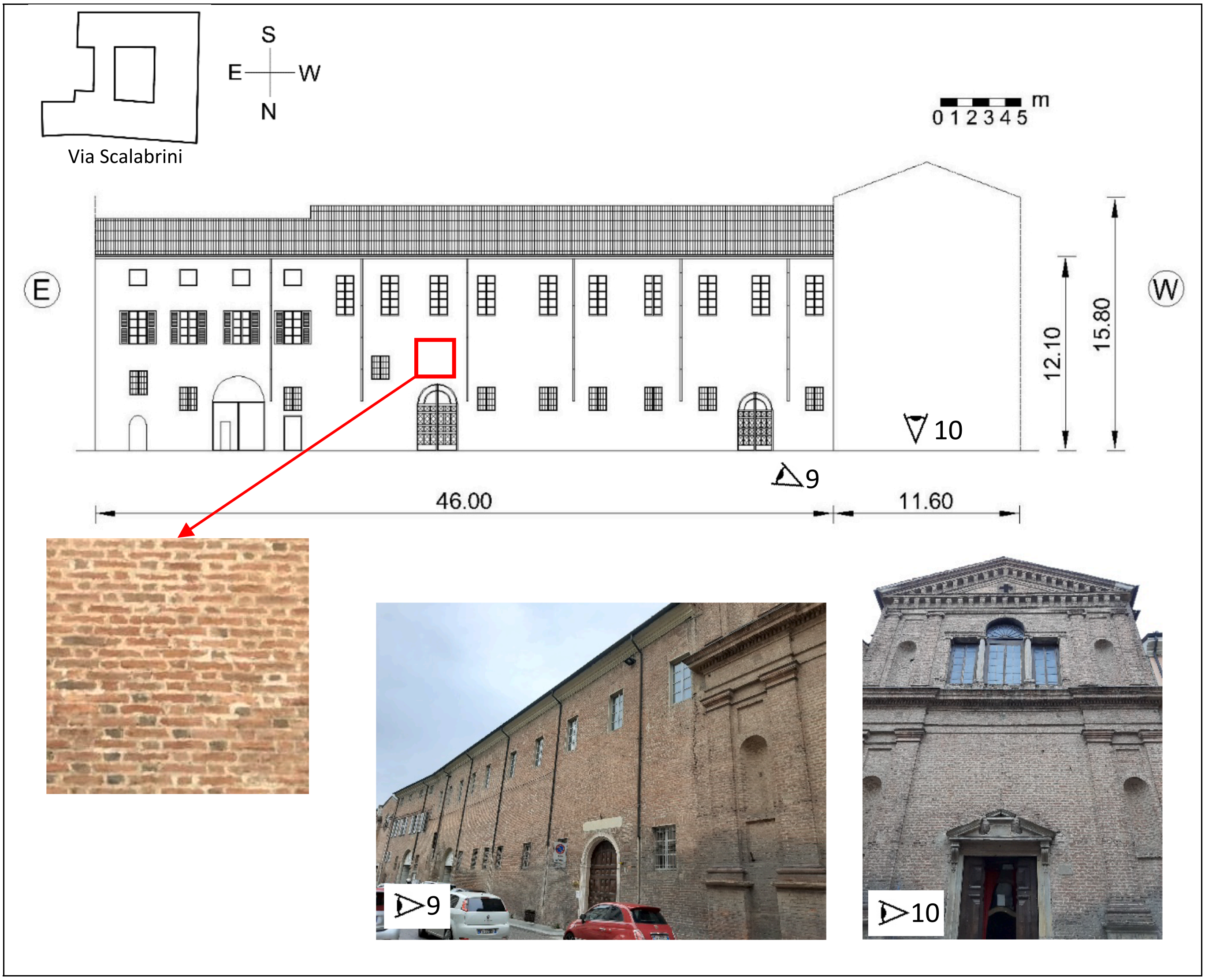


Fig. 7. Façade on Via Scalabrini.

masonry panels and then extended in 2000 for its application to 3D masonry buildings [53]. In such model, a masonry wall is idealized with an equivalent frame, which is composed by pier elements (with vertical axis), spandrel elements (with horizontal axis) and cross joints. Piers and Spandrels are modelled as deformable beams, while the cross joints, assumed infinitely stiff, are modelled with rigid offsets located at the geometric intersection between piers and spandrels [54]. The limitations of the equivalent frame approach are well known and they were discussed for instance in [55]. In order to simulate the plastic behaviour of masonry, concentrated flexural and shear plastic hinges are inserted when either the ultimate bending moment or the ultimate shear are reached [53]. SAM-II is implemented in the commercial software PRO_SAM by 2SI [56].

Thus, when dealing with complex structures, once the most likely out-of-plane failure mechanisms have been evaluated using the linear kinematic analysis, pushover analyses performed on significant masonry panels of the structure, modelled by means of the equivalent frame, allow one to evaluate their in-plane behaviour. It is worth noting that when vaults and arches are present, a discretization with an equivalent frame should be avoided. However, in common practice, the equivalent frame is in some cases the only non-linear Finite Element analysis that can be carried out with the commercial software available. However, the output information provided, albeit approximate, still furnishes a certain insight into the behaviour beyond the elastic limit. By means of these analyses, practitioners have a fast global overview of the seismic response of the structure, that can be deeply studied by means of more refined models and analyses, such as pushover analyses on 3D FE models.

2.5. Pushover analysis on 3D finite element (FE) models

The global behaviour of complex structures, with massive walls, vaults and arches, can be studied by means of pushover analyses performed on full 3D FE models. Using this method, the real geometry of the structure is accounted for and the results may be considered as the most reliable. The disadvantage is that the modelling phase and the computations require much longer time with respect to the equivalent frame; furthermore the user should own both a robust theoretical background, as well as sufficient experience in advanced non-linear FE modelling and in masonry behaviour understanding. Such analysis, indeed, takes into account the non-linear behaviour of the material in multi-axial stress state. One of the results obtainable with this approach is the detailed crack pattern, which is lost in equivalent frames. The crack pattern so obtained can be compared to the actual one (if existing) giving the possibility to understand and confirm the origin of the cracks. 3D FE models allow to monitor the displacements of different points on the

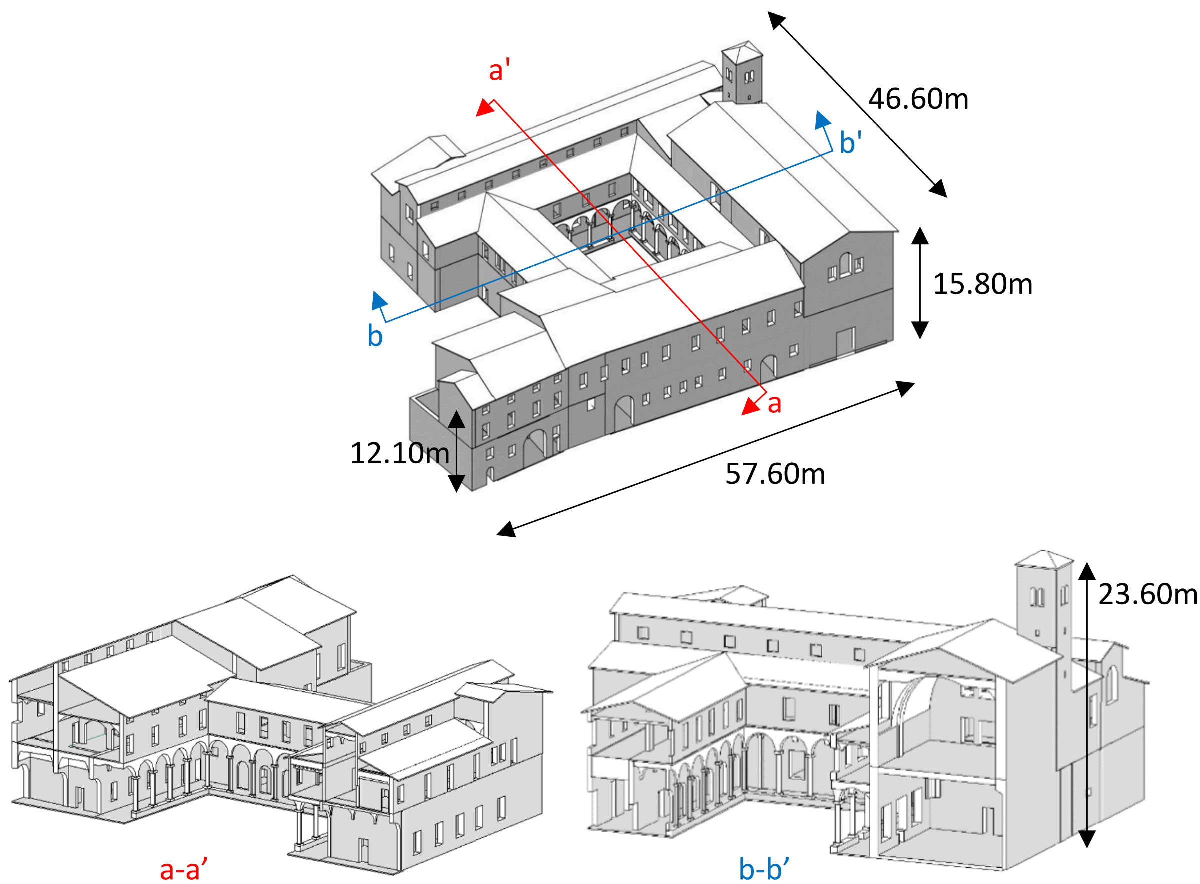


Fig. 8. 3D geometrical model of the ex-monastery.

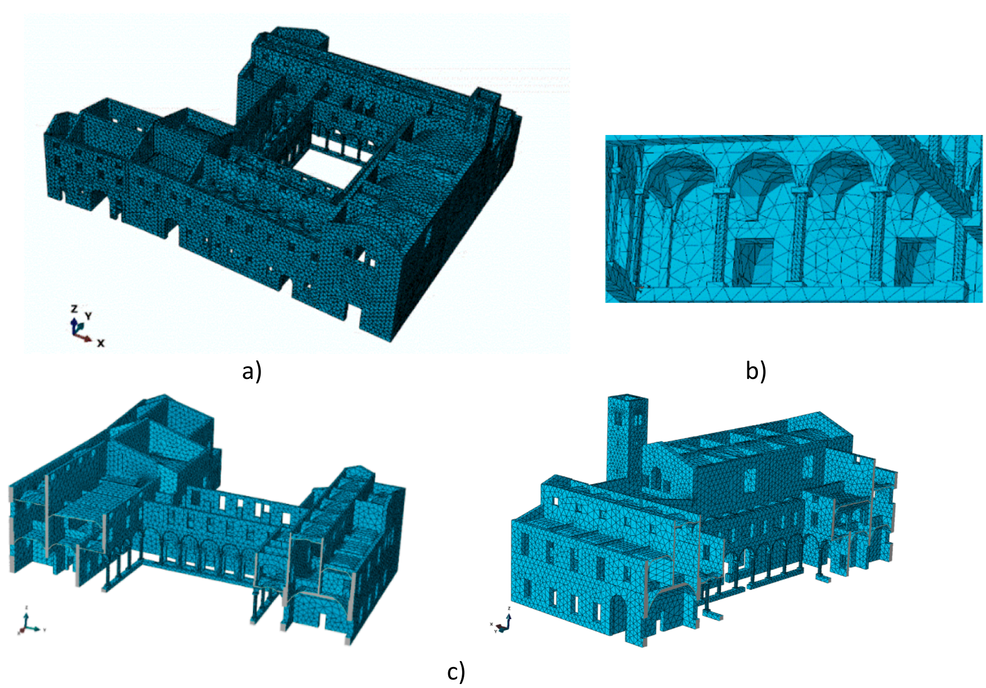


Fig. 9. FE numerical model: a) North-West view, b) and c) mesh details.

structure under increasing values of horizontal loads, an information useful to study triggering of local mechanisms which characterize this type of structures. Here the pushover analyses on 3D FE models were carried out using the software Abaqus/CAE® [57].

3D pushover analysis can be carried out on the whole structure or on significant portions of the building aggregate. It is possible to isolate and

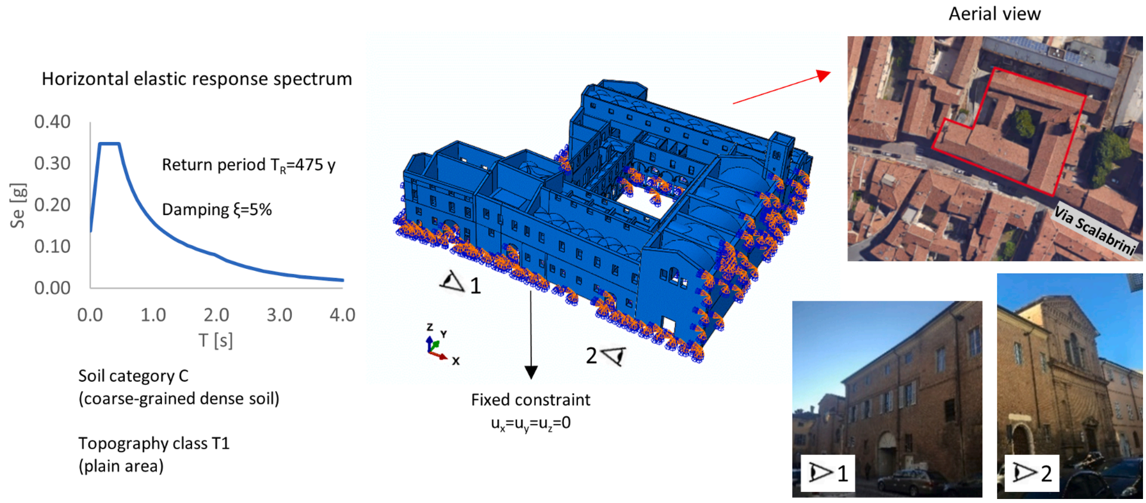


Fig. 10. Elastic response spectrum [71], base and lateral constraints.

Table 1
Masonry mechanical properties.

f [MPa]	τ_0 [MPa]	f_{v0} [MPa]	E [MPa]	G [MPa]	ρ [kN/m^3]	υ [-]
2.0	0.05	0.12	1230	474	18	0.25

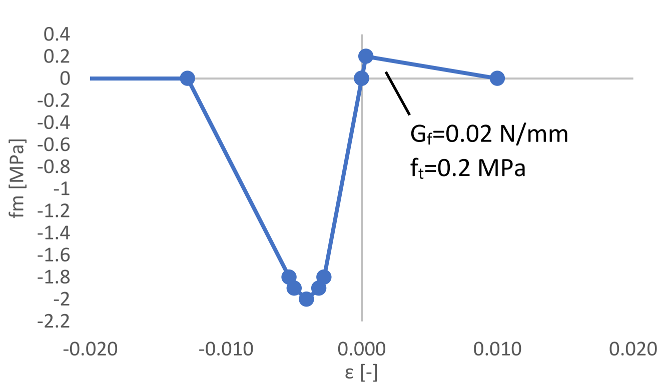
study portions representing the behaviour of a particular part of the compound. In order to create partial models of the building aggregate, first of all, portions having different structural and geometric features shall be identified. It is important to identify the presence of embedded churches, or rooms with large spans, towers and vaults; moreover, portions with discontinuities in elevation can be object of ad-hoc investigations. The results coming from the partial analyses are intuitively conservative, since the interaction among the parts of the building aggregate – which are neglected in partial models – has beneficial effects and the structure can withstand higher horizontal seismic actions, as reported in the literature [30,37,39]. In particular, it is worth mentioning that in the selection of the parts to analyse as isolated, building aggregate portions characterized by vulnerable elements should be studied.

3. Ex monastery of santa maria della pace

3.1. Historical overview

The monastery was built by the Benedictine nuns in the XVI century; during the subsequent centuries several changes were done but the only portion of the original structure that survived till now was not subjected to significant structural changes (Figs. 2–7). The structure is made of masonry, it is characterized by a cloister layout into two levels. The cloister and the corridors of the first floor are covered by cross vaults. The rooms are mainly covered by cloister vaults, sometimes with the presence of lunettes. Few rooms are covered by decks made of timber beams and joists. The roof is pitched and has different heights depending on the geometry in elevation of the different parts of the structure. The bearing structure is made of timber beams and joists and is covered by tiles [58–67].

The structure can be defined as a building aggregate because, besides being confined by adjacent buildings on two sides, there is also the presence of a church and a bell tower located in one wing of the complex. The church has a large single nave covered by a barrel vault with reinforcing arches, while the bell tower is a slender hollow structure. Both the church and the bell tower are embedded in the structure of the monastery.



Kc – Shape factor	0.667
e - Eccentricity	0.1
f_{b0}/f_{c0} – Compressive yield ratio	1.16
ψ [°] – Dilatancy angle	10
μ - Viscosity	0.001

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

Fig. 11. Masonry stress–strain curve.

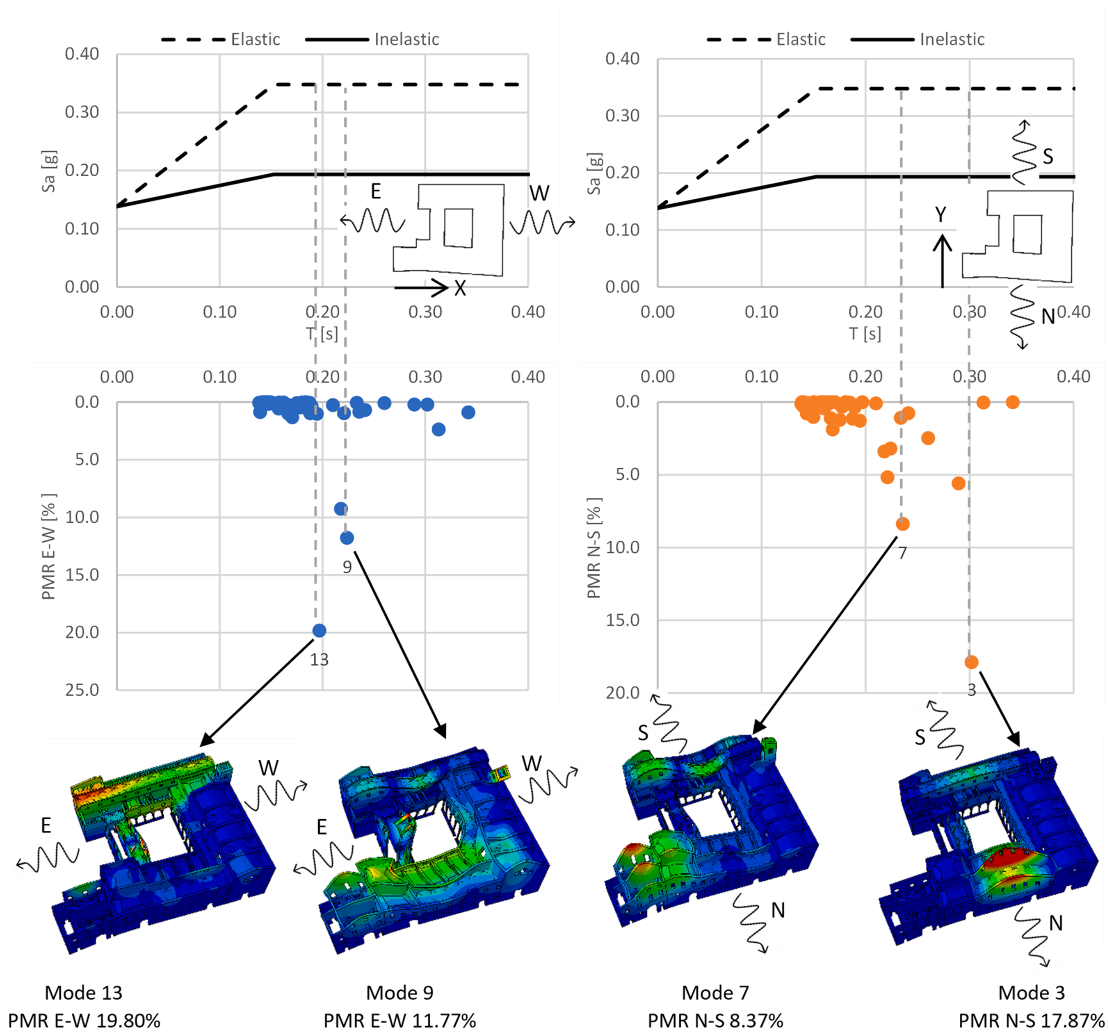


Fig. 12. Eigenvalue analysis results of the first 50 modes.

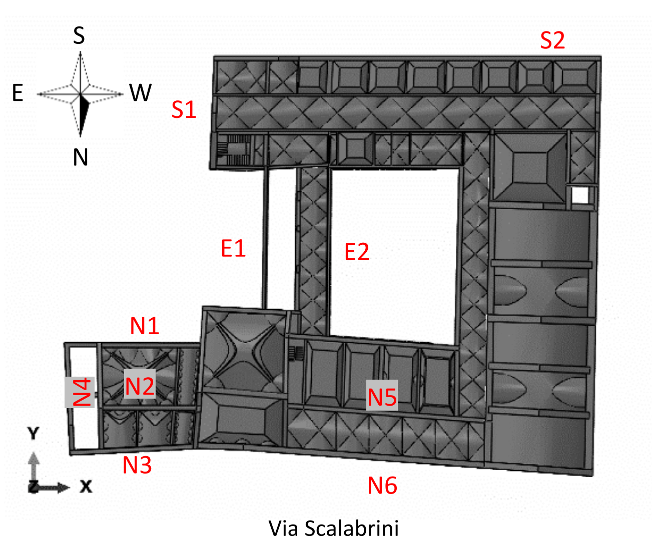


Fig. 13. Walls analysed for the activation of possible local mechanisms.

3.2. Geometrical model

A detailed geometrical model of the building aggregate was obtained starting from the plan views and the sections available, and then refined through measurements done with in place surveys. The 3D geometric model was built within Revit 2022® (Fig. 8), focusing only on structural elements, since the architectural elements, which are those elements contributing to the aesthetic of the structure without influencing its static and dynamic structural response, do not play a significant role in the prediction of the actual structural behaviour.

4. Numerical modelling

4.1. Structural model

In the pre-processing phase, the 3D geometric model was imported from Revit 2022® to the software Abaqus/CAE® [57]. The structure was divided in many portions and then connected through surface-to-surface tie constraints. In this case the roof and the timber decks were modelled as applied loads on the supporting walls. This assumption was made since the roof and the timber decks are characterized by light weight and bad connections with the walls. Moreover, such elements are not stiff enough in their plane, therefore the rigid diaphragm hypothesis cannot hold. For instance, [68–70] adopted a similar strategy for the seismic assessment of complex historical masonry structures. This implies that walls should be considered not constrained at the different heights,

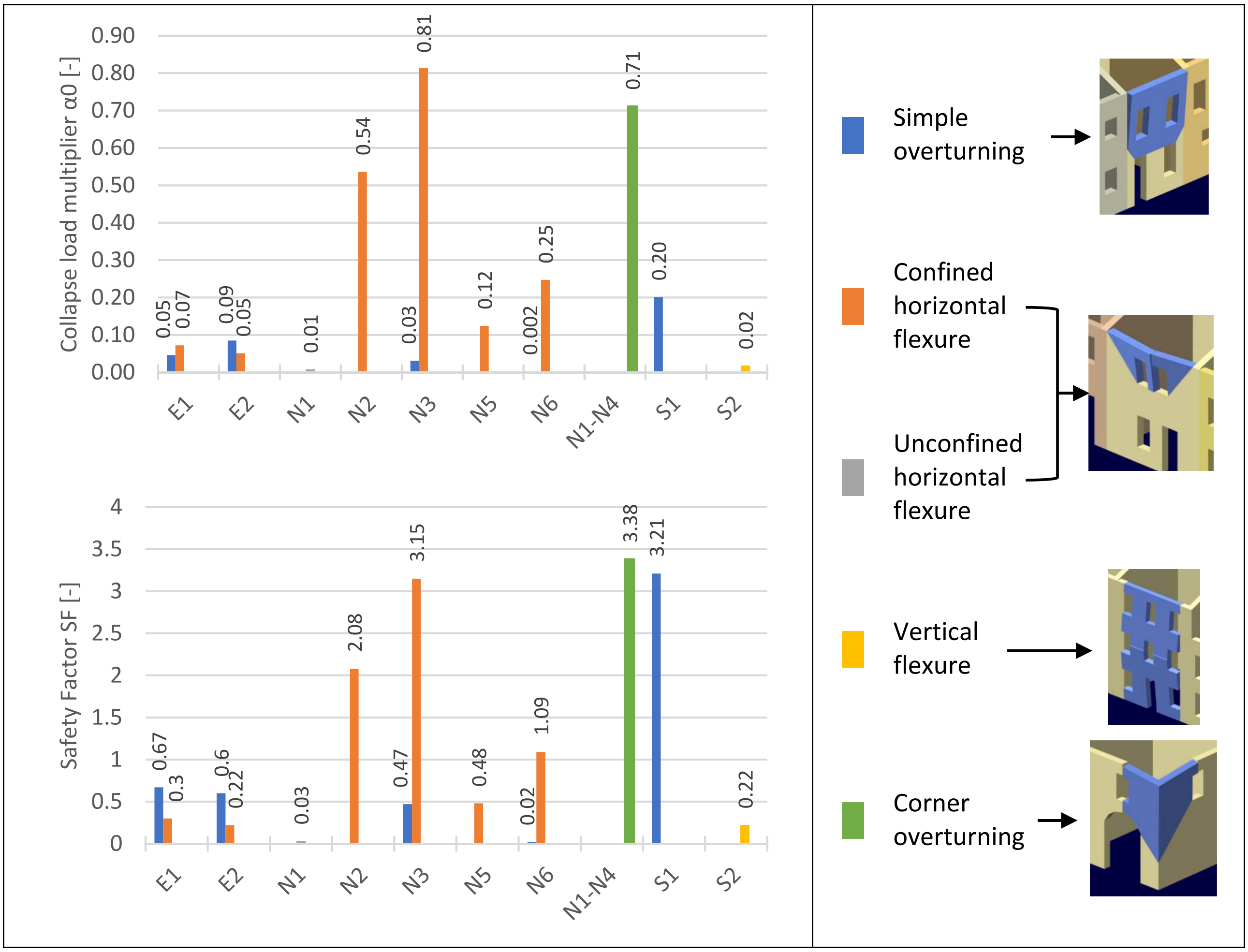


Fig. 14. Linear kinematic analysis results for some selected walls.

resulting slenderer; such hypothesis is in any case on the safe side and it should be also considered that generally in a masonry historical structure the wall-to-floor connection is rather poor. In case of heavy floors and heavy roofs, they should be modelled as mass elements because they surely influence the dynamic response of the building. On the other hand, for the case study considered, the backfill of the vaults was modelled by means of mass elements applied on the vaults. Indeed, it has a significant weight that cannot be disregarded, since the inertia forces are relevant and may influence considerably the dynamic behaviour of the structure.

4.2. Mesh and boundary conditions

The model was discretised by adopting tetrahedral linear elements, see Fig. 9, with a discretization constituted by 310,234 elements, having maximum size equal to 0.80 m and minimum size equal to 0.10 m. Such discretization has been adopted for a first identification of the most vulnerable areas through an elastic eigenfrequency analysis. Then, the mesh was refined for non-linear analyses using as maximum size 0.35 m in order to have at least two elements through the thickness of the bigger walls (the thickness of the wall varies from 0.15 m to 0.90 m as shown in Fig. 2 and Fig. 3, whereas the thickness of the valuts is about 0.12 m), to limit the computational burden and to avoid convergence issues during the analyses. The boundary conditions at soil level were defined using fixed constraints, where rotations and displacements were assumed equal to zero; such assumption is a standard one, because a further insight into the soil-structure interaction would require dedicated ad-hoc considerations, much advanced numerical modelling and detailed in-situ inspection. Fixed constraints, both for rotation and translation, were also used to represent the interaction with adjacent buildings. This latter hypothesis considers the adjacent buildings infinitely stiff, which was a quite reasonable assumption in the case under study since the cluster of buildings is continuous for a long portion of the street and the adjacent structures share common walls with the building aggregate portion analysed, see Fig. 10.

In general, fixed constraints can be adopted when the structural unit analysed is confined by adjacent structures that extend for several tens of meters and they share the same perimetral walls. If structural units perimetral walls are in contact but are not shared by the two structures, constraints given by elastic springs must be adopted; moreover, if the dynamic behaviours of the structures in contact are different (e.g., different construction materials, different geometries) the pounding effect must be considered. Also when the adjacent structures do not extend for a sufficient length, which at least could correspond to one half of the length and width of the building, and consequently cannot provide fixed constraints, elastic springs shall be used as constraints. Elastic springs shall be accurately calibrated each time. As far as the seismic spectrum adopted is concerned, see Fig. 9, in general we referred to the indication provided by the Italian seismic code, but particular attention was also paid to the local seismicity of the Piacenza municipality [71].

4.3. Material properties

The structure is mainly made of masonry. The information retrieved

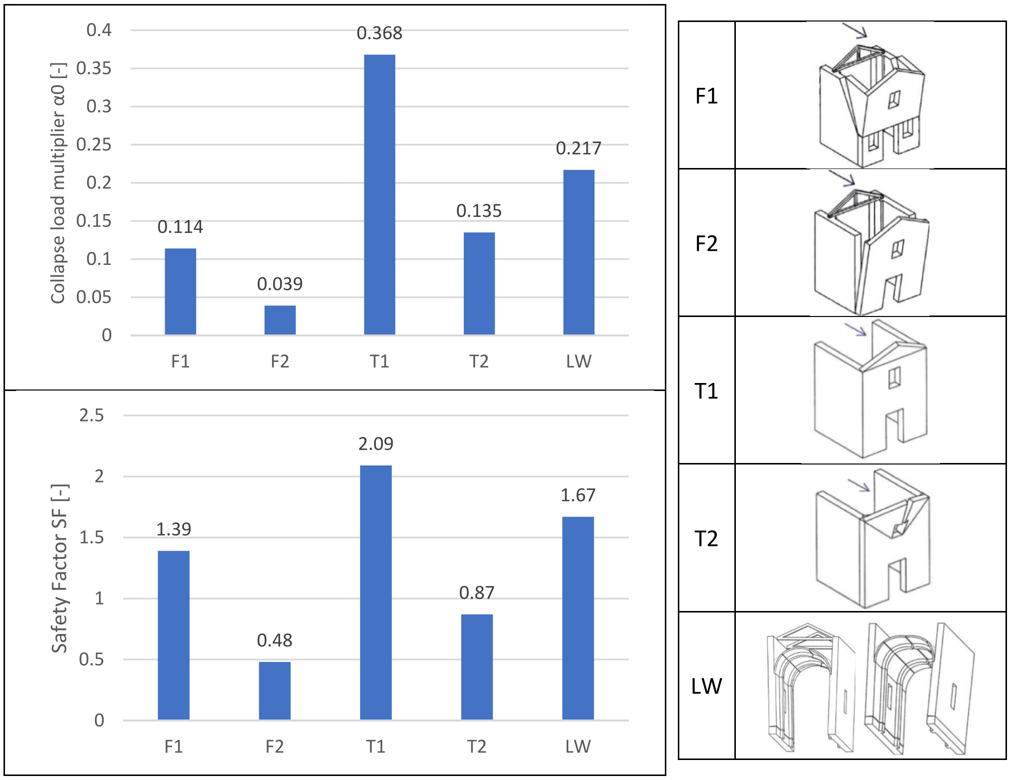


Fig. 15. Linear kinematic analysis results for the church.

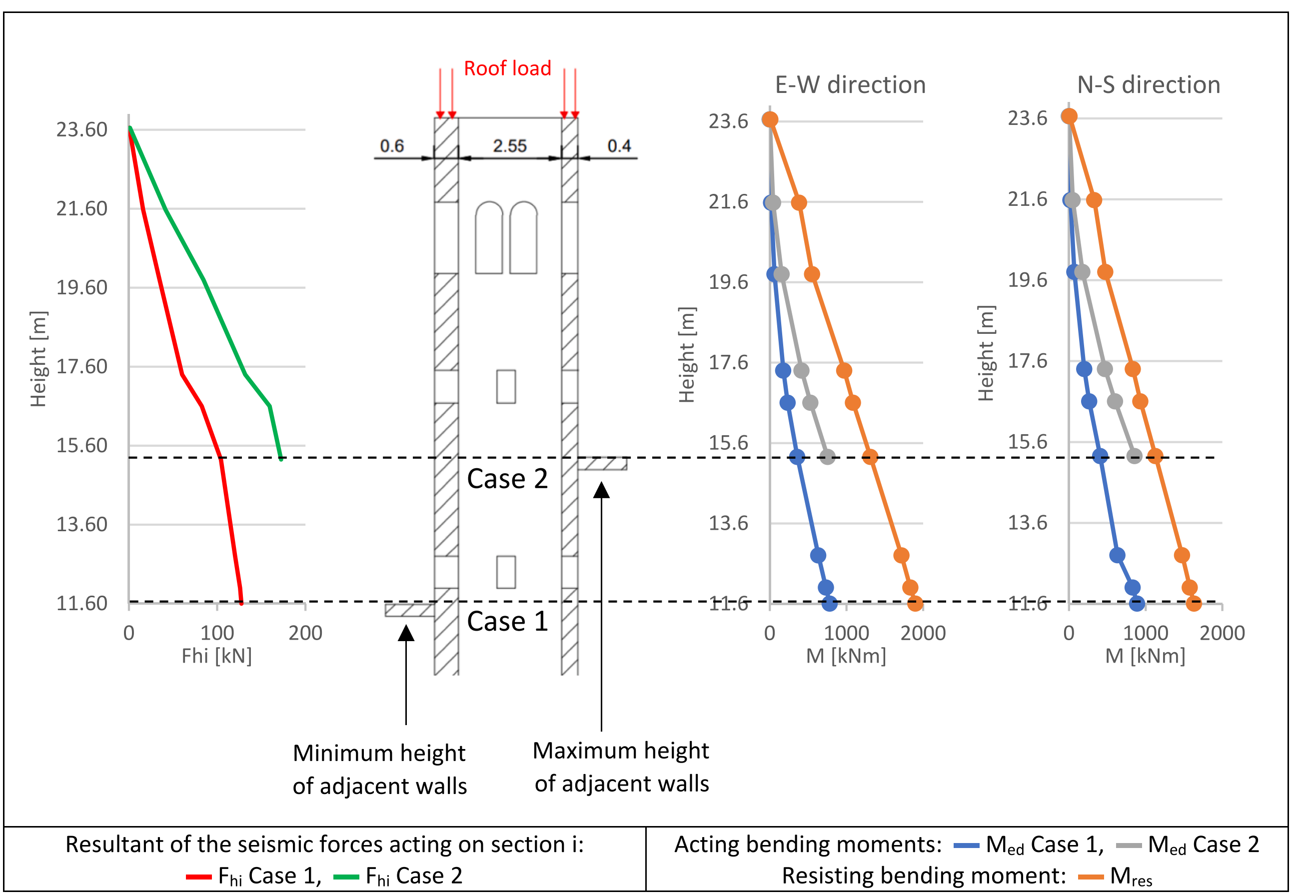
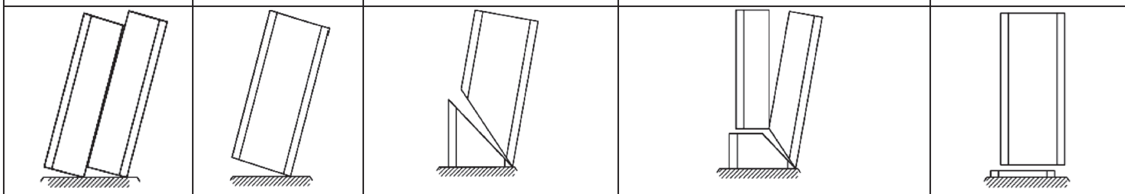


Fig. 16. LV1 analysis results.

Table 2
Collapse load multipliers of the bell tower.

	#1	#2	#3	#4	#5
Mechanism					
	rocking with vertical splitting	monolithic rocking	Heyman's diagonal cracking and rocking	mixed Heyman's mechanism with vertical splitting	base shear sliding
Case 1	0.503	0.629	0.245	0.148	0.444
Case 2	0.865	1.171	0.300	0.221	0.574

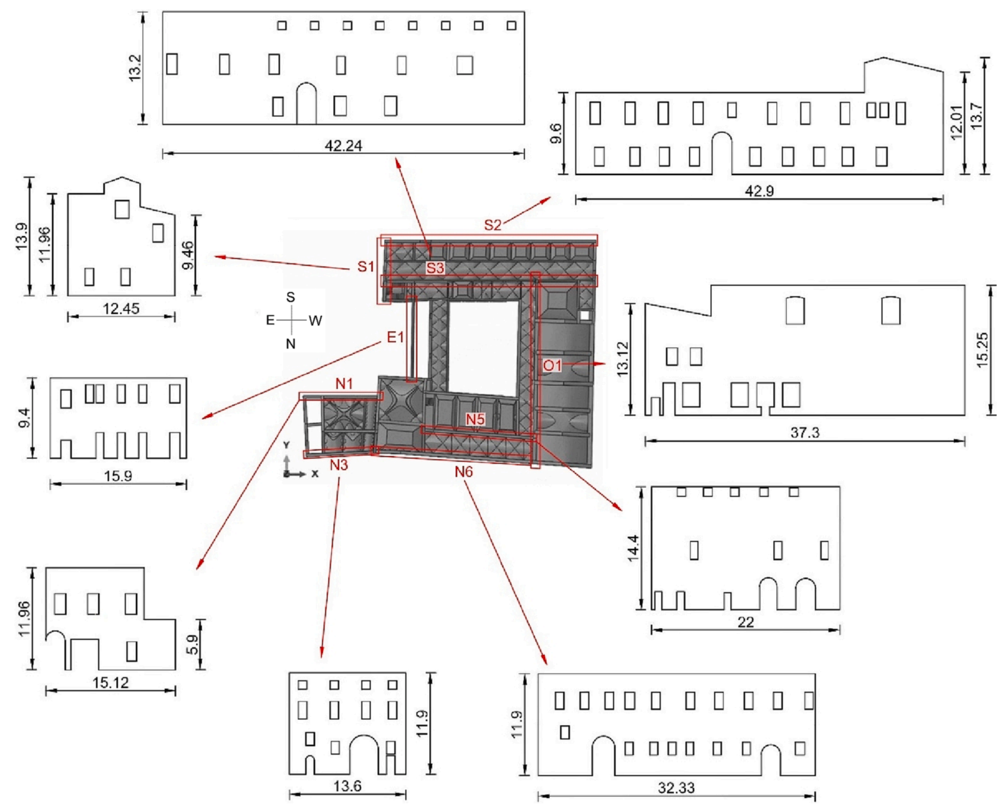


Fig. 17. Masonry panels studied in-plane with the equivalent frame approach (dimensions in meters).

about the original masonry was limited, thus according to [27] and [28] the mechanical characteristics were deduced using the Masonry Quality Index (MQI) method, proposed by Borri and De Maria [72]. The MQI method leads to the evaluation of a numerical index of the masonry quality (IQM). The numerical value of the index depends on the observance of some conditions related to the good construction practice, such as the presence of regular horizontal rows of bricks and transversal interlocking, the element shape and size, the lack of alignment of vertical joints and the mortar quality. From the IQM it is possible to define with a certain probabilistic accuracy the mechanical parameters of masonry.

For the case under study, masonry is made by regular solid clay bricks, with the alternance of stretchers and diatons and the lack of alignment for vertical joints (Fig. 7). To define the mechanical properties of masonry, the level of knowledge reached for the materials shall be accounted for. In this case the level of knowledge was LC1, which is the lowest for the Italian code, since only a visual inspection was carried out. According to [28], the minimum value in the range suggested by the MQI method must be considered for what concerns the strength, while the mean value is used for the elastic modulus. According to the previous approach, masonry elastic properties, used for linear elastic analyses, were defined considering the safety factor associated to LC1 equal to 1.35, and are listed in Table 1.

Having in mind to perform non-linear analyses, it was necessary to define the non-linear behaviour of the material. The Concrete Damage Plasticity (CDP) model, available in standard versions of Abaqus/CAE® [73,74], is relatively suitable in the case of masonry. This model, proposed by Lubliner et al., [75] and modified later by Lee and Fenves [76], was originally conceived to describe the non-linear behaviour of concrete, but it proved also its suitability in the prediction of the non-linear computations of quasi-brittle materials such as rocks, mortar, masonry and ceramics. It was shown that the CDP model can be adopted for the

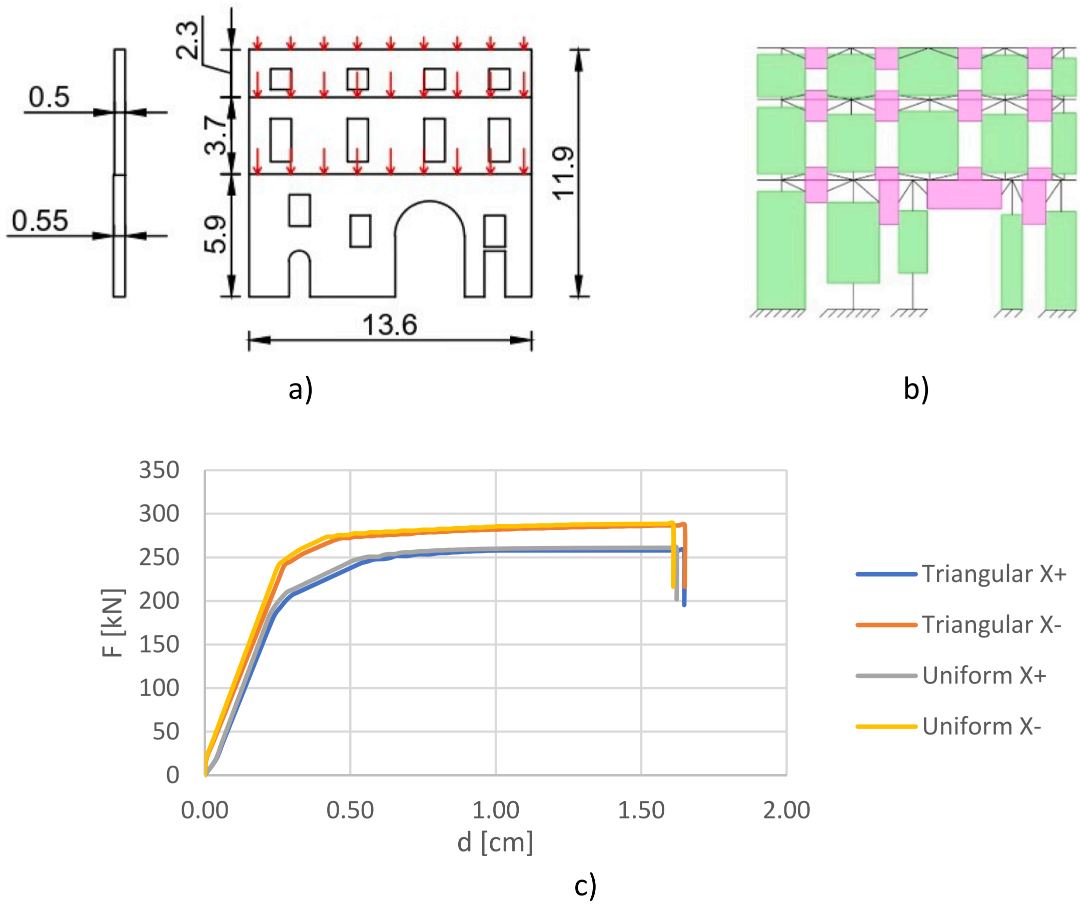


Fig. 18. Wall N3: a) geometry dimensions in meters, b) equivalent frame model, c) capacity curves.

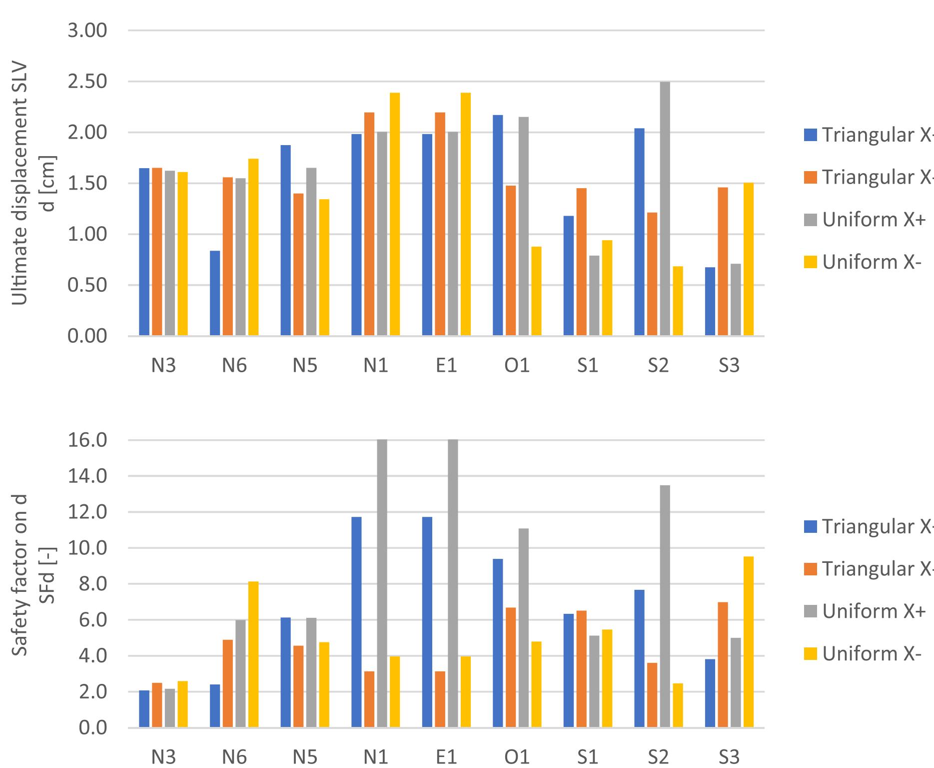
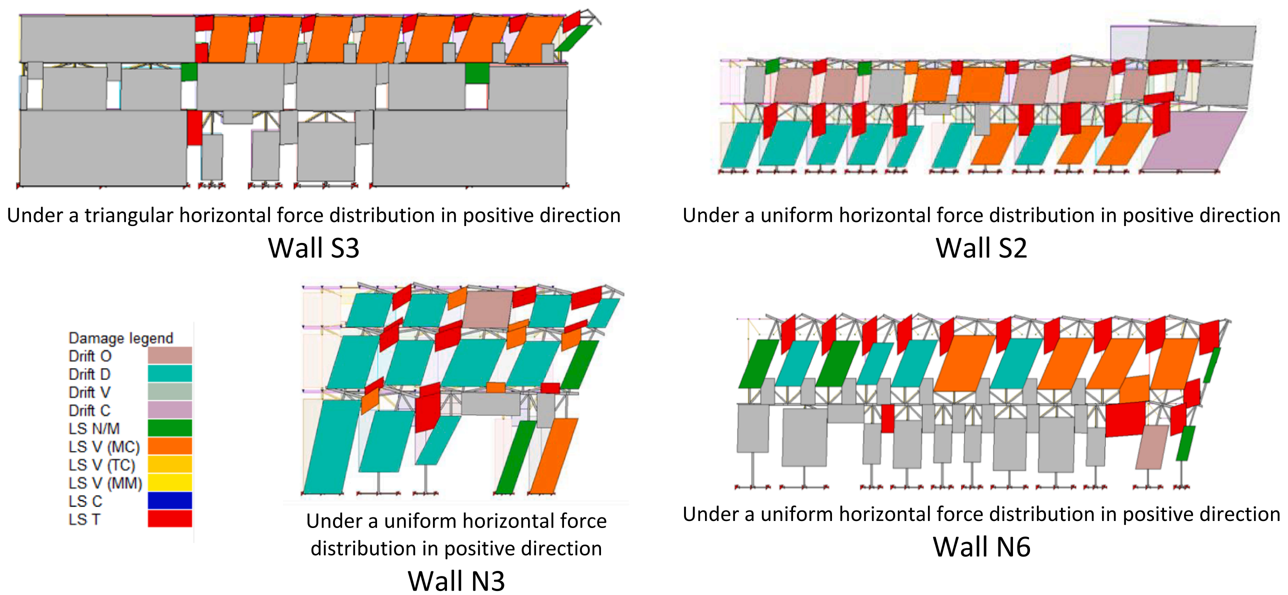


Fig. 19. Pushover analysis on equivalent frame models results in terms of ultimate displacement and safety factor.



Drift O = Overcoming of the inter-storey drift at the operability limit state (SLO), Drift D = Overcoming of the inter-storey drift at the damage limit state (SLD), Drift V = Overcoming of the inter-storey drift at the life safety limit state (SLV), Drift C = Overcoming of the inter-storey drift at the collapse limit state (SLC), LS N/M = Failure in bending and compression, LS V (MC) = Shear sliding failure (Mohr-Coulomb criterion), LS V (TC) = Diagonal cracking failure (Turnsek-Cacovic criterion), LS V (MM) = Diagonal cracking failure (Mann-Müller criterion), LS C = Failure in compression, LS T = Failure in tension

Fig. 20. Failure mechanisms of significant masonry panels.

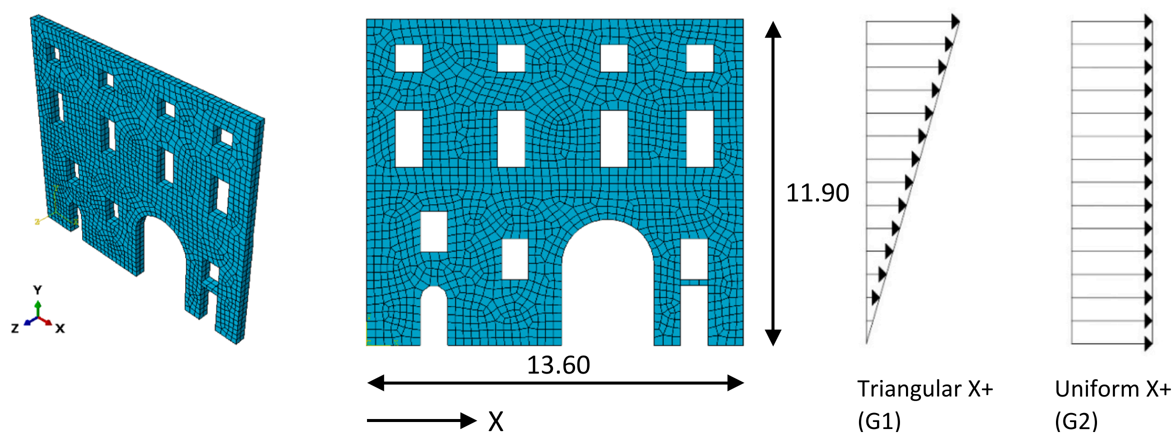


Fig. 21. Wall N3: 3D geometry and mesh.

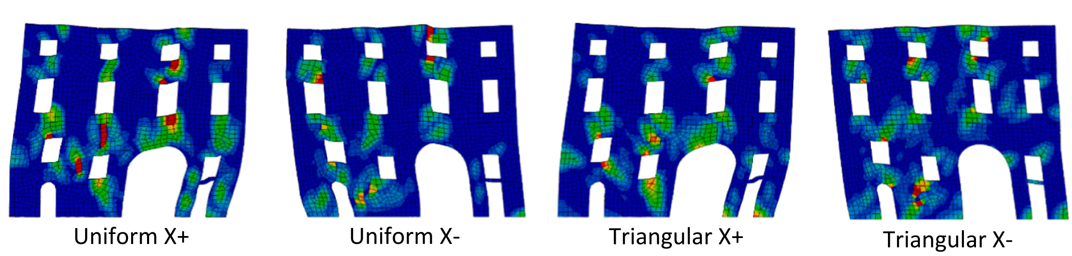


Fig. 22. Wall N3 pushover analysis results: equivalent plastic strain distribution (representing in the model the crack pattern) at collapse obtained with the 3D model.

analysis of masonry structures under cyclic or dynamic loadings [77]. The main failures that can be reproduced are cracking in tension and crushing in compression. CDP is a continuum plasticity-based model, with independent tensile and compressive damage variables and strengths. The parameters used to calibrate the CDP model are listed in Fig. 11; they were taken according to the results found in the technical literature [42,78–82]. Apart from the mechanical parameters to set, the viscosity parameter is probably the most important and controversial;

Table 3
Wall N3: pushover analysis results.

	Full 3D		Equivalent Frame	
	Abaqus/CAE		PRO_SAM/SAM II	
Load distribution	a_g [g]	d [cm]	a_g [g]	d [cm]
Uniform X+	0.23	2.07	0.24	1.62
Uniform X-	0.28	2.01	0.24	1.61
Triangular X+	0.05	2.06	0.24	1.65
Triangular X-	0.06	2.01	0.24	1.65

indeed, the smaller is its value the more accurate is the output obtained. However, when it drops down, the computational burden increases exponentially and premature halting for lack of convergence occurs with higher frequency. The value chosen for the analyses, which is in agreement with existing literature, is a good compromise between the accuracy of the results and the computational stability. Furthermore, authors experienced – training on some sub-models presented in this paper – that the global pushover curves did not change significantly dropping down by an order of magnitude the viscosity from the selected value. Abaqus/CAE® requires to define the stress–strain diagram in the post-elastic domain in compression, Fig. 11 and [83], while the behaviour of masonry in tension has been defined through the fracture energy. Some columns in the model (e.g. those of the cloister) are made of granite: they were modelled with an elastic material (Young's modulus 5000 MPa, Poisson's ratio 0.25, density 27 kN/m^3) also in the non-linear analyses since their strength is much higher compared to the masonry one.

5. Numerical results for the case study analysed

5.1. Modal analysis

Modal analysis was carried out on the FE model described before, where masonry was modelled as an elastic material (Table 1) and the stiffness of the vaults was considered 1/4 of that of the masonry walls, in agreement with what is stated in [44]. The most relevant natural frequencies and the participating mass ratios (PMR) in the North-South and East-West directions are shown in Fig. 12. Since masonry historical buildings generally are conceived only to withstand vertical actions, it was assumed that connections with floors and roof are not effective, modelling them only with applied loads on the walls, consequently only local mechanisms were found. Only the modes with participating mass ratio higher than 1% have been taken into consideration in Fig. 12.

From Fig. 12, it is possible to notice that significant modes exhibit periods falling on the plateau of the design spectrum of accelerations, defined assuming a behaviour factor equal to q = 2.25 according to the value suggested by Italian guidelines for cultural heritage for irregular structures [44]. This means that all those portions of the structure involved in the active mode will be subjected to the highest acceleration, a condition that leads one to think that the activation of a mechanism is most likely. This is true at least when masonry is still in the elastic range, immediately after the application of the seismic action.

5.2. Linear kinematic analysis

Modal analysis supplied modal shapes with a significant percentage of excited mass, suggesting possible local collapse mechanisms that can be activated. It is therefore possible to evaluate the collapse acceleration using limit analysis. In Fig. 13 the walls analysed with C.I.N.E. are shown.

In the eastern portion of the structure, eigenvalue analysis suggested to check confined horizontal flexure of walls E1 and E2. Moreover, it was considered also the possibility of simple overturning, because this portion of the structure was added only in the 20th century and the connection between orthogonal walls is presumed to be not effective. In the northern portion of the building, modal analysis highlighted confined horizontal flexure also for walls N2, N3, N5, N6, while wall N1 could be subjected to unconfined horizontal flexure since it belongs to an angular portion of the building. The walls facing via Scalabrini, N3 and N6, were studied also under simple overturning. Moreover, the intersection of walls N1 and N4 is most likely a critical portion of the building since it is a free corner, consequently corner overturning can be also activated. In the southern part of the structure, the critical portions are the free façade (S1) and the portion of wall in correspondence of the big vaulted room at the ground level (S2). The first one has been studied for simple overturning, while the second one for vertical flexure. Results are shown in Fig. 14 in terms of collapse load multiplier (α_0) and safety factor (SF). The safety factor is defined by the ratio between the spectral acceleration needed for the activation of the mechanism and the design spectral acceleration at the site. When the safety factor is smaller than 1, the structure is unsafe.

The collapse load multiplier for simple overturning of walls of limited thickness (such as E1, E2, N3 and N6) is very low, as expected. Wall S1 is characterized by a higher collapse load multiplier because of the presence of tie rods, which constrain the overturning of the wall. From the analysis it can be deduced that the simple overturning for aggregates with walls of limited thickness is a critical point. For this

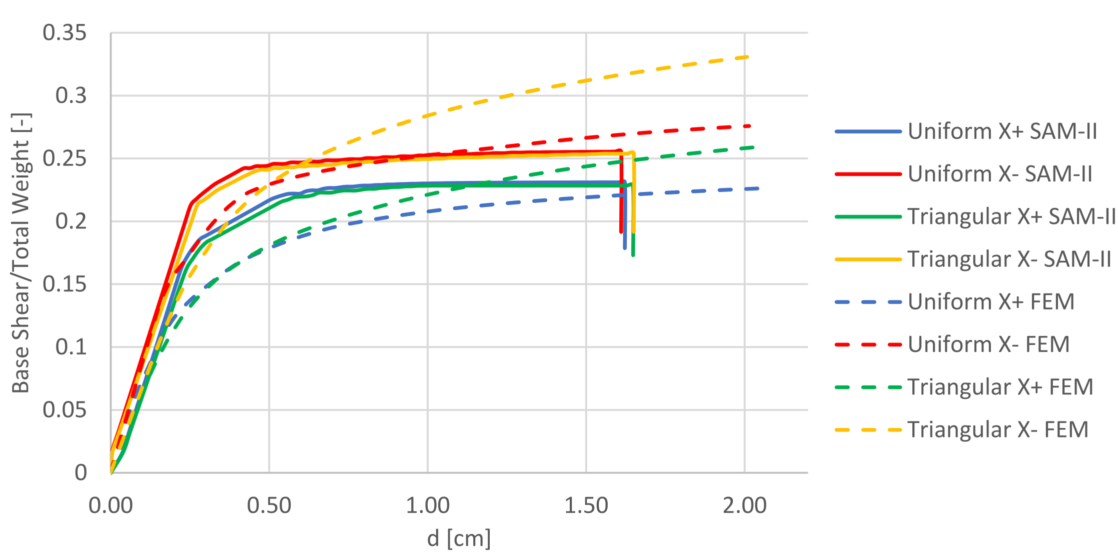


Fig. 23. Wall N3: normalized capacity curves.

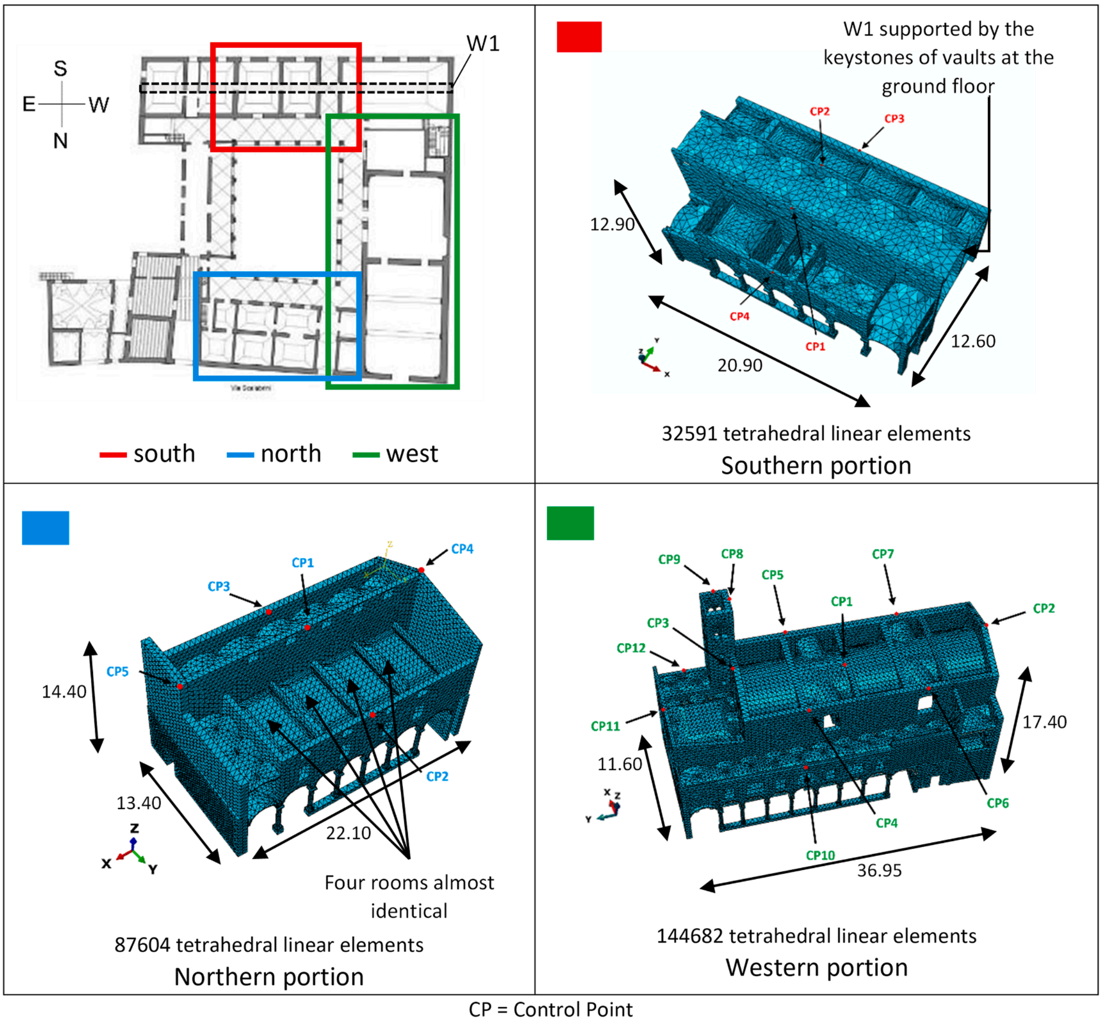


Fig. 24. Partial models of the structure analysed (for more details on the geometry and the pictures see Fig. 2-Fig. 7).

reason, it is necessary to accurately evaluate the effectiveness of the connection between orthogonal walls, floors and roof and the presence of tie rods. Preliminary surveys are paramount to corroborate all the hypotheses assumed, to achieve an established level of knowledge, as required for instance by the Italian code. If connections between orthogonal walls are effective, the collapse mechanism can be activated for confined horizontal flexure, which is associated to higher values of collapse loads for walls of limited width, because the resistant arch effect is activated better, as for walls N1, N2 and N6. On the contrary, for walls E1, E2 and N5, which are longer and with small thickness, the resistant arch effect cannot give a significant contribution against the out-of-plane actions. Moreover, vertical flexure shall be considered when there are walls characterised by limited thickness and significant height. In this case only wall S2 has been studied under vertical flexure; the result obtained, not surprisingly, indicates a very low collapse load, because the wall is characterized by small thickness and a significant height. Also in this case, it is very important to accurately evaluate the effectiveness of the connections between orthogonal walls to understand if the mechanism can be activated, and consequently evaluate which retrofitting interventions shall be adopted. The corner overturning at the intersection between walls N1 and N4 was also analysed, since it is free and not constrained by adjacent buildings. The collapse load multiplier is very high because a good connection between orthogonal walls has been assumed and the roof is not of thrusting type.

Among the 28 possible local collapse mechanisms suggested in [52], the ones that could be activated in the portion of the building aggregate comprising the church are (i) the overturning of the façade involving perpendicular walls (F1), (ii) the base overturning of the façade (F2), (iii) the overturning of the tympanum (T1), (iv) the tympanum rocking involving horizontal flexure (T2), and (v) the transversal out-of-plane failure of the church nave (LW). Results are shown in Fig. 15.

Again, it is evident how the assumptions made on the effectiveness of the connections between orthogonal walls is of fundamental importance; if the connections between the façade and lateral walls are effective, the mechanism that could be activated is F1, corresponding to a higher collapse PGA with respect to mechanism F2. The simple overturning of the tympanum, T1, is characterized by a higher collapse load multiplier with respect to that of the mechanism involving horizontal flexure, T2. The roof is not of thrusting type, therefore the simple overturning of the lateral walls of the nave is not activated. Even if longitudinal walls have limited thickness, they are constrained by tie rods positioned inside the church; moreover, on one side the wall is constrained by cross vaults belonging to the cloister outside the church, and on the other side by adjacent buildings. In this case the configuration of the church inside the building aggregate has beneficial effects in avoiding the lateral overturning of the nave walls.

5.3. Bell tower LV1 analysis

The seismic vulnerability of the bell tower, belonging to the ex-monastery object of study, has been first evaluated by means of the so-called LV1 analysis as established in [44]. Since the bell tower is

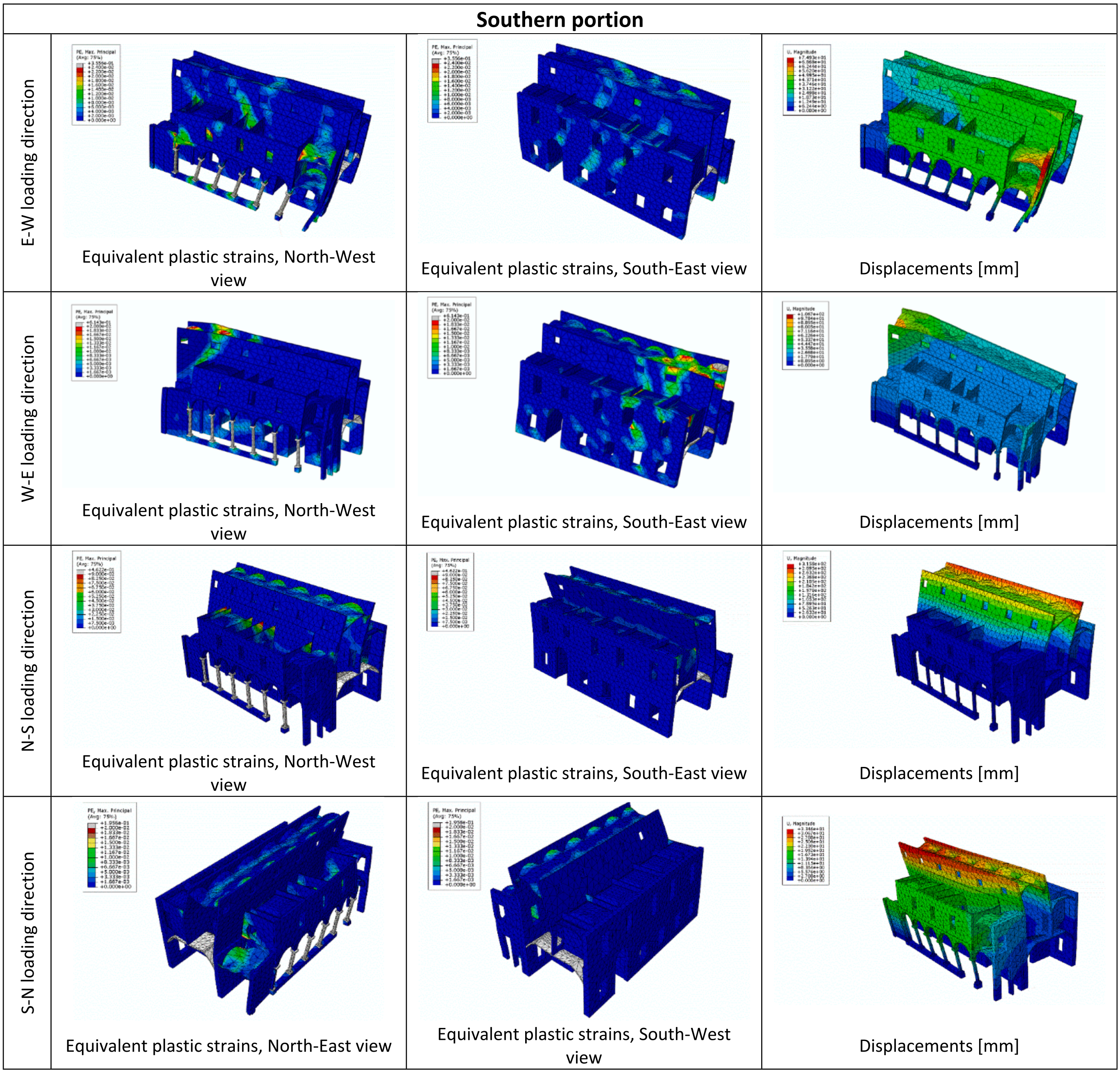


Fig. 25. Southern portion.

laterally connected with adjacent buildings of different heights, two cases have been considered, which most probably furnish a lower and upper bound of its actual behaviour: the case where the tower is assumed fixed at a height corresponding to the maximum of the neighbouring walls and the case where the fixed base is put in correspondence of the minimum height. Such approach can be followed any time there is a tower emerging from an urban aggregate, since generally the height of adjacent building walls is not constant.

The results, obtained assuming the mechanical properties listed in Table 1 for masonry, are summarized in Fig. 16, where the acting (two cases) and resisting bending moments are compared.

From Fig. 16, the critical section is located at the base, at 11.60 m for case 1 and 15.25 m for case 2; the small openings present on the tower do not significantly reduce the resisting moment of the sections, as it is possible to see by the regularity of the resisting bending moment shape. From the diagrams, it is clear that case 2 is the most unfavourable. Anyway, in both cases the safety factor is greater than one, meaning that the tower is safe. More refined analysis considering the real height of the neighbouring structures can be carried out by means of FE models, if possible.

A first evaluation of further failure modes of masonry towers can be done using simple computations, such as in [84], where five failure modes on isolated regular masonry towers were analysed: rocking with vertical splitting, monolithic rocking, Heyman's diagonal cracking and rocking, mixed Heyman's mechanism with vertical splitting, base shear sliding (see Table 2). Among these failure modes, the one with the lowest collapse load multiplier is the most likely. Computations for the case study were performed on the emerging portion of the bell tower, considering Case 1 and Case 2 described before, accounting for the real thickness of the walls and disregarding the small openings for the sake of simplicity. The lowest collapse load multiplier was obtained for mechanism #4 (mixed Heyman's mechanism with vertical splitting), see

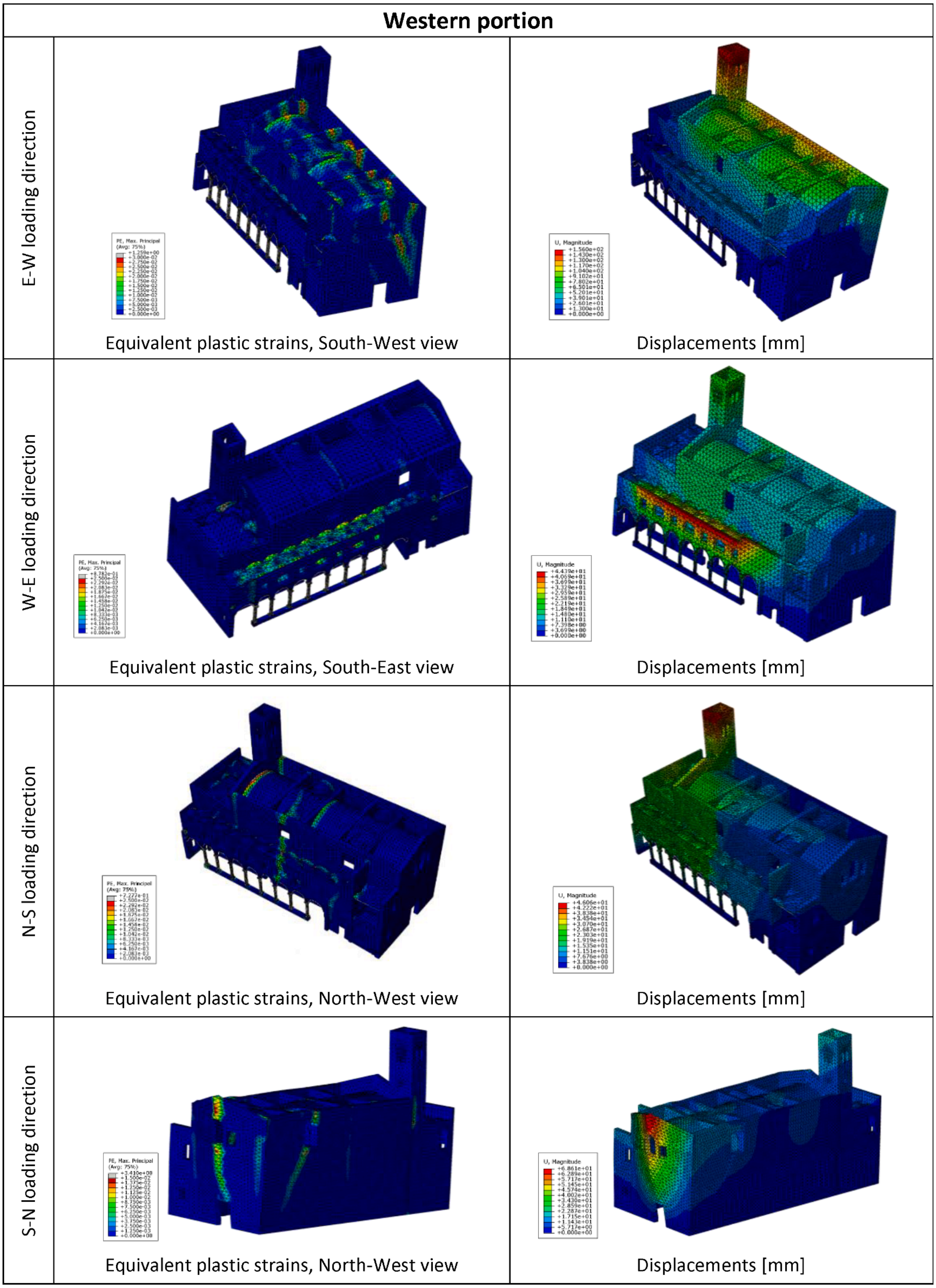


Fig. 26. Western portion.

Table 2. Such result highlights that the failure mechanism proposed by the LV1 method in [44], very similar to mechanism #2 according to [84], is not always the most probable one. Thus, more refined analysis shall be performed.

5.4. Pushover analysis on equivalent frames

Once the out-of-plane behaviour of the structure has been assessed in a simplified way, it is possible to study the in-plane behaviour of meaningful in-plane loaded walls using the equivalent frame method described before. The panels that have been considered are shown in Fig. 17. Each wall has been idealized using the equivalent frame model, as shown in Fig. 18 a) and b) for wall N3. In the equivalent frame model, arches are modelled as simple masonry portals. Pushover analyses have been carried out under uniform and triangular distributions of horizontal forces in positive and negative directions; the results obtained are

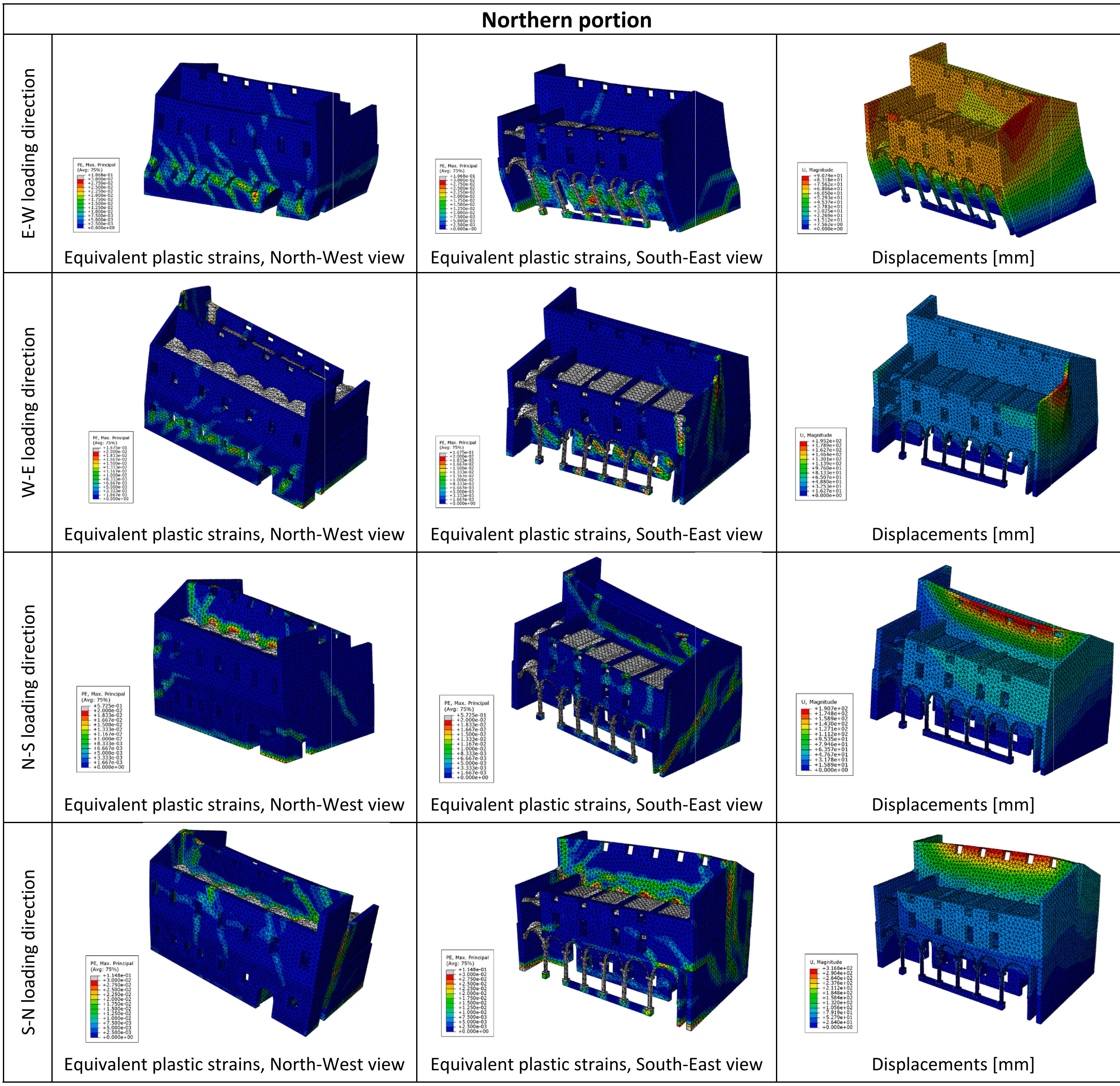


Fig. 27. Northern portion.

the capacity curves in terms of total shear at the base and displacement of a control point, as shown in Fig. 18 c) for wall N3, deformed shapes with activation of the different plastic hinges on the elements, and failure mechanism.

From the results obtained, almost all the walls reached the SLV limit state for the same peak ground acceleration, activating a shear failure of the piers.

The results in terms of ultimate displacement and safety factor, given by the ratio between the displacement capacity and the displacement demand, are synoptically depicted in Fig. 19.

Even if the collapse PGA is roughly the same in all cases, the displacement capacity is different form case to case. The displacement capacity can change significantly also between positive and negative direction. This is clearly a consequence of the geometry of the panel, in particular the dimension of the different piers and the disposition of the openings play a crucial role. Looking at Fig. 19, it is evident that the displacement capacity of wall S3 is greater in the negative direction. From Fig. 20, where deformed shapes at the ultimate displacement are shown, the reason becomes clear; the left upper portion of the wall does not have openings, this leads to a higher resistance when loaded in the negative direction, therefore to a larger displacement capacity. While looking at Fig. 19, the displacement capacity of wall S2 is greater for positive loading direction. Fig. 20 shows that the right portion of the masonry panel S2 is characterized by large piers, leading to a higher resistance when loaded in positive direction and consequently to a greater displacement capacity.

From the analysis it appears clear that pushover coupled with equivalent frame models can be used to evaluate masonry in-plane behaviour. The most critical walls are those having big openings at the ground level, as wall N3, or having several openings and thin piers, as it happens for wall N6, Fig. 20. Walls characterized by a significant number of openings are more flexible, as a consequence the

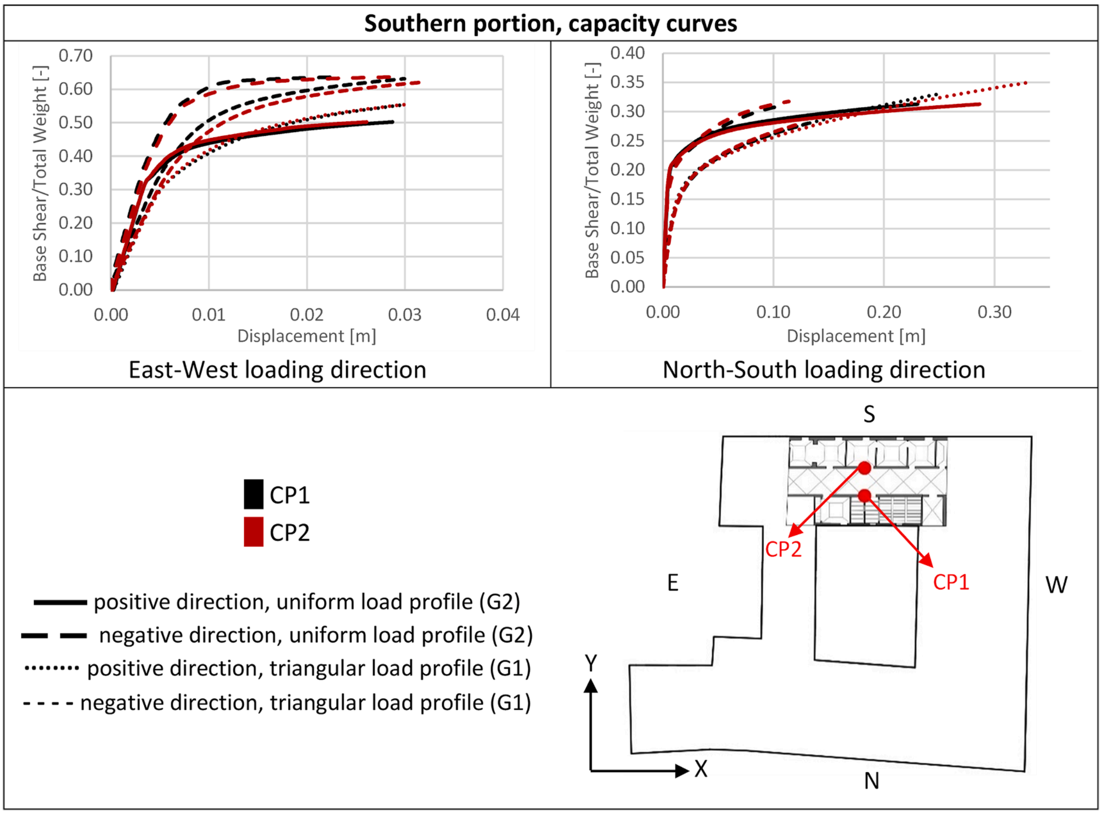


Fig. 28. Southern portion, capacity curves.

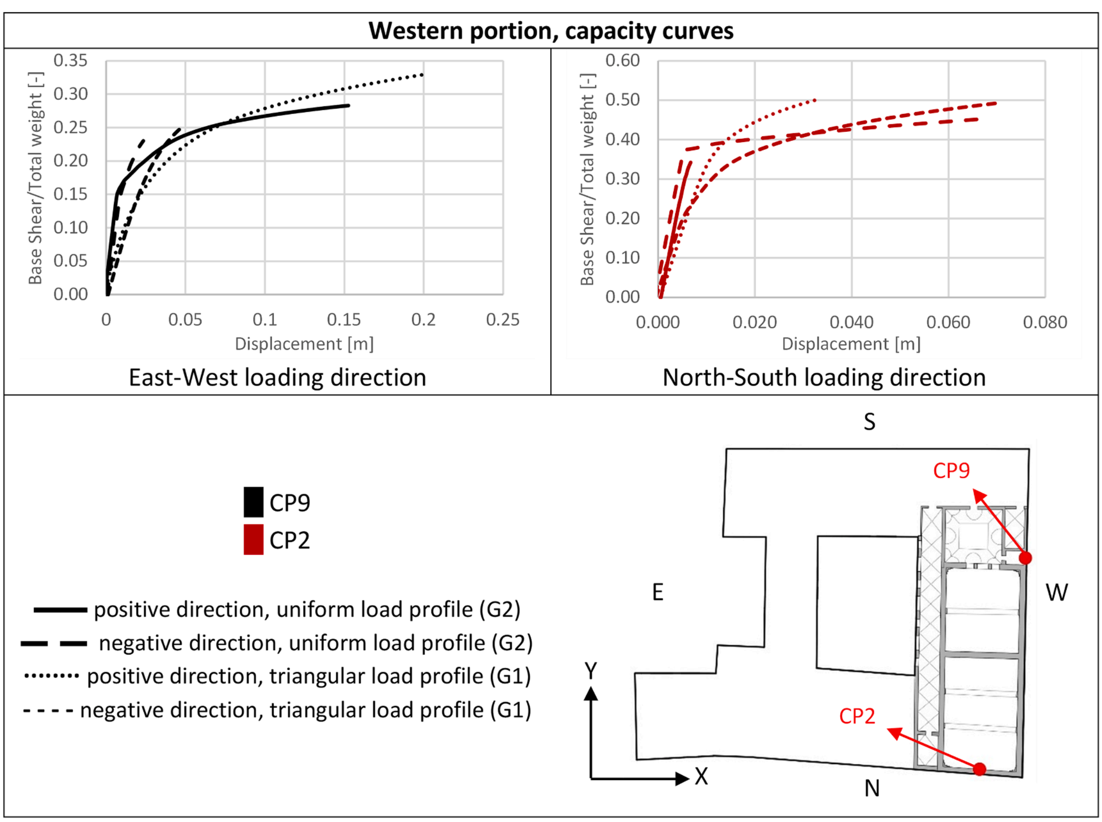


Fig. 29. Western portion, capacity curves.

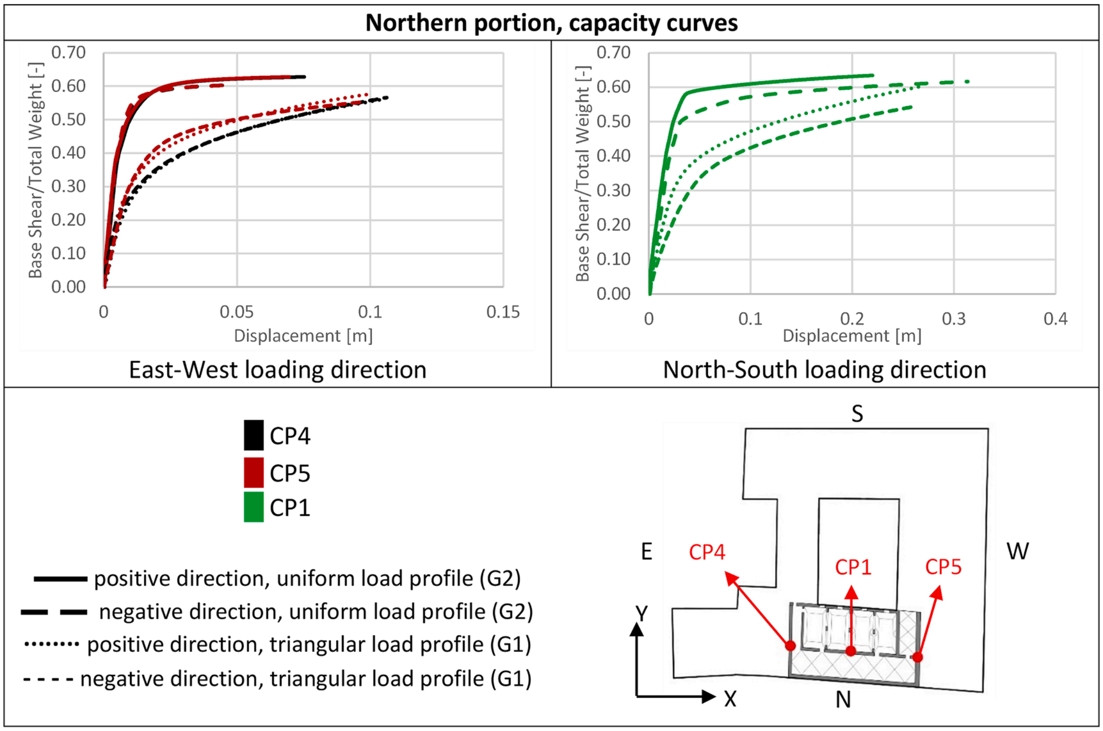


Fig. 30. Northern portion, capacity curves.

displacement demand is higher, and the safety factor lower.

The in-plane behaviour of wall N3 was also studied performing a pushover analysis with a full 3D model in Abaqus/CAE®. The mesh used is shown in Fig. 21. Masonry was assumed obeying a CDP model as described before. It was possible to estimate the collapse acceleration, the ultimate displacement and the crack pattern (Fig. 22) performing pushover analyses along positive and negative X directions with uniform and triangular distributions of horizontal loads. The results obtained with the softwares Abaqus/CAE and PRO_SAM/SAM II (i.e. full 3D and equivalent frame models) are listed in Table 3.

Looking at the pushover analysis under uniform distribution of forces, similar results in terms of acceleration have been found, while for the triangular distribution, for a similar displacement the acceleration found with the software Abaqus/CAE are much lower. This could be due to triggering of a different failure mechanism. On the one hand, the model developed in Abaqus/CAE considers the actual geometry of the arch, while in the equivalent frame model it is modelled as a simple masonry portal. Indeed, analysing Fig. 22 it is evident a concentration of damage on the lintel above the arch, when the wall is subjected to a uniform distribution of load in the positive direction. Instead, the failure mechanism depicted in Fig. 20 for wall N3 does not involve damage in the lintel of the masonry portal corresponding to the arch. This is because the software PRO_SAM represents the instantaneous damage, not the cumulated one. When the angular deformation threshold is overcome in piers or spandrels, the resistance of the element goes to zero. Therefore, there is a force redistribution in the elements of the equivalent frame and some of them are released. On the other hand, in the equivalent frame model, the coupling between shear and normal stresses is taken into consideration only in a simplified manner, a feature which may trigger in some cases inaccurate failure mechanisms. In addition, the CDP model was primarily developed to simulate the brittle behaviour of concrete under low confining pressures and not specifically for masonry, thus the difference in the results may be partially ascribed to such adaptation of CDP in a different context.

The capacity curves are shown in Fig. 23, in terms of normalized shear force (given by the ratio between the base shear force and the total weight of the wall). The results are comparable both in terms of base shear force and in terms of ultimate displacement.

5.5. Pushover analyses on 3D FE models

5.5.1. Analyses with partial models

In the case study, three significant portions of the aggregate have been identified and analysed (Fig. 24). The southern portion is representative of the whole southern part of the aggregate, characterized by a basilica layout. Moreover, it is interesting because there are at the ground level big rooms covered by cloister vaults, bearing at the middle span one of the highest walls of the first level (W1), while generally the first level is characterized by smaller rooms covered by cloister vaults or timber decks connected by a corridor with cross vaults, see Fig. 24. The northern portion is characterized by a more regular layout; at the ground level there are small rooms covered by cloister vaults, while the first floor presents a corridor with cross vaults connecting four rooms of almost identical dimensions covered by cloister vaults. The western portion is representative of church inserted in the building aggregate. It is expected to have a different behaviour with respect to the other parts of the structure, because it is geometrically characterized by a big nave with the neighbouring bell tower.

As a matter of fact, for building aggregates with deformable floors, the activation of partial failure mechanisms is expected. For this reason, it is recommended in common practice to monitor different control points, located in all those panels that can be potentially affected by local overturning. Fig. 24 shows the control points (CPs) selected for each portion. According to modal analysis, the structure is mainly characterized by local mechanisms, therefore, CPs are located on the most vulnerable elements to detect the activation of local failures. In Figs. 25–30 the results of the analyses performed on the considered portions are synoptically shown.

Results are reported in terms of equivalent plastic strains and displacements at the end of the analyses; subdivided for each portion considered and for each loading direction (East-West, West-East, North-South and South-North); furthermore, the capacity curves of the most significant control points are shown. The load profiles (G1 and G2) adopted for the pushover analyses performed on partial models were

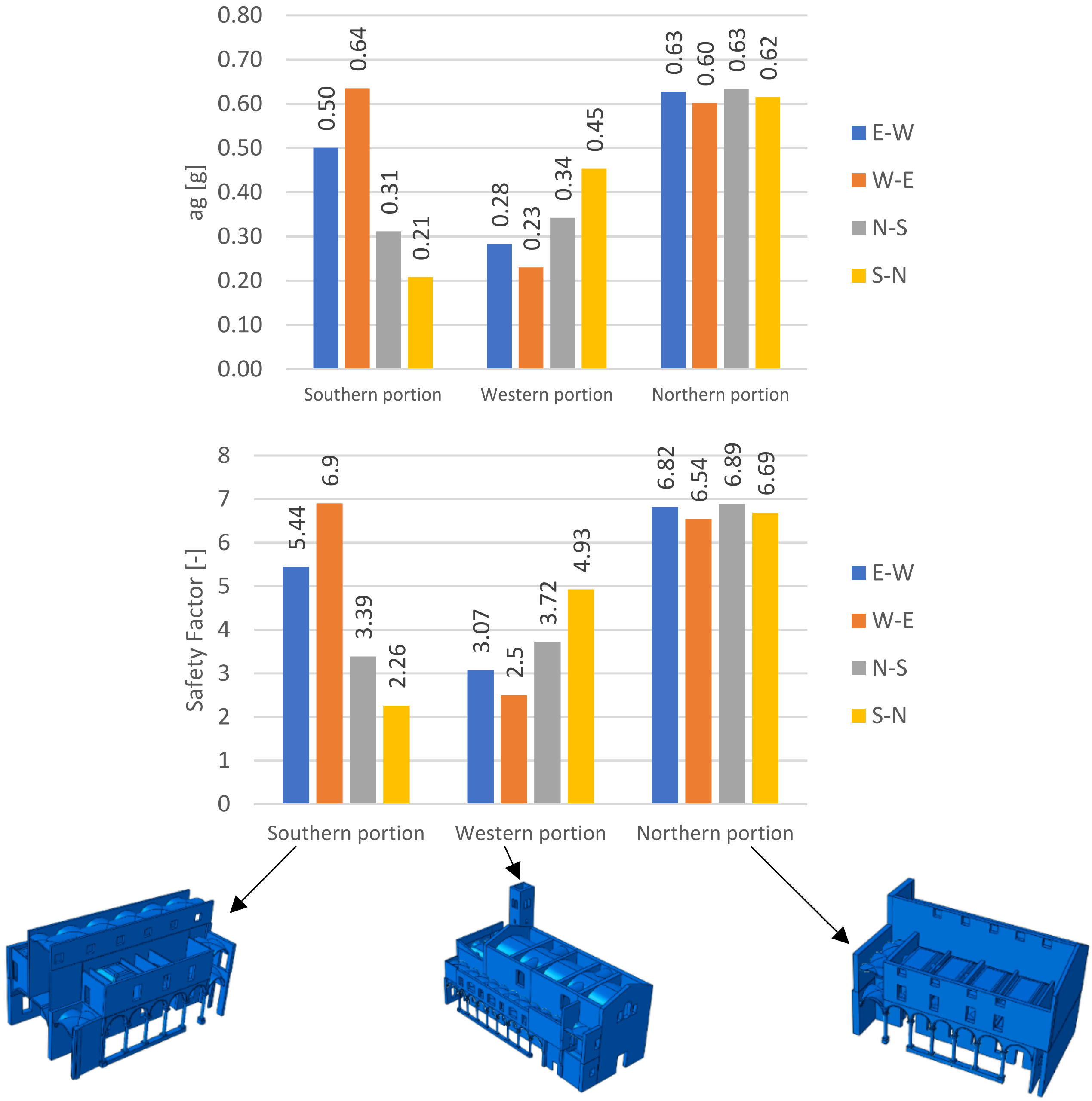


Fig. 31. Pushover analysis on partial 3D FE models, synopsis of the results in terms of collapse acceleration and safety factor.

assumed in agreement with the Italian building code [27]. The first one (G1) is the so-called principal distribution of forces, proportional to the mass and linearly proportional to the height of the structure (inverted triangle distribution), whereas the latter (G2) is the so-called secondary distribution of forces, proportional to the mass and uniform along the height of the structure. Pushover curves for significant control points are shown for both load profiles; instead, for the sake of brevity, the plastic strains and the displacements for each portion are shown only for the G2 load profile.

Fig. 31 shows a synopsis of the results of the analyses carried out, namely the collapse accelerations and the safety factors in terms of acceleration of all the portions, considering the minimum values found among the CPs of the portions in each direction. It is evident that the most vulnerable portion of the aggregate turns out to be the western one, which comprises the church. This is not surprising, considering its irregularities. The longitudinal walls are not constrained by perpendicular walls giving extra strength against out-of-plane bending, differently from the other cases analysed. The lowest maximum PGA was found for East-West horizontal loading direction, i.e., that direction perpendicular to the longitudinal walls of the nave. From the results (Fig. 26), the CPs located on longitudinal walls exhibit significant displacements when subjected to a seismic action along the East-West direction. Furthermore, it is evident that another vulnerable part is the bell tower, which shows significant displacements when compared with the other CPs, considering the East-West and North-South positive load directions (Fig. 26, Fig. 29). Its slenderness plays a crucial role in this case. Moreover, it is possible to see the activation of a local mechanism when the structure is subjected to a horizontal load along the South-North direction; CP2, which is the control point located at the top of the church façade, exhibits a very high displacement (Fig. 29). Looking at the equivalent plastic strains depicted in Fig. 26, the horizontal bending of the façade is quite evident. The activation of such rocking mechanism was found to be critical also applying the linear kinematic analysis, but the collapse acceleration found was lower, probably because of the

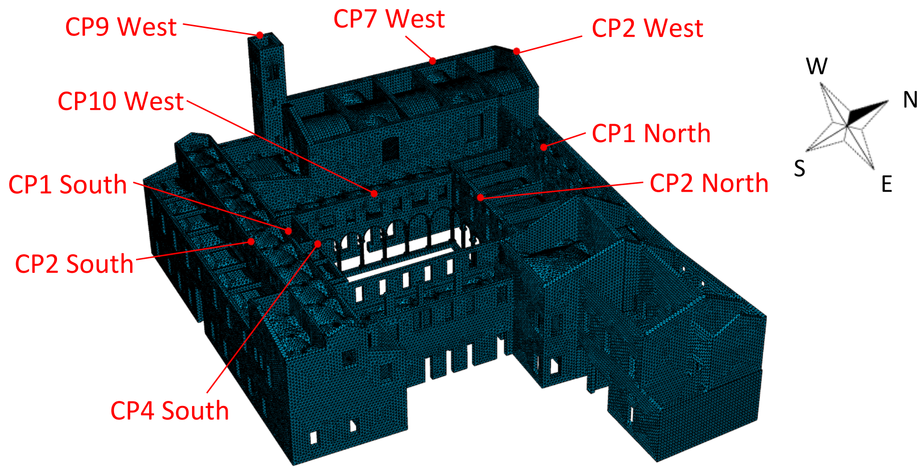


Fig. 32. Full model Control Points.

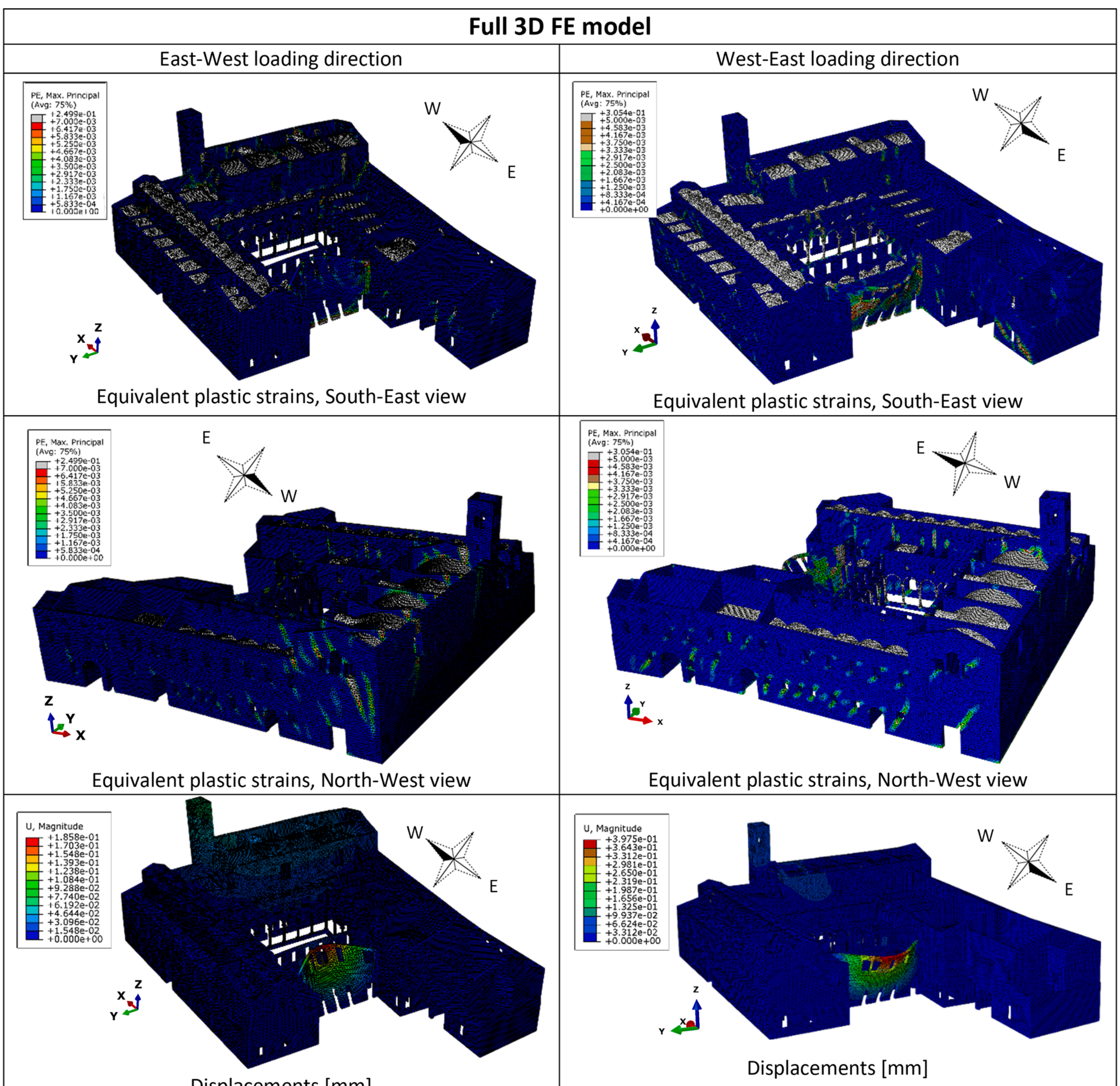


Fig. 33. Equivalent plastic strains and displacements of the full model under E-W (left) and W-E (right) seismic load directions.

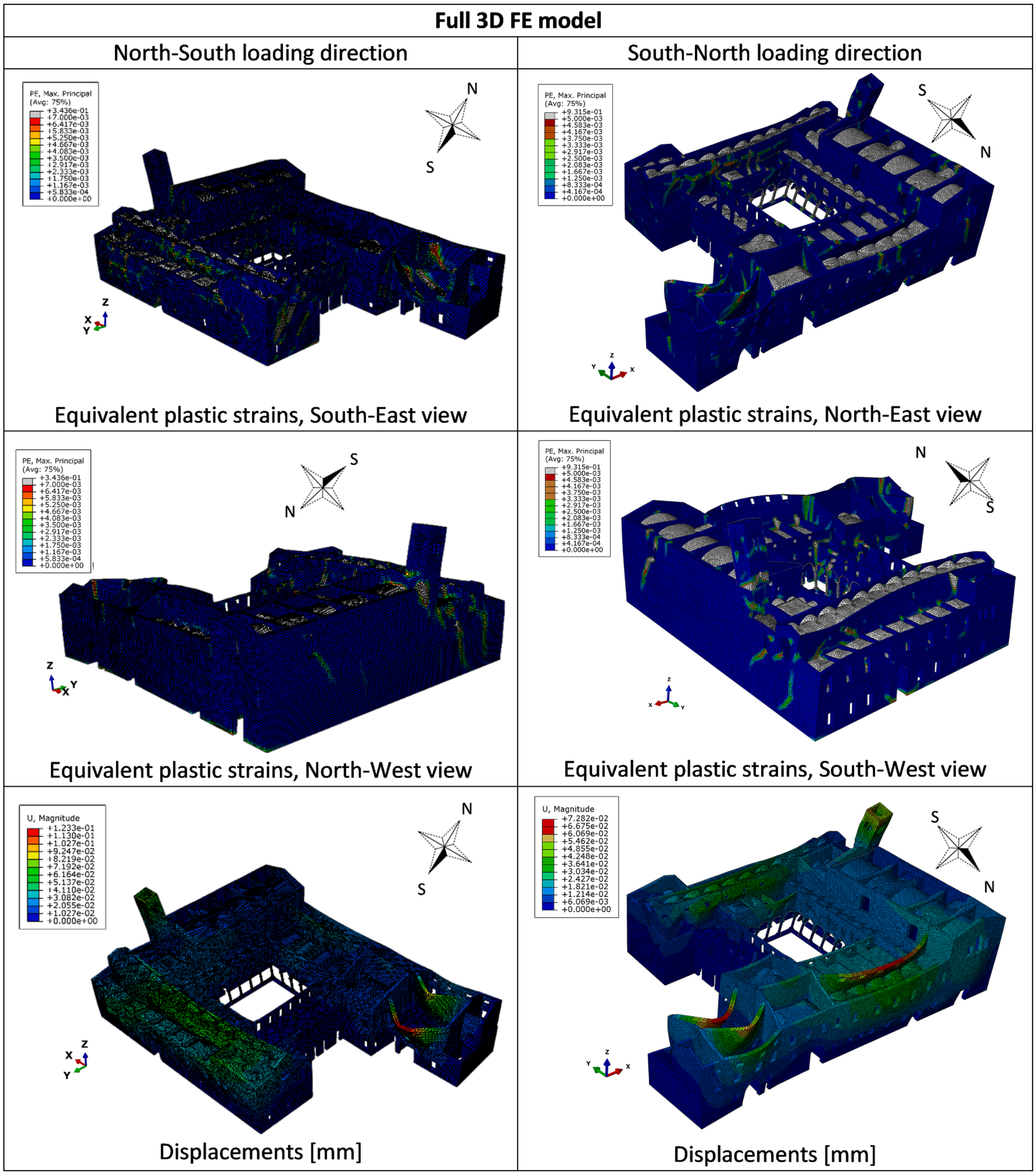


Fig. 34. Equivalent plastic strains and displacements of the full model under N-S (left) and S-N (right) seismic load directions.

Table 4
Results comparison for the northern portion.

Northern portion								
Model	**Full model**				**Partial model**			
Loading direction	E-W	W-E	N-S	S-N	E-W	W-E	N-S	S-N
a_g [g]	0.469	0.535	0.626	0.602	0.627	0.602	0.634	0.616
CP1 north [m]	0.009	0.013	0.042	0.068	0.069	0.044	0.220	0.314
CP2 north [m]	0.010	0.014	0.021	0.022	0.073	0.047	0.067	0.065

conservative assumption made in that case for the material and the interlocking. Moreover, the shape of the failure mechanism turns out to be different in the two approaches. Another control point exhibiting a significant displacement is CP10, which is located on the wall above the columns of the cloister as shown in Fig. 24, under a West-East application of the seismic load. That wall is not constrained by orthogonal walls, only by cross vaults.

From Fig. 31, it is possible to notice that when the structural layout is more regular, as in the northern portion, the portion of the aggregate can withstand larger horizontal loads. Comparing the results obtained for the northern and southern portions, it can be seen that for the seismic load acting along the East-West direction the maximum PGAs are

Table 5
Results comparison for the southern portion.

Southern portion								
Model	**Full model**				**Partial model**			
Loading direction	E-W	W-E	N-S	S-N	E-W	W-E	N-S	S-N
a_g [g]	0.469	0.535	0.626	0.602	0.501	0.635	0.312	0.208
CP1 south [m]	0.006	0.008	0.070	0.030	0.501	0.023	0.230	0.031
CP2 south [m]	0.006	0.008	0.061	0.034	0.029	0.029	0.286	0.029
CP4 south [m]	0.005	0.009	0.023	0.027	0.026	0.023	0.007	0.018

Table 6
Results comparison for the western portion.

Western portion								
Model	**Full model**				**Partial model**			
Loading direction	E-W	W-E	N-S	S-N	E-W	W-E	N-S	S-N
a_g [g]	0.469	0.535	0.626	0.602	0.283	0.230	0.342	0.453
CP2 west [m]	0.025	0.013	0.018	0.017	0.036	0.006	0.007	0.068
CP7 west [m]	0.045	0.025	0.009	0.010	0.123	0.014	0.003	0.011
CP9 west [m]	0.072	0.055	0.071	0.048	0.152	0.024	0.039	0.016
CP10 west [m]	0.020	0.030	0.020	0.017	0.029	0.044	0.022	0.007

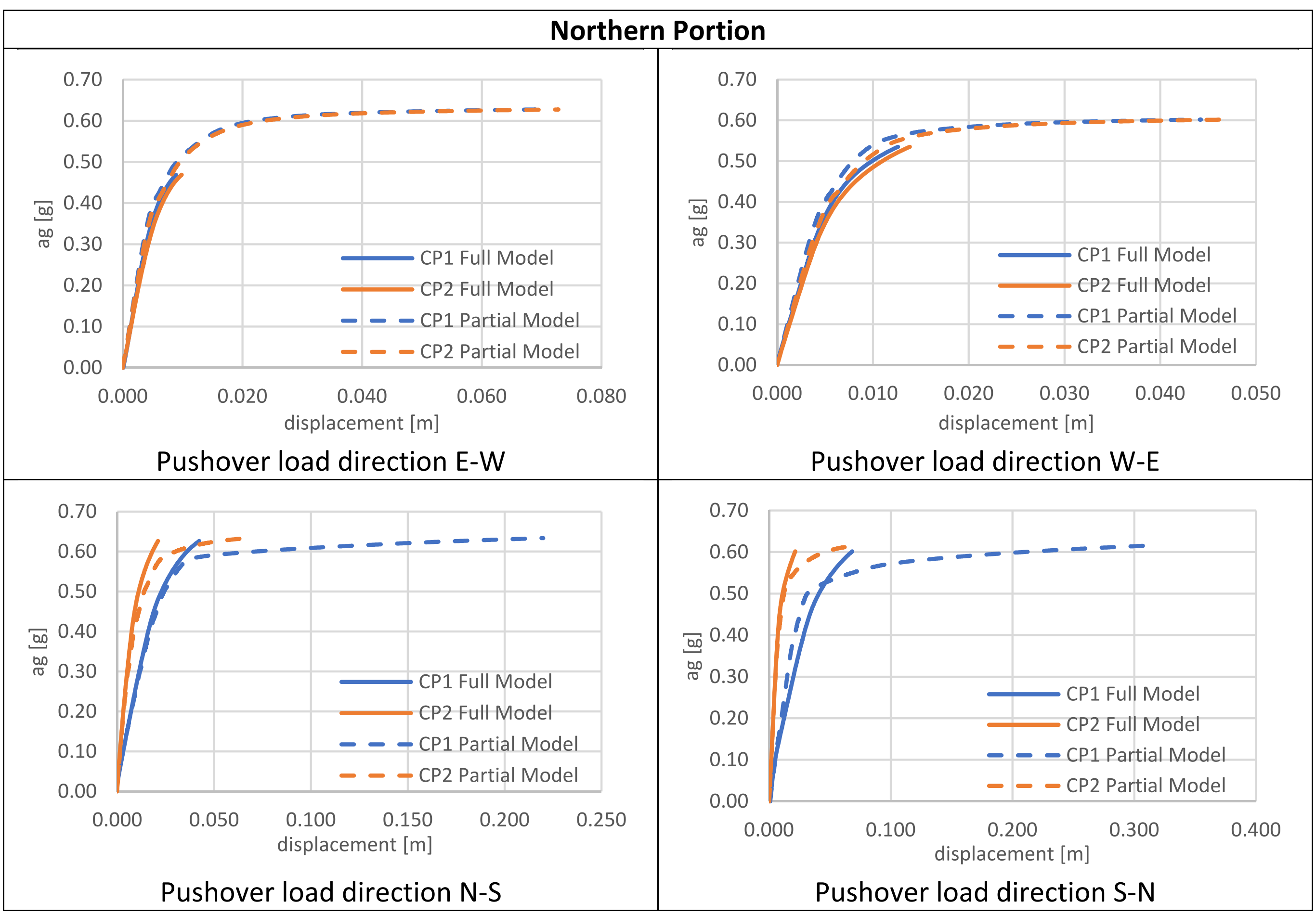


Fig. 35. Comparison among pushover curves obtained in the different cases for the norther portion.

similar, while when the horizontal action is oriented along the North-South direction, the maximum PGA of the southern part is much lower than that of the northern portion.

The CPs of the southern portion having the highest displacement in North-South loading direction are CP1 and CP2, both located on the highest part of the wall characterizing the basilica layout (Fig. 25, Fig. 28). They exhibit large value of displacement associated to a collapse acceleration that is much lower than that associated to the northern portion, suggesting that the higher seismic vulnerability of the southern portion may be a consequence of the irregularity in elevation

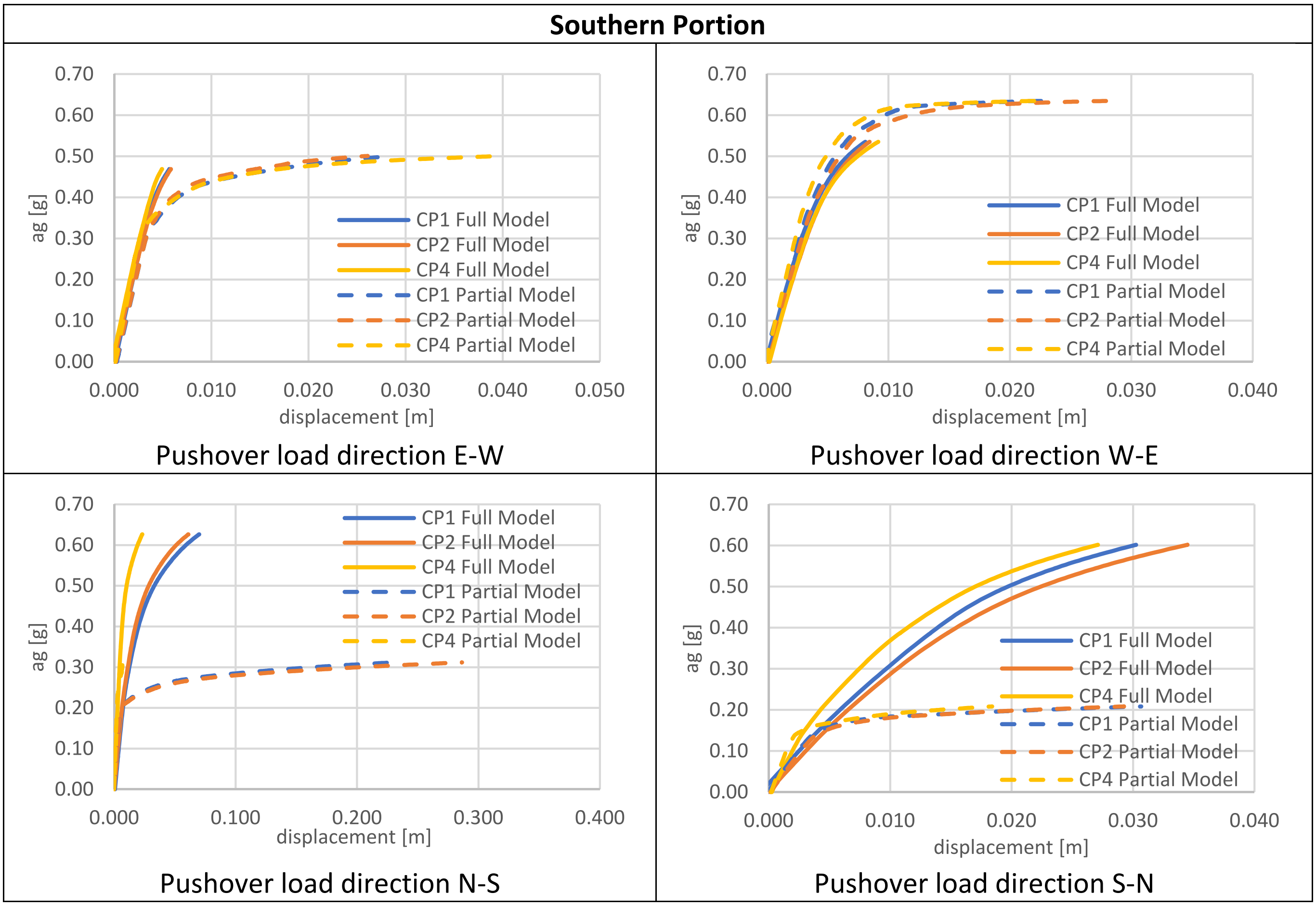


Fig. 36. Comparison among pushover curves obtained in the different cases for the southern portion.

typical of the basilica layout.

Looking at Fig. 27, it is evident that CP1 is characterized by quite large displacements if compared with those of the other CPs for a North-South direction of the seismic load, both positive and negative. CP1 is indeed located on the spine wall, as shown in Fig. 24. Looking at the equivalent plastic strains (Fig. 27), it is evident the activation of a local mechanism due to confined horizontal flexure. The activation of this mechanism was found also through the linear kinematic analysis, but associated with a lower acceleration which is again a consequence of the conservative hypotheses made.

5.5.2. Analysis with the full 3D FE model

In order to show that isolating portions of the structure and performing pushover analyses on partial models results in finding conservative results, a full 3D FE model of the whole structure was used to carry out additional non-linear static analyses (Fig. 32). In Figs. 33–34 the results of such analyses are reported. In particular, Fig. 33 depicts, for the sake of brevity, only the equivalent plastic strain colour patches at failure and ultimate deformed shapes when the aggregate is loaded with a uniform horizontal distribution of forces (G2) along the East-West and West-East directions. Fig. 34 shows the same results but in the case of application of the seismic load along the North-South and South-North directions. In Table 4, Table 5 and Table 6 it is possible to compare the results obtained with the full 3D FE model and with the partial ones, for the four load directions inspected, in terms of collapse acceleration and ultimate displacement of some control points. Furthermore, in Figs. 35–37 the capacity curves obtained with the full 3D FE model and with the partial models are compared.

From an insight of all the pushover curves (Figs. 35–37), it is possible to see that the initial stiffness is very similar, between full and partial models. Moreover, it is evident that the isolated models are generally characterized by a larger ductility and the ultimate resistance is reached for lower values of horizontal acceleration. Such behaviour was expected, since in the literature it was proved that isolated buildings exhibit higher ductility than that of the aggregates.

Generally speaking, from the simulations, it can be stated that the ultimate load carrying capacity of the full 3D-model is higher than that of the partial models, because of the larger redundancy. The only exception was found for the East-West load direction for the southern and northern portions, where lower collapse accelerations were found. This may be explained by the activation of the local mechanism of wall E1, where confined horizontal flexure is activated as shown in Fig. 33.

To briefly conclude, it may be stated that taking into account isolated portions of the aggregate as reference can be considered conservative only if the load carrying capacity is assumed as the main parameter to maximize. An opposite trend is observed for the ultimate displacements, which however are very difficult to determine in an accurate way for historical buildings and in complex 3D FE models. For this reason a displacement based assessment is not recommended for such kind of structures.

5.5.3. Analyses adopting an “hybrid” model

Pushover analyses carried out by means of partial models provide results quite different from those obtained by means of a full 3D FE discretization. Moreover, the circumstance that the numerical simulations are affected by premature halting because a portion of the structure is affected by the activation of partial failures, could make impossible the correct characterization of the seismic vulnerability.

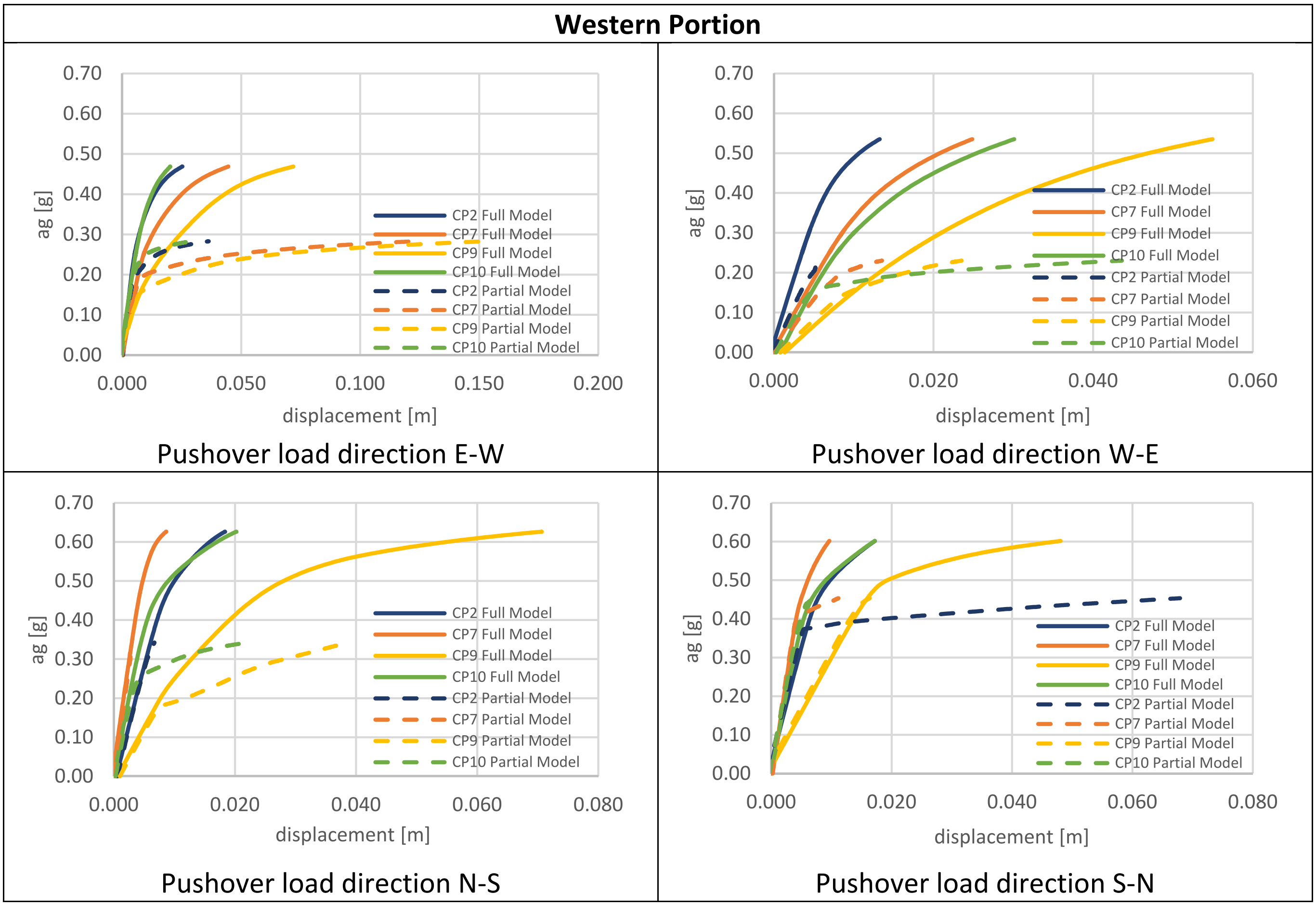


Fig. 37. Comparison among pushover curves obtained in the different cases for the western portion.

Such condition is the more likely the more wide and complex the model is. It is therefore needed to operate with partial models as discussed previously. On the other hand, it would be difficult to identify the minimum length of the portions of the neighbouring walls that actually contribute to the extra-redundancy of the structural system, that cannot be taken in any consideration within partial models. Hence the idea to consider "hybrid" full models, in which, only for the portion subject of investigation, the behaviour of the materials is assumed non-linear, while for the context the rest it is assumed linear elastic, may have a certain interest.

In the analyses carried out here on the "hybrid" models, the material behaviour of the vaults is considered either linear elastic or non-linear. In Figs. 38–40 the capacity curves, obtained assuming a uniform distribution of horizontal forces (G2), are reported and compared with previously presented results. It is possible to notice that using the "hybrid" model with linear elastic vaults, the capacity curves are similar to the ones obtained with the full model but with a larger global ductility meaning that the analyses do not stop due to premature local collapses of other portions of the aggregate; therefore the characterization of the studied portion turns out to be more accurate since the actual collapse acceleration of the portion under study can be detected. However sometimes the capacity curves are stiffer than the full model, as it happens in the pushover analysis carried in the North-South positive direction, shown in Fig. 39. Instead, the capacity curves obtained with the "hybrid" model with non-linear material behaviour of the vaults are characterized by a lower collapse acceleration with respect to that obtained with the full model, but higher with that obtained using partial models; the initial stiffness is on the contrary similar. Since the interruption of the analyses due to local collapses of other portions of the building is not possible, this type of model allows for a more accurate evaluation of the structural behaviour of the studied portion of the aggregate, taking correctly into account the confinement given by the neighbouring portions and the non-linearity of the materials. Moreover, since the vaults are considered non-linear, all the possible local collapse mechanisms can be taken into account, as for instance the horizontal flexure of the church façade (CP2), see Fig. 40.

From the results obtained, at least for this case study, it is possible to state that the more suitable model to characterize the structural behaviour of a building aggregate portion is the "hybrid" model with non-linearity of all the elements, including the vaults. Partial models tend to provide too conservative results, largely underestimating the global stiffness and not considering the beneficial effects given by the constraints provided by the rest of the structure. Such issue could be overcome by introducing appropriate boundary conditions in the partial models; the difficulty consists in the calibration of such appropriate boundary conditions, a feature that could be time consuming. On the other hand, the hybrid model with linear elastic behaviour of the vaults exhibits a too large stiffness, because the curved elements do not crack during the deformation process, in contrast with their observed experimental behaviour.

6. Conclusions

In this paper, a general methodological approach to evaluate the seismic vulnerability of masonry aggregates has been proposed, based on well-known strategies of analysis that nowadays can be commonly used by practitioners with a standard commercial code. The procedure is a protocol that consists in performing several different types of

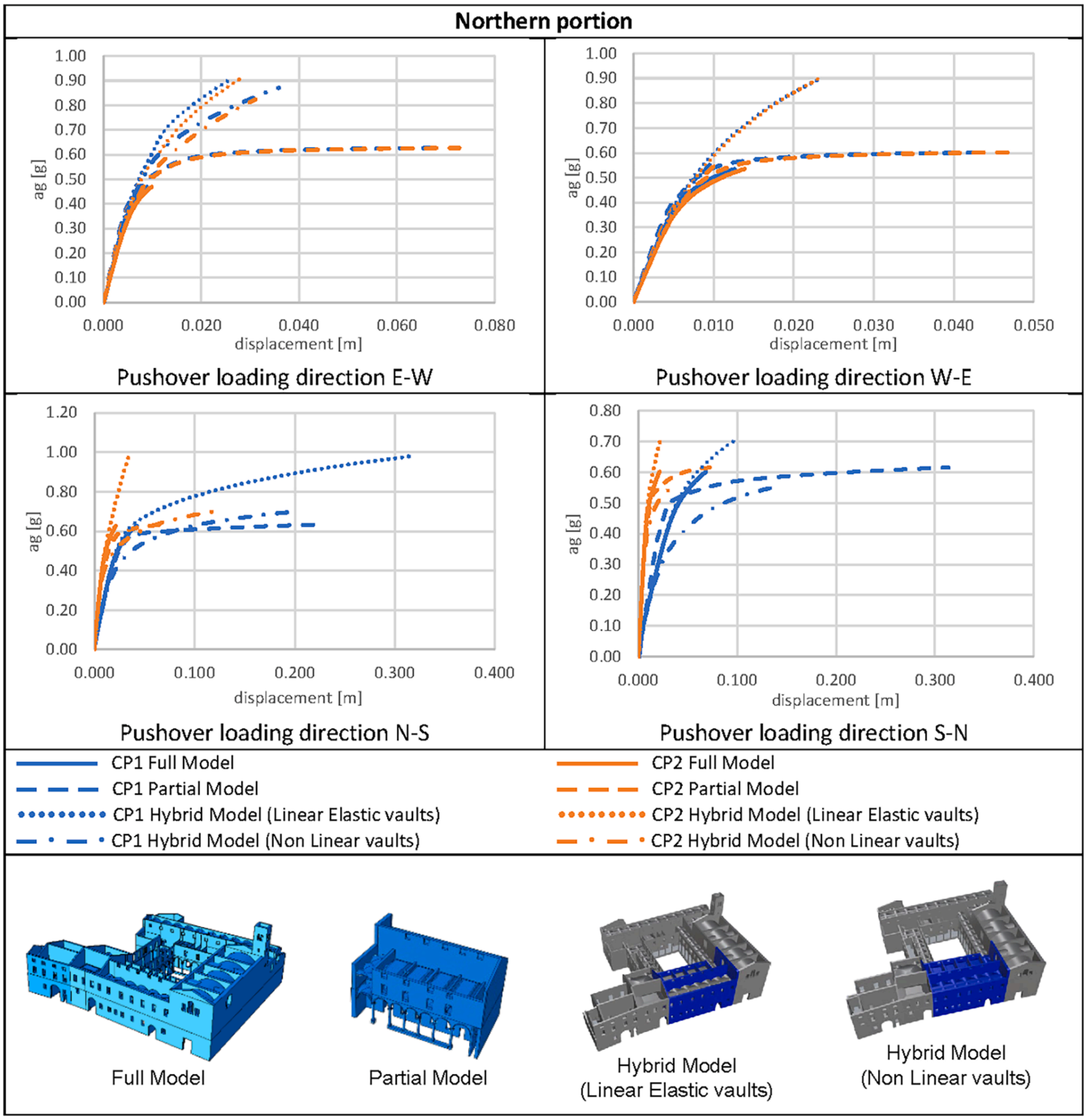


Fig. 38. Comparison of the capacity curves of the northern portion obtained with the full model, the partial model and the "hybrid" model.

computations, from the simplest to the most advanced, then in comparing the results and finally in drawing conclusions rating the relative goodness of the results obtained by means of the different methods. Following the methodological approach proposed, it is possible to evaluate effective retrofitting interventions to decrease the seismic vulnerability of the aggregates. The aim was to focus in particular on the definition of a protocol to follow in case of historical buildings. Such a protocol allows to evaluate all the data necessary (in practice it allows to identify the most vulnerable portions of the building) for a future seismic retrofitting intervention, that will be studied in dedicated future research. The procedure has been benchmarked on a case study in Italy: the ex -monastery of Santa Maria della Pace in Piacenza.

The 3D geometrical model of the structure was built by means of the commercial software Revit 2022®. A linear eigenfrequency analysis has been carried out on the structure, by means of the software Abaqus/CAE® using a full 3D FE discretization. A global seismic response is unlikely for the problems under study and the identification of local modes with relatively high excited mass is interesting to preliminary screen those portions of the aggregate whose out-of-plane failure is most probable.

After that many possible local collapse mechanisms have been identified from the eigenvalue analysis, limit analysis has been used to evaluate the collapse acceleration. The spreadsheet C.I.N.E. has been used to evaluate the horizontal load multipliers. It was found that, for masonry aggregates, it is necessary to evaluate with care – by means of accurate surveys within a certain level of knowledge – the connections between orthogonal walls, in order to correctly evaluate the possibility of activation of simple overturning mechanisms, which are those characterized by the lowest collapse acceleration in case of walls with limited thickness. In this case, extra-care should be used because even for small PGAs, the safety factor could be smaller than 1, with the consequent need of retrofitting interventions. If the connections between orthogonal walls are effective, confined horizontal flexure could be critical; if walls are characterized by a significative length and a small thickness, the collapse load multiplier can be small and associated to a safety factor smaller than 1. When there are portions of the structure not confined by adjacent buildings, unconfined horizontal flexure may be activated and its safety should be checked, especially on slender walls. In this case the effectiveness of the connections with floors or the presence of tie rods should be accurately evaluated. Finally particular attention should be paid to the type of roof. In case of thrusting roofs, free corners

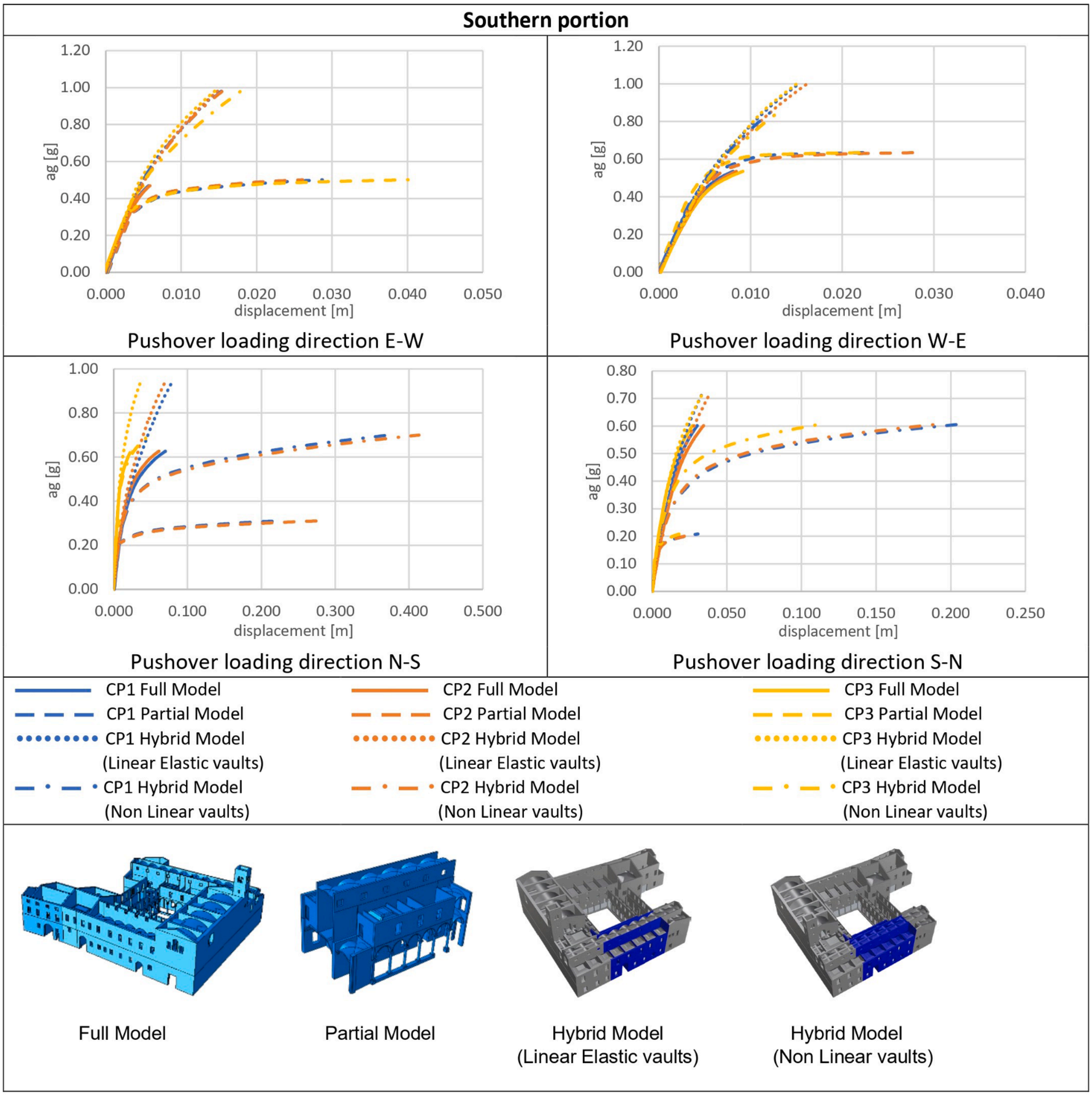


Fig. 39. Comparison of the capacity curves of the southern portion obtained with the full model, the partial model and the "hybrid" model.

of the structure could be subjected to overturning.

Since the structure studied as benchmark is a special aggregate with a church embedded, all the possible local collapse mechanisms that can take place among the 28 suggested by the Italian guidelines for the cultural heritage for churches, have been also analysed. It has been found that the most vulnerable part is the façade, which can suffer simple overturning, if connections with orthogonal walls are not good, or the out-of-plane bending of the tympanum.

The structural behaviour of the bell tower, another special portion within the aggregate, has been analysed by means of the LV1 approach suggested by the Italian guidelines. In the case analysed, it has been found that it can withstand the design seismic action.

Once the out-of-plane response of the structure has been evaluated, the in-plane response of significant masonry walls in-plane loaded has been investigated by means of pushover analyses. The equivalent frame model has been adopted and the in-plane response has been found through the SAM-II method. In order to carry out the analyses, the commercial software PRO_SAM has been used. From the outcome of the analyses, it is possible to state that the in-plane response of all the masonry panels analysed is adequate to withstand the design seismic action at the site, suggesting that the out-of-plane behaviour is critical. The results obtained with the equivalent frame for one of the walls have been compared to those provided by a full 3D discretization with damaging materials. It has been found that the equivalent frame provides conservative results; this could be due to the assumptions made for plastic and shear hinges and on the different type of elements used to model the wall.

Finally, the seismic response of different portions of the structure has been studied by means of full 3D FE meshes and non-linear static analyses using the software Abaqus/CAE®. Masonry non-linearity has been accounted for by means of the CDP model. For the case under study, it has been found that all the portions analysed can withstand high accelerations.

Finally, the results obtained using partial models have been compared to the ones coming from pushover analyses on a full 3D FE model of the entire aggregate and "hybrid" models. The outcomes show higher collapse accelerations and lower ultimate displacements for the full model, as it was expected since, due to its geometry, it is stiffer than the partial ones, which are isolated portions of the structure. However, the results for a portion of the building aggregate using the full model could be sometimes influenced by local collapses of other portions of the structure, thus a so-called "hybrid" model has been used to avoid the activation of collapses outside the portion of the aggregate under investigation. From the results obtained, it has been found that the most

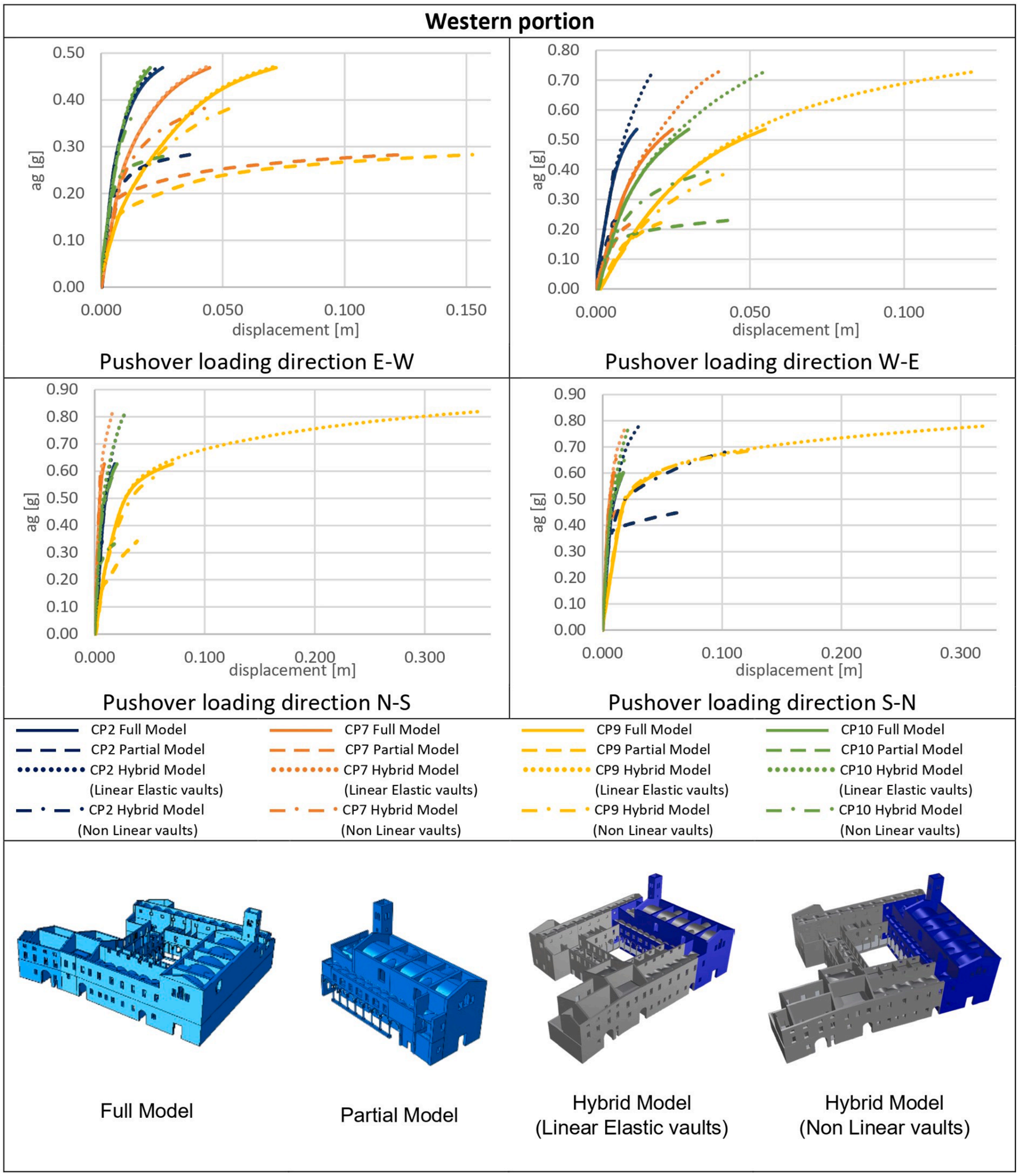


Fig. 40. Comparison of the capacity curves of the western portion obtained with the full model, the partial model and the “hybrid” model.

suitable model for the characterization of building aggregate portions is the “hybrid” model, where masonry non-linearity is accounted for only in the part subjected to investigation, including the vaults.

Aggregate effects in the proposed methodology are accounted for in the hybrid model, despite in an approximate but technically meaningful way. The increment of action in the header portion is considered, since both the actual geometry of the structure and the surrounding buildings are modelled. Instead, the whip effect could be considered if the boundary conditions with adjacent buildings are suitably modified, after a comprehensive evaluation of the role played by adjoining walls not belonging to the aggregate under study. This is a very complex topic that is postponed to dedicated future research.

Summing up, when dealing with building aggregates attention must be paid to the partial out-of-plane collapses and to the possible presence of churches and bell towers, which are characterized by a higher seismic vulnerability due to their geometry and plan layout. Churches are characterized by big slender naves, whose longitudinal walls are subjected to out-of-plane bending; the presence of adjacent buildings has beneficial effects because the displacement of the longitudinal walls is to some extent constrained. Towers and bell towers, due to their geometry, are characterized by high slenderness, therefore they are subjected to considerable displacements at the top; failure due to bending and compression must be considered. The presence of adjacent buildings may modify the location of the cracks activating the collapse. Another important aspect to be considered is the regularity in elevation; structures regular in elevation are characterized by lower seismic vulnerability, thus it is important to study the seismic behaviour of portions exhibiting irregularities (one typical case is the basilica layout). In aggregates, the effectiveness of the connections between walls, roof and floors are paramount; indeed, local collapse mechanisms may be activated making the structure more vulnerable. The activation of local collapse mechanisms can be identified first using linear kinematic analysis, and then verified through pushover analyses on FE models, in order to estimate with sufficient accuracy the PGA that triggers the mechanism, since the first method could be too conservative.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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