Visual Abstract

Structural and seismic vulnerability assessment of the Santa Maria Assunta Cathedral in Catanzaro (Italy): classical and advanced approaches for the analysis of local and global failure mechanisms

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ABSTRACT. The evaluation of the seismic vulnerability of existing buildings is becoming very significant nowadays, especially for ancient masonry structures, that represent the cultural and historical heritage of our countries. In this research, the Cathedral of Santa Maria Assunta in Catanzaro (Italy) is analyzed to evaluate its structural response. The main physical properties of the constituent materials were deduced from an extensive diagnostic campaign, while the structural geometry and the construction details were derived from an accurate 3D laser scanner survey. A global dynamic analysis, based on the design response spectrum, is performed on a finite element model for studying the seismic response of the structure. Moreover, a local analysis is conducted to evaluate the safety factors corresponding to potential failure mechanisms along preassigned failure surfaces. Furthermore, pushover analyses are performed on macro-elements, properly extracted from the whole structure and with an independent behavior with regard to seismic actions. A novel model based on inter-element fracture approach is used for the material nonlinearity and its results are compared with a well-known classical damage model in order to point out the capability of the method. Finally, the results obtained with the three different models are compared in terms of seismic vulnerability indicators.

KEYWORDS. Historical masonry structures; Pushover analysis; Cohesive finite element models; Damage models; Seismic vulnerability assessment.

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INTRODUCTION

In the last decades, in Italy many seismic events have highlighted that our buildings heritage is manly made of masonry structures that are highly vulnerable to damage and collapse events [1,2]. Moreover, the presence of many hazard seismic zones in the whole Italian country, has emphasized that diffuse operations and retrofitting are necessary to improve the seismic response of the existing buildings and to protect our cultural and historical heritage [3–7].

The seismic vulnerability assessment and the structural rehabilitation of the existing buildings are becoming very significant, especially for complex masonry structures of historical importance. In such cases, the primary aim is to enhance the structural integrity of the building, meanwhile preserving its artistic and historical value.

According to the Italian standard for constructions [8] and the guidelines for cultural heritage [9], an essential step for the vulnerability assessment of historical buildings consists in developing a proper background knowledge of the examined case. The background knowledge aims to (i) define the geometric survey, (ii) reconstruct the historical building process, and (iii) determine the mechanical properties of the materials used for the structural elements [10,11].

The background knowledge of a given building improves with the amount of available data. However, acquiring exhaustive information is often cumbersome, especially for this kind of construction for which in situ investigation must be minimally invasive.

For instance, inaccessible zones of the building may hinder the execution of survey works and/or field tests. For this reason, the Italian code defines three levels of knowledge (LC), named LC1, LC2, LC3, which differ from each other depending upon the quantity and the quality of information collected. LC3 defines the best knowledge level, whereas LC1 the worst. At each LC corresponds a confidence factor (FC) that modifies the mechanical properties of the existing materials. Despite the detailed prescriptions, the operative approach for defining LC varies according to the typology of the structural system of the building. In particular, there are no guidelines for concrete/masonry hybrid structures, which are typical of several historical buildings in Italy, that have suffered from restoration works during the early 1950s. In that period, reinforced concrete (RC) elements have been used extensively to enhance the capacity of the existing buildings, thus modifying the structural behavior. The result is a new structure in which masonries and RC elements work together. This condition further complicates the structural analysis phase.

The Italian Standards for constructions and related guidelines [8,9] define two basic analysis methods. The first is the analysis of the global behavior of the structure, whereas the second is the analysis of its local portions. The former analysis assesses the overall resistance of the building considering the interaction between all the structural elements, while the latter one evaluates local failure mechanisms such as out-of-plane overturning hazards of unconstrained masonry portions. Note that the Italian code prescribes to perform local analyses before evaluating the global behavior of the structure. Moreover, when the structure does not manifest a clear overall behavior, the structural analysis can be carried out by considering an appropriate ensemble of local analyses on macro-elements. Such methods are useful when ancient buildings are studied, in particular for those that are composed by different substructures built in different periods. The Cathedral of Santa Maria Assunta falls within such kind of buildings and local analyses carried out on macro-elements can be also useful to assess the seismic vulnerability.

Although the Italian code admits the use of macro-elements to assess seismic vulnerability, fewer guidelines are reported on how to perform such analysis. An effective analysis methodology should include nonlinearities associated to material damage, especially for masonry elements. This because the behavior of a masonry structure is highly complex, especially for churches and ancient buildings that are usually conceived to resist only to vertical loads and, therefore, are rather vulnerable to seismic actions, also because of the small ductility of masonry walls. As a consequence, the development of efficient approaches aimed to performing the vulnerability assessment of ancient structures is currently desirable.

In the present work, both innovative and standard methods are used for the static and seismic vulnerability assessment of the Santa Maria Assunta Cathedral in Catanzaro (Italy); moreover, the results of alternative structural modeling techniques in terms of seismic vulnerability indicators are compared.

Due to the high complexity of the building and the related uncertainties of its structural behavior, different types of modeling approaches have been adopted, ranging from global to local models (see, for instance, [12–16] and references therein). In particular, according to the classification proposed by D'Altri et al. [17], the most common modeling approaches belong to the following groups: block-based models, continuum models, macro-element models, and geometry-based models. In block-based models masonry is represented at the mesoscopic scale, i.e. at the level of its constituents (typically, only units and dry/mortar joints) [18–20]. Continuum models do not account for any distinction between units and joints, and can be of either phenomenological or homogenization/multiscale kinds [21,22] [23–25]. Macro-element models idealize masonry structures into panel-scale elements (typically, piers and spandrels), also including commonly used equivalent frame models [13,26,27]. Finally, geometry-based models refer to rigid-plastic models to be used in the framework of limit analysis (based

on either static or kinematic theorems) [28,29] [30,31]. In principle, all the modeling strategies belonging to the first three groups could be used to perform different numerical analyses, including both linear/nonlinear static and dynamic analyses based on 3D models, whereas the fourth group is mainly used to perform the analysis of local mechanisms along predefined failure surfaces. The last group of modeling approaches is usually required for taking into account all possible partial failures of existing masonry structures (via the so-called linear kinematic analysis), especially in the presence of weak connection between adjacent structural elements. It is useful to note that, generally speaking, global modeling approaches usually requires the adoption of efficient numerical solution strategy (see, for instance [32], in the framework of base-isolated structures), while local models are often associated with simple closed-form analytical solutions. Nevertheless, the combined adoption of different modeling approaches provides a useful strategy to identify the more conservative results in terms of structural safety.
As a matter of fact, the actual structural behavior of ancient masonry buildings, characterized by complex geometry and with different types of sub-elements built over the years, is affected by several uncertainties such as those arising from the variability in geometric and mechanical properties of materials and structural elements and the degree of connection between the various structural elements.
To this end, firstly a global analysis based on 3D finite element model of the structure has been performed; this kind of analysis is significant owing to the presence of masonry and reinforced concrete elements interacting each other. Then local analyses on single masonry walls or on macro-elements have been performed. In the former case a linear kinematic methodology is adopted, whereas in the latter one pushover analyses [33] are performed also based on a novel model to account for material damage in masonry structures.
The layout of the paper is the following: firstly, a description of the structure and its historical analysis are presented. Secondly, the model used for the linear analysis is introduced. Then, the nonlinear analysis is presented and the different nonlinear material models adopted are illustrated, with a special attention devoted to an innovative diffuse interface model (DIM) based on the cohesive/volumetric finite element approach. After this, the numerical results are illustrated, together with the procedure adopted for the evaluation of the Cathedral's structural safety. Finally, a brief discussion about the outcomes of the proposed methods is presented, as well as some concluding remarks.

DESCRIPTION OF THE CASE STUDY

The old historical center of the city of Catanzaro, in southern Italy, is made of many ancient buildings and churches, built over the centuries from the different cultures and people that have lived here. Indeed, in the city center there are several churches built in different phases among which the Cathedral of Santa Maria Assunta (Fig. 1(A)). The structural conformation of these types of structures makes them vulnerable to earthquake actions [34].
In this study, the approach adopted for assessing the seismic vulnerability of Santa Maria Assunta Cathedral is that prescribed by the Italian standards for constructions [8], together with the related explicative notes [35], and the Italian Guidelines for Cultural Heritage [9]. Both these codes provide detailed guidelines to (i) get adequate background knowledge of the building, (ii) perform vulnerability analyses, and (iii) enhance structural performances without compromising the heritage value.
In the sequel, a detailed description of the data regarding the building survey, the historic evolution, and the material properties of structural elements of the Cathedral is presented. According to the amount of collected information, the knowledge level (referred to LC in the Italian) and the corresponding confidence factor (FC in the Italian Code) are defined.

(A) (B)

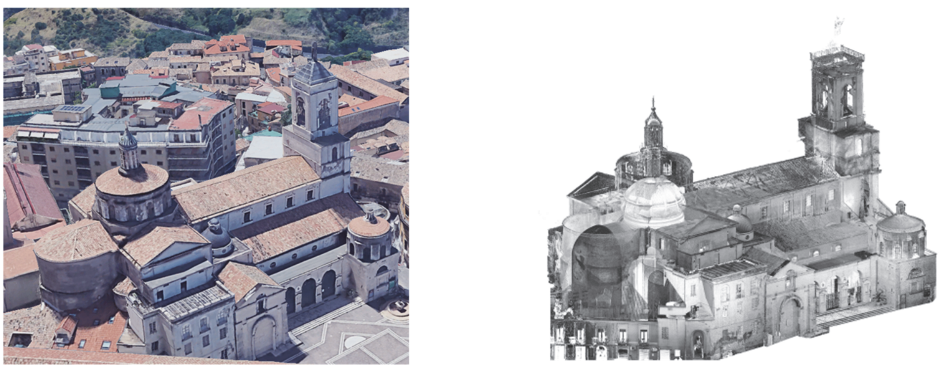

Figure 1: S. Maria Assunta Cathedral: general view (A) and 3D model derived from laser scanner survey (B).

Geometric survey

The geometry survey of an ancient and complex building is a challenging task because of various issues, such as the presence of articulated shape geometries, the differences in the size of structural elements, and the difficulties of correctly identifying all the portions of the building.

The survey of the Cathedral has been performed through a 3D laser scanner procedure, which involves the construction of a dense 3D points cloud of the building (see for instance [36]). Because of the massive size of the Cathedral, different locations of the laser scanner have been considered, thus recording many clouds of points. The achieved clouds have been subsequently converted to triangular meshes, forming the surfaces of the sampled parts (Fig. 1(B)). Figs. 2 and 3 show the output of the triangular mesh operations obtained via laser scanner procedure, useful to achieve a preliminary representation of the geometric schemes of the whole structure. Finally, triangular meshes have been transformed into geometric entities, thus providing the final geometrical representation of the Cathedral. Besides, a thermographic analysis has been performed to recognize discontinuities in the masonry walls and to check the presence of reinforced concrete (RC) elements embedded in masonry structures. The actual configuration of the Cathedral is different from the original structures, that exhibited a Latin cross plan. These differences are present due to the several events that changed the structural system over the decades, such as the earthquake of the 1783 and the damage caused by the Worl War II bombing.

Figure 2: Vertical section derived from the 3D laser scanner survey.

(A) (B)

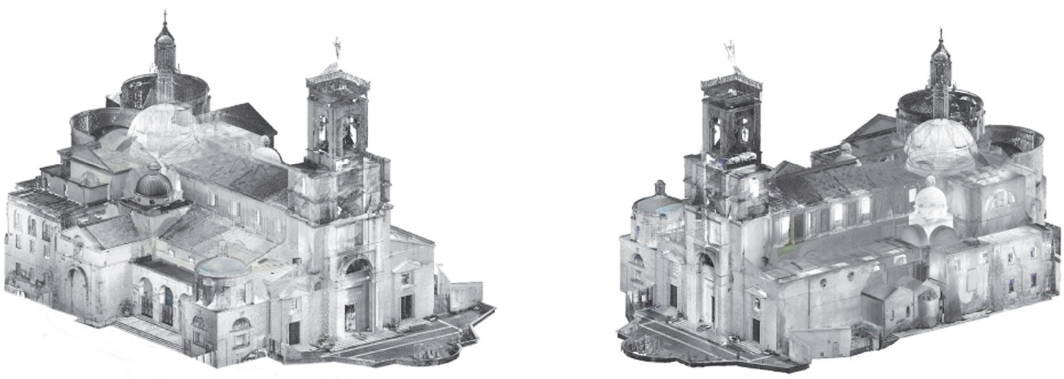

Figure 3: North-west (A) and South-west (B) general view derived from the 3D laser scanner survey.

The central nave is 30.70 m long and 10.95 m large, whereas the transept is rectangular with a length of about 14.00 m and a width of 13.00 m. Both the central nave and the transept have a height of 19.60 m and are covered by a gable roof made of reinforced concrete truss structures. The intersection of the central nave with the transept defines the core of the

Cathedral, where the main altar is located. This central zone is covered by a massive dome, made of reinforced concrete and masonry.

Both the lateral naves are of 22.70 m length, 7.70 m width and 9.40 m height, covered by a single-pitched roof made of reinforced concrete truss structures. Externally, they are flanked by several chapels; three are on the right side and one is on the left. In particular, the one on the left side is dedicated to San Vitaliano, the patron saint of Catanzaro. Along the left side, there are also a baptistery and a portico made of reinforced concrete.

In the position currently taken by the baptistery there was, in the original structures built during the 1000s, the bell tower with a carrying masonry structure. After the damage caused by the War (see Fig. 4), its structure has been destroyed and then rebuilt in the actual position over the West façade. On the top of the tower, we find the statue of Santa Maria Assunta. This is the highest point of the church, with a height of 41.5 meters.

Historical evolution of the Cathedral

According to many historical studies, the Cathedral was built during the XII century. The original plan was a Latin cross with three naves. Such a layout was like that of several churches built in that period. Also, in the original structure, there were only two chapels placed symmetrically regarding the longitudinal axis of the building along the lateral naves. The original structure of the Cathedral presented a bell tower located on the north side in the position where today the baptistery is placed, with an independent structure from the rest of the building. Both the structures of the church and the tower were entirely in masonry.

Throughout history, several unfortunate circumstances have altered the original plan of the Cathedral. Various parts of the structure have suffered from massive collapse because of seismic events. The most devastating one occurred in 1783 and caused the failure of the entire left nave. Subsequent reconstruction works have permitted to repair damaged parts, thus enabling the use of the church for many years afterward.

In 1943, the building suffered other massive damages because of the aerial Anglo-American bombing. The raids destroyed the left nave of the church and the bell tower (see Fig. 4). In the second half of the 40s, the architects Fasolo and Domestico were commissioned to design the reconstruction project, that was approved in 1949. New reinforced concrete structures embedded in the ancient masonry structures were built, and RC bond-beams were constructed over the masonry walls in order to improve their structural behavior with regards to seismic loads. Further, to enhance the robustness of the building against local strength failure, crumbling masonry walls were re-built entirely. The reconstruction works provided a new bell tower made of RC beams and columns integrated into the masonries and placed on the west side, incorporated in the façade of the Cathedral. The project included new structures. These were a baptistery and a portico next to the left nave, and a windowed dome over the altar zone.

Figure 4: Left nave of Cathedral destroyed by World bombing. Calabria superintendency storage.

Mechanical properties of materials structure

The material properties of the structural elements are determined through specific in situ tests for both masonry and RC structural elements.

The analysis on masonry elements comprises visual inspections, flat jack tests, endoscopic tests, and "Darmstadt" tests on the masonry joints. Such investigations permit identifying all the masonries of the Cathedral, which are illustrated in Fig. 5:

- Disordered rubble stone masonry (Fig. 5A);
- Solid brick and lime mortar masonry (Fig. 5B);
- Squared block stone masonry (Fig. 5C);
- Soft stone ashlar masonry (Fig. 5D);
- Irregular soft stone masonry (Fig. 5E).

RC structures have been investigated using compressive tests on concrete cores and tensile tests on steel bars according to guidelines reported in the Italian Standards (par. C8.5.IV) [8] and corresponding explicative notes [35]. Concrete cores and steel bar samples have been extracted from RC structures in restricted portions of the building. This strategy has led to limited knowledge of the mechanical properties of materials but has avoided damaging several portions of the building, thus promoting heritage value preservation. Such an approach is also recommended by the Italian guidelines on Cultural heritage, which admits poor levels of knowledge instead of performing invasive tests.

The sampled data have been examined using the formulations reported in the Italian standards for constructions to define an average value of the mechanical properties of both concrete and steel to be adopted in the numerical analyses.

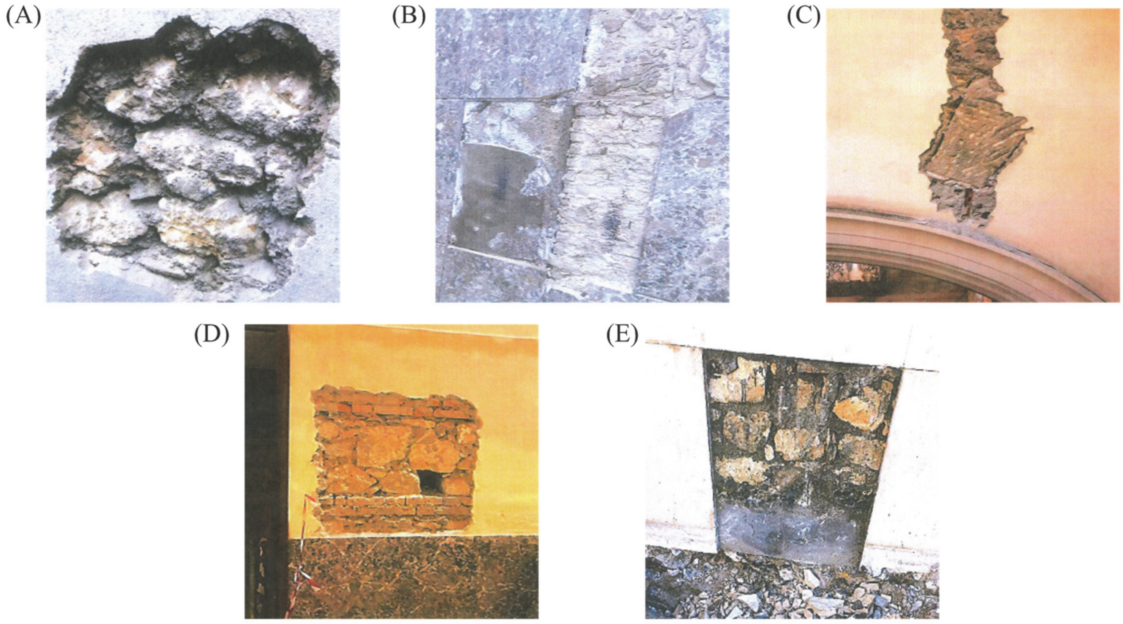

Figure 5: Masonry types derived from on-site survey.

Definition of the knowledge level

As described in previous sections, the Cathedral is an articulated building involving both masonry and RC structural elements. Such a peculiar feature complicates the definition of the knowledge level of the building essentially for two major reasons. First, Italian standards provide different prescriptions about confidence factors for masonries and RC elements. Second, there are no guidelines concerning hybrid structures made of both masonries and RC elements. Therefore, the vulnerability assessment for the case under investigation has required adopting a combined approach.

The proposed strategy evaluates the confidence factors for masonries and RC elements according to the procedures reported in the standards. Then, the worst value is assumed as a reference confidence factor.

For masonries, the method reported in chapter 4.2 of the Italian guidelines for cultural heritage serves as a reference [9]. Such approach defines the confidence factor (FC) using an algebraic summation of four partial scores depending upon the quality and quantity of background information about the geometric survey (C1), history (C2), mechanical properties of masonries (C3), and foundation (C4) of the building:

$$FC_M = 1 + \sum_{k=1}^{4} FC_M^{Ck} \tag{1}$$

with $FC_M^{C1} = [0,\ 0.05]$, $FC_M^{C2} = [0,\ 0.06,\ 0.12]$, $FC_M^{C3} = [0,\ 0.06,\ 0.12]$, and $FC_M^{C4} = [0,\ 0.03,\ 0.06]$.

According to Eq. (1), the confidence factor can vary between 1 and 1.35, which correspond to LC3 and LC1 level of knowledge, respectively. Note that the proposed approach offers great flexibility due to the different values of the previous interval. The recorded data on masonries have led to.

The mechanical properties of masonries adopted for the parameters are taken in agreement with the Italian Code [8], assuming the average values between lower and upper bound of the strength and the elastic moduli suggested in Tab. C8.5.I. For Reinforced Concrete elements, the approach reported in Italian standards for constructions [8] and related explicative notes [35] is adopted. Although there is substantial background information about geometry survey and historical evolution, the limited on-site tests for sampling mechanical properties of materials have negatively affected the knowledge level, thereby involving LC2 and FC_{RC} = 1.2. The mechanical parameters for concrete and steel have been sampled from 46 cores and 12 bars extracted from several elements of the building. The mechanical properties of the samples have been evaluated through destructive tests thus achieving a collection of data. The representative value of Young modulus's and strength have been assumed as the averages of the collected data.

However, it is worth noting that during the on-site survey many parts of the building were not investigated, hence more accurate investigations are necessary to grow the knowledge level and the accuracy of the structural analyses. Moreover, the knowledge of existing buildings and their structural analyses are related to each other, and integrative surveys should be done based on the Cathedral structural behavior.

ANALYSIS METHODOLOGIES

The present paper provides a comparative analysis for the seismic vulnerability assessment of the Cathedral, relying on the use of different analysis methodologies (also used in combination to each other), with the final aim of identifying the most vulnerable structural elements with respect to seismic actions. The following analysis methodologies have been used:

- Linear static and dynamic analyses performed on a global 3D model;
- Nonlinear static analysis performed on individual macro-elements;
- Analysis of local failure mechanisms through a linear kinematic approach.

The latter analysis is mainly aimed at assessing the structural safety against the so-called out-of-plane overturning failure modes of both single and grouped masonry elements. This approach is recommended by the Italian code to be performed prior to global (and macro-element) analyses, in order to identify and eventually remove all the brittle failure mechanisms potentially activated under seismic actions, which usually are not taken into account when the first two analysis methodologies are adopted.

After this preliminary step, a global analysis is performed on a fully 3D model to investigate the overall structural response of the Cathedral under both static and dynamic loads, and to evaluate its modal response (in terms of both natural frequencies and related vibration shapes). Despite being a very efficient numerical tool, this analysis methodology is known to provide only approximate results for ancient masonry structures. In facts, global models usually neglect common features of historical masonry constructions, such as the absence of good connection between the masonry walls or the presence of different slabs with a finite stiffness in their own plane, which make the global analysis not always suitable for capturing the actual structural behavior. To avoid these issues, the Italian guidelines for the cultural heritage [9] suggest the use of different analysis methodologies, including the nonlinear static analysis, also referred to as pushover analysis, to be performed on individual macro-elements properly extracted from the global model and with an independent behavior with reference to earthquake-induced horizontal actions. In this paper, this analysis methodology has been used to analyze more in depth the structural behavior of two macro-elements, which are representative of the transverse and longitudinal behavior of the Cathedral against seismic actions. As an important aspect of novelty of the present work, an innovative modeling approach has been adopted for the pushover analyses, relying on a cohesive/volumetric finite element method, which has been proposed and subsequently refined by some of the authors in Ref. [37]. Such an approach has the advantage of allowing complex crack patterns to be naturally predicted, by embedding suitably calibrated cohesive interfaces along all the internal boundaries of the bulk mesh used to represent the structural element under investigation. This novel approach has been validated in this work by providing a comparison with a well-established damage model, which assumes an approximate smeared representation of fracture phenomena.

Global analysis of the structure

Starting from a fully 3D finite element model ad hoc developed for the Cathedral, a linear dynamic analysis has been performed using a commercial software for structural analysis and design. This model adopts shell elements for masonry members, relying on a Kirchhoff-Love formulation for the out-of-plane flexural behavior, as well as frame elements for reinforced concrete members, incorporating a Timoshenko formulation for the bending response.

An important detail in the structural modeling of ancient masonry buildings is the correct simulation of the real connection between the transversal walls. Indeed, if this aspect is not adequately captured, an independent behavior of the structural walls is predicted, so that they are exposed to brittle failure mechanisms, such as their out-of-plane overturning induced by horizontal forces. According to the results of in-situ surveys and tests, a good connection between the structural walls has been assumed for the present 3D model. The hollow clay pot slabs of chapels and sacristy are modeled as plate elements. Note that the Italian standards admit the modeling of such slab typologies as elements having infinite in-plane stiffness. Such a modeling approach usually adopts rigid-diaphragm constraint equations between the top nodes of the walls able to account for the infinite in-plane stiffness behavior. However, in the present structural model, slabs are modeled as elements with a finite stiffness in their own plane. This because the results of in-situ tests and surveys have shown that the existing slabs are not in good condition, due to their old age as well as to the diffuse deterioration of their structural materials. Moreover, all the material properties have been assigned to the model by using the confidence factor (FC) evaluated at the end of the background analysis. Indeed, in the vulnerability assessment of existing buildings through both response spectrum dynamic and nonlinear static analysis, the confidence factor is used as additionally safety factor, in addition to partial coefficient of the material, for the evaluation of the brittle mechanisms (e.g., shear failure), and as the only safety factor for the assessment of ductile mechanisms (e.g., bending failure).

Tab. 1 shows the properties of the different masonry typologies developed in the structure during the on-site survey. The reinforced concrete members have been characterized via the results of the tensile test on the steel bars and the output of the compressive study on the concrete samples.

Material	E (MPa)	G (MPa)	ν (-)	γ (kN/m^3)	f_c (MPa)	τ_0 (MPa)	f_{v0} (MPa)
Disordered rubble stone masonry	1305	435	0.25	19.0	2.92	0.049	-
Solid bricks and lime mortar masonry	2067	689.05	0.25	18.0	4.75	0.124	0.276
Squared block stone masonry	3420	1140	0.25	22.0	8.40	0.126	0.277
Soft stone ashlar masonry	2256	720	0.25	14.5	4.16	0.096	0.232
Irregular soft stone masonry	1620	540	0.25	14.5	3.24	0.063	-

Table 1: Mechanical parameters of the five masonry types involved in the global model.

A reinforced concrete with a value of 15 MPa for the average compressive strength, has been considered for the structural elements according to the results of the considered samples.

Based on the necessity of overcome the uncertainty on the mechanical behavior of the masonry connection-elements, another model has been created, in which the masonry arches were modeled like masonry spandrels and not with their effective geometry (see Fig. 6).

Despite missing the pushing contribution, characteristic of arch, such model should be suitable for the axial-bending analysis of the masonry structure, highlighting eventually deficits and issues.

The geometric dimensions of the coupling beams using in this model, have been evaluated with an area equivalence between the arches and the beams elements. It is important to note that the Italian Code [8] provides the conditions that define the masonry spandrels as coupling beams (see Section 7.8 of the Italian Code).

In addition to the FE model of the Cathedral, a preliminary analysis of the soil-structure interaction has been performed analyzing the geotechnical assessment of the structure, even if in a preliminary manner and not reported in this paper for the sake of brevity. It is important to note that this type of assessment, according to the Italian Code [8], is mandatory only if local failure, caused by settlements foundation, or global instabilities of the whole structure are present.

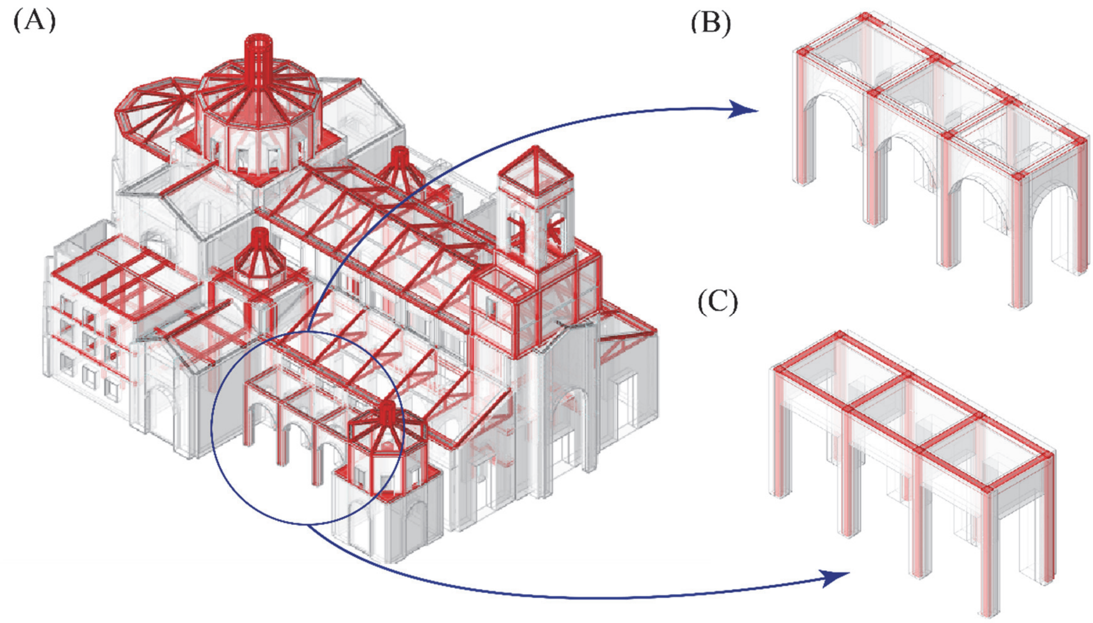

Figure 6: Schematic example of the two models defined for the global analysis: 3D global model (A); detail of the arcade modeled with masonry arches (B) and masonry spandrels (C).

Starting from the global model, structural analysis and assessment were performed for different load combinations, including seismic and no-seismic load cases.
The first load combination was a non-seismic load case, known as Ultimate Limit State (ULS) combinations (see EC2 [38]), that includes only the permanent imposed loads and the service-imposed loads, without considering the other variable actions (i.e., snow on roofing or wind actions). This first approach provides useful information on the actual state of the building, highlighting eventually preliminary issues already for the human-controlled load. Moreover, using this procedure, it is possible to reduce the use of the structure and prevent unsafety situations. Subsequently, complete non-seismic load combinations were defined including snow, wind, and thermal variation actions.
On the other hand, the response of the whole structure for seismic load combinations were analyzed based on the classical combination case that includes the partial factors of the different variable loads (see Tab. 2.5.I of the Italian Code [8]).

Nonlinear static analysis of macro-elements

Complex buildings frequently comprise different sub-units of various geometric configurations and material compositions that behave differently from each other from a structural point of view. Such a condition occurs also for the case under investigation because the architectonical portions of the Cathedral (i.e. naves, transept, chapels, apex, and bell tower) are distinct sub-structures that are aggregated to each other. These portions can be regarded as the structural macro-elements of the building. Hence, the overall seismic vulnerability of the Cathedral can be assessed through the analysis of its individual macro-elements. These macro-elements represent the parts of the Cathedral whose behavior is independent from the whole structure with respect to the seismic forces, especially in terms of their collapse mechanism.
Italian Guidelines for the historical and cultural heritage [9] (chapter 5.4.3 and Annex C) do not provide detailed recommendations to analyze the different macro-elements of historic buildings. For church buildings, these guidelines only define the most probable collapse mechanisms of usual macro-element geometries, but they do not provide exhaustive information about the choice of failure mechanisms in more complex structural configurations.
In the present paper, two macro-elements are extracted from the global 3D model described in the previous section, and subsequently considered for a nonlinear static (i.e., pushover) analysis, as shown in Fig. 7. The first element (referred to as longitudinal macro-element) has the length of almost the whole structure and contains the narthex, the colonnade, and the final part of the longitudinal section of the structure (excluded the circular apse), as depicted in Fig. 7(A). The second one (called transverse macro-element) is selected from the lateral section of the church and corresponds to the transept (see Fig. 7(B)). These two macro-elements have been chosen to be representative of the seismic response of the Cathedral in the longitudinal and transverse directions, respectively. However, for the sake of brevity, only the numerical results related to

the transversal macro-element have been reported, which is believed to be more vulnerable due to its peculiar configuration and location with respect to the other structural elements of the Cathedral. This macro-element involves two different masonry types, i.e. irregular soft stone masonry for the two lateral portions, and squared block stone masonry for the central part. The inelastic parameters required to perform the nonlinear static analysis by using the two models presented in the following section are reported in Tab. 2. In particular, the strength parameters (tensile strength f_t, uniaxial and biaxial compressive strengths, f_c and f_b) were deduced starting from the average compressive strength reported in the Italian code [35], whereas the fracture energies G_f are assumed on the basis of typical values adopted in the technical literature for these masonry types.

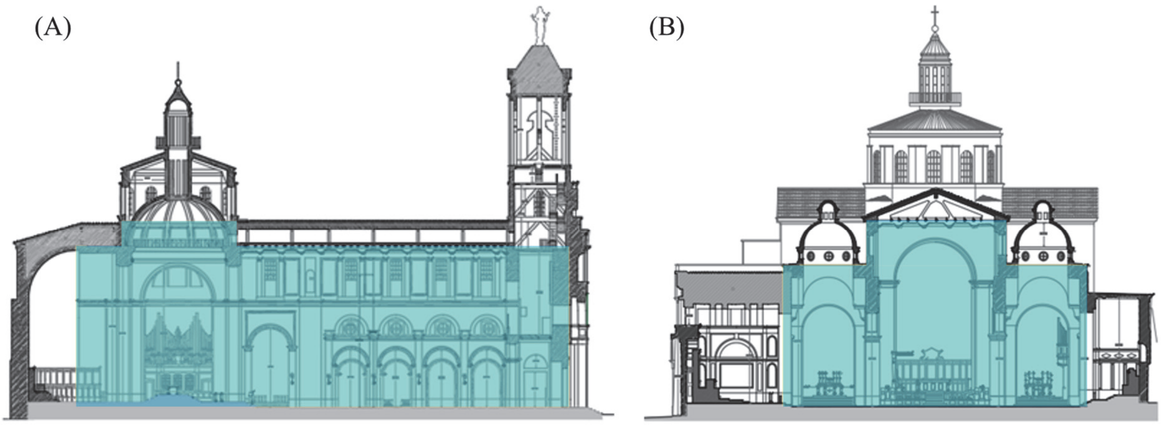

Figure 7: Vertical section generating from the global geometrical model: longitudinal macro element (A) and transverse macro element (B).

Material	f_t (MPa)	f_c (MPa)	f_b (MPa)	G_f (J/m²)
Irregular soft stone masonry	0.095	3.24	3.89	100
Squared block stone masonry	0.189	8.40	10.08	500

Table 2: Inelastic parameters of the masonry elements.

Limit analysis of local collapse mechanisms

In addition to the global analysis of the structure, the seismic vulnerability assessment of an existing masonry building also requires the study of local collapse mechanisms, that are local portions of the structure that are susceptible to rigid overturning falls under moderate seismic actions. Indeed, most of the existing masonry structures present several vulnerable portions, such as unconstrained perimeter bearing walls (because of the lack of orthogonal connections) or soaring sub-structures ready to fail because of equilibrium losses rather than strength exceedance. For an existing masonry building, the equilibrium loss of local portions represents undoubtedly an unsafe condition. In extreme cases, the sudden loss of a local masonry portion can trigger a sequence of consecutive local collapse mechanisms up to the fall of the entire structure. Hence, the vulnerability of all unrestrained elements of a masonry structure against overturning risks represents an essential investigation, which is also helpful in achieving timely information for adopting proper retrofitting strategies to avoid them (including those based on the external application of FRP-like systems [39]).

The study of local collapse mechanisms consists of comparing the horizontal load capacity of all the vulnerable portions of the masonry structure with the seismic demand of the building site. To this end, the Italian Codes prescribe using the kinematic approach of limit analysis to evaluate the horizontal capacity of a vulnerable masonry element, under the assumption for the masonry of being (*i*) rigid in compression, (*ii*) with no tensile strength, and (*iii*) unsusceptible to sliding failures.

Quantifying the horizontal capacity of a masonry portion via the kinematic approach comprises several consecutive steps. The analysis begins by extracting the vulnerable element from the building, together with all the vertical and horizontal forces acting on it (i.e., all the actions transmitted by the structure, that are, for instance, all floor and service loads, and the trust of adjacent masonry arches). The subsequent step involves introducing a sufficient number of kinematic hinges on the extracted masonry portion to configure a single-degree-of-freedom system capable of reproducing a potential collapse mechanism. Note that the location of the kinematic hinges strictly depends on the investigating structural element. For instance, for examining the seismic vulnerability of an unrestrained perimeter masonry bearing wall (i.e., out-of-plane overturning), a single kinematic hinge is usually placed at the base of the wall itself. Next, an additional set of horizontal forces is imposed on the investigating masonry portion devoted to reproducing the effect induced by seismic actions. They are usually expressed as the product between the vertical loads acting on the masonry element and a load multiplier factor (α_0). Note that the horizontal load multiplier α_0 represents the commonly adopted parameter for quantifying the capacity of a local masonry portion against overturning hazards. Finally, the last step of the kinematic approach involves the evaluation of α_0 through the analysis of the equilibrium state by the classic relationship of the Principle of Virtual Work:

$$\alpha_0\left(\sum_{i=1}^{n} P_i \cdot \delta_{x,i} + \sum_{j=n+1}^{n+m} P_j \cdot \delta_{x,j}\right) - \sum_{i=1}^{n} P_i \cdot \delta_{y,i} - \sum_{h=1}^{o} F_h \cdot \delta_h = L_{fi} \tag{2}$$

where P_i is the weight force of the i-th element forming the local masonry portion, P_j is the j-th weight force not directly applied on the element whose mass generate horizontal loads, and F_h is the h-th external load. Besides, $\delta_{x,i}$, $\delta_{x,j}$ and δ_h are the virtual displacements of P_i, P_j and F_h, defined according to the assumed collapse mechanism. Finally, L_{fi} is the work of the internal loads.
The horizontal load multiplier evaluated through Eq. (2) is usually converted into the ground acceleration threshold (a_0^*) that activates the considered local collapse mechanism through the following relationship provided by the Italian Codes:

$$a_0^* = \frac{\alpha_0 g}{e^* FC} \quad \text{with} \quad e^* = \frac{gM^*}{\sum_i P_i} \quad \text{and} \quad M^* = \frac{\left(\sum_i P_i \delta_{x,i}\right)^2}{g\sum_i P_i \delta_{x,i}^2} \tag{3}$$

where g is the gravity acceleration, FC is the confidence factor associated to the knowledge level, e^* is the participating mass ratio, and M^* is the participating mass involving in the mechanism.
To assess the vulnerability of the local masonry portion, the acceleration capacity a_0^* is compared with the seismic demand, which is either the peak ground acceleration of the building site (when the element lies on the ground floor) or the action derived by using a floor response spectrum approach provided by the Italian codes, when the investigated masonry portion is located at a Z-height of the building.

A REFINED DIFFUSE INTERFACE MODEL FOR THE NONLINEAR ANALYSIS OF MASONRY MACRO-ELEMENTS

As a key novelty point of the present paper, a refined Diffuse Interface Model (DIM) is proposed for the nonlinear analysis of masonry macro-elements. Such a fracture model is based on the insertion of cohesive interface elements along all the interelement boundaries of the computational mesh, thus allowing arbitrary crack paths to be accurately predicted. This strategy can be regarded as an alternative to well-established approaches, based on either global/local remeshing techniques or node relocation approaches (including some recent moving mesh methodologies [40,41]). The adopted model, proposed in Ref. [42] and successfully used in Refs. [43,44] to simulate crack propagation in different classes of homogeneous and heterogeneous materials, is here adapted to the case of 2D masonry substructures (here referred to as macro-elements) modeled at the macroscopic scale.
As a further aspect, the proposed DIM approach is validated through a suitable comparison with a well-established regularized damage model, incorporating a Rankine-type failure criterion (according to the maximum principal stress theory) for tensile damage onset, as well as an exponential softening law for damage evolution.
Both modeling approaches have been implemented in a standard finite element setting, within the commercial simulation environment COMSOL Multiphysics [45]. Such a computational tool has been chosen by virtue of its advanced scripting capabilities, required for the practical development and validation of the proposed DIM approach. In the following, an overview of these two models is reported, providing their theoretical background, together with some computational details.

Diffuse Interface Model

The proposed Diffuse Interface Model is based on the cohesive/volumetric finite element approach, proposed a few decades ago [42] and successfully adopted in different models [46], according to which a finite set of zero-thickness interface elements are preinserted along the boundaries of a standard finite element mesh. Such an approach is here followed, by considering these main assumptions: quasi-static loading, small deformations, linearly elastic bulk materials and purely cohesive mesh boundaries (i.e., no frictional neither plastic behavior is accounted for). This model, which has been widely used by some of the authors for the failure simulation of different materials, including plain and reinforced concrete, fiber-reinforced concrete, and concrete enhanced with embedded nanomaterials [46,47], is able to accurately predict arbitrary crack patterns experiencing during the quasi-brittle damage evolution.

Starting from the 2D continuum depicted in Fig. 8(A), occupying the region Ω and delimited by the boundary Γ, its discretized version (denoted by the subscript h) with embedded discontinuity lines Γ_h^{int} coinciding with the internal mesh boundaries and representing the potential crack paths, is used for the simulation of a fracturing body (see Fig. 8(B)). Such a body is subjected to applied tractions $\bar{\boldsymbol{t}}$ on Γ_N as well as to prescribed displacements $\bar{\boldsymbol{u}}$ on Γ_D, the subscripts N and D indicating the Neumann and Dirichlet portions of Γ, respectively). The mechanical response of the bulk phase is assumed to be linearly elastic and isotropic, so that the unique nonlinearity source is the constitutive behavior of the embedded cohesive interfaces. Such a constitutive behavior is written as a damage-driven traction-separation law, in terms of the cohesive traction vector $\boldsymbol{t}_{\text{coh}} = \boldsymbol{t}_{\text{coh}}\left(\llbracket \boldsymbol{u} \rrbracket\right)$ being, in turn, a function of the displacement jump vector $\llbracket \boldsymbol{u} \rrbracket$. Such a law is conveniently written with respect to a local reference system, defined for each pair of adjacent bulk elements Ω_h^+ and Ω_h^- by the unit vectors $\boldsymbol{s}$ and $\boldsymbol{n}$, the latter being oriented outward with respect to Ω_h^+. In such a way, these cohesive interfaces behave as nonlinear spring beds involving both the tangential $\llbracket u_s \rrbracket$ and the normal $\llbracket u_n \rrbracket$ components of the displacement jump vector $\llbracket \boldsymbol{u} \rrbracket$ (see Fig. 8(C)).

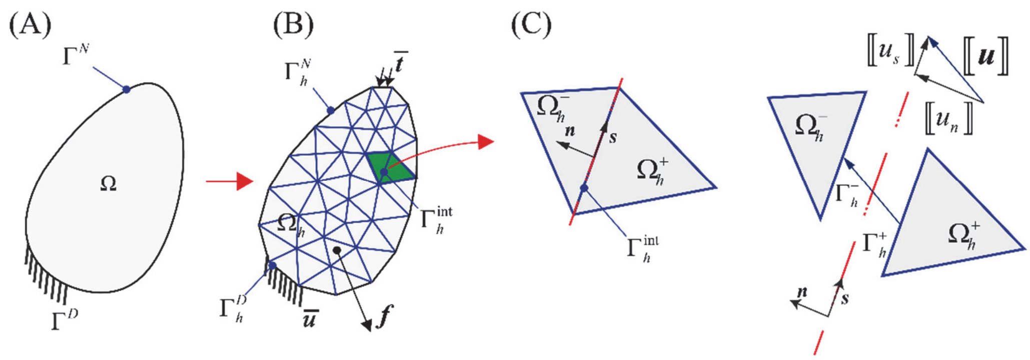

Figure 8: Schematic representation of the DIM approach. Continuum unfractured body (A); discretized body with embedded discontinuity lines (B); zero-thickness cohesive interfaces element (C).

As widely known from the literature, the structural performance of masonry structures is highly influenced by the nonlinear behavior of its constituents, thus resulting in a globally anisotropic behavior in both elastic and post-elastic range [24,48]. In this study, a degenerated Drucker-Prager criterion has been used for the macroscopic failure analysis of masonry structures (see for instance [49]). According with the works reported in Ref. [50], the following traction-separation law is considered:

$$\boldsymbol{t}_{\text{coh}} = (1-d)\boldsymbol{K}\llbracket \boldsymbol{u} \rrbracket \tag{4}$$

where $\boldsymbol{K}$ is the initial stiffness matrix (containing the normal and tangential stiffness constants in its principal diagonal), and d is the damage variable, expressed as a function of the maximum effective displacement jump over the entire loading history, denoted as $u_{\max}$, which depends on the tensile strength f_t of masonry, as well as on its uniaxial and biaxial compressive strengths, f_c and f_b, respectively (see [50] for additional details). In particular, the following expression for d is considered:

$$d = \begin{cases} 0 & if\ u_{\max} < u_0 \\ 1 - \dfrac{u_0}{u_{\max}} \exp\left[\beta\left(1 - \dfrac{u_{\max}}{u_0}\right)\right] & if\ u_{\max} \geq u_0 \end{cases} \tag{5}$$

where $\beta = \dfrac{1}{\dfrac{G_f}{f_t u_0} - \dfrac{1}{2}}$ is a dimensionless parameter governing the brittleness of the cohesive response, f_t and G_f are the tensile strength and the mode-I fracture energy of masonry, respectively (both assumed to be independent of the crack orientation with respect to the material axes), while u_0 is the normal displacement jump at damage onset. For more details about this constitutive model, the reader is referred to the work [50].

Regularized damage model

As widely known from the technical literature [51–53] damage models are well-established approaches to incorporate the effects of material degradation into the constitutive behavior of quasi-brittle materials at the macroscopic scale, including masonry structures [54,55]. Is it also well-known that the application of local damage mechanics theories to softening materials may result in a spurious mesh sensitivity and a convergence towards physically inadmissible solutions, as consequences of the loss of ellipticity and of the resulting well-posedness of the underlying equilibrium equations. Therefore, a simple regularized isotropic damage model has been used in the present paper for comparison purposes. The related constitutive relation is written as $\boldsymbol{\sigma} = (1-d)\boldsymbol{C}:\boldsymbol{\varepsilon}$, where $\boldsymbol{C}$, $\boldsymbol{\varepsilon}$ and $\boldsymbol{\sigma}$ are the elastic stiffness tensor, the strain and the stress tensors, respectively. The scalar quantity d denotes the damage variable, which depends on the maximum level of the Rankine equivalent strain, ε_{eq}, achieved during the entire deformation history, here denoted by κ and playing the role of internal state variable, according to the following evolution law:

$$d = \begin{cases} 0 & \text{if } \kappa \leq \varepsilon_0 \\ 1 - \dfrac{\varepsilon_0}{\kappa} \exp\left(-\dfrac{\kappa - \varepsilon_0}{\varepsilon_f - \varepsilon_0}\right) & \text{if } \kappa > \varepsilon_0 \end{cases} \tag{6}$$

where $\varepsilon_0 = ft/E$ is the limit tensile elastic strain, f_t and E being the tensile strength and the Young's modulus, respectively, whereas ε_f denotes the ultimate strain, defined as

$$\varepsilon_f = \frac{\varepsilon_0}{2} + \frac{G_f}{h_{cb} f_t} \tag{7}$$

which controls the slope of the softening curve. In Eq. (7), G_f is the energy dissipated per unit area at complete failure (i.e., the fracture energy), and h_{cb} is the crack bandwidth, coinciding with the size of the numerically resolved localized damage. For the adopted regularization approach, relying of the crack band concept, this size is strictly related to the size, shape and orientation of finite elements. In this work, only 2D triangular elements have been employed, so that it has been always assumed as $h_{cb} = \sqrt{2A}$, A being the bulk element area.

The resulting stress-strain constitutive diagram under pure tensile loading is characterized by a linearly elastic behavior up the tensile peak, followed by an exponential softening branch so that the area under this diagram represents the fracture energy per unit volume $g_f = G_f/h_{cb}$.

It is worth noting that the adopted damage model requires fewer physical parameters than the diffuse interface one, since only the tensile strength of masonry appears in the damage onset criterion.

NUMERICAL RESULTS

In this section, the results of the numerical simulations are illustrated. In particular, the main outcomes obtained via the different modeling approaches are reported, together with their critical discussion.

Response spectrum analysis on the global model

The linear dynamic analysis indicated the Italian Standards relies on the evaluation of the modal properties of the entire structure, and subsequently, of its seismic response by means of the response spectrum technique. According to the Italian Standards, the seismic response of the Cathedral is predicted by considering the first 30 modes obtained from the generalized eigenvalue problem, assuring that the cumulated participating mass ratio is greater than 85% for two horizontal ground motion components. The period and the participating mass ratio of the first ten natural modes are reported in Tab. 3. The associated mode shapes are shown in Fig. 9. It can be observed that modes 3 and 4, whose shapes are predominantly translational in nature and involve almost the entire structural masses, are associated with the greatest values of mass participation ratios in Y and X directions, respectively. Instead, modes 1 and 2 involve only the motion of the bell tower along X and Y directions, respectively. The remaining modes have almost negligible mass participation ratios, being associated with local deformed shapes with no dominant horizontal translations.

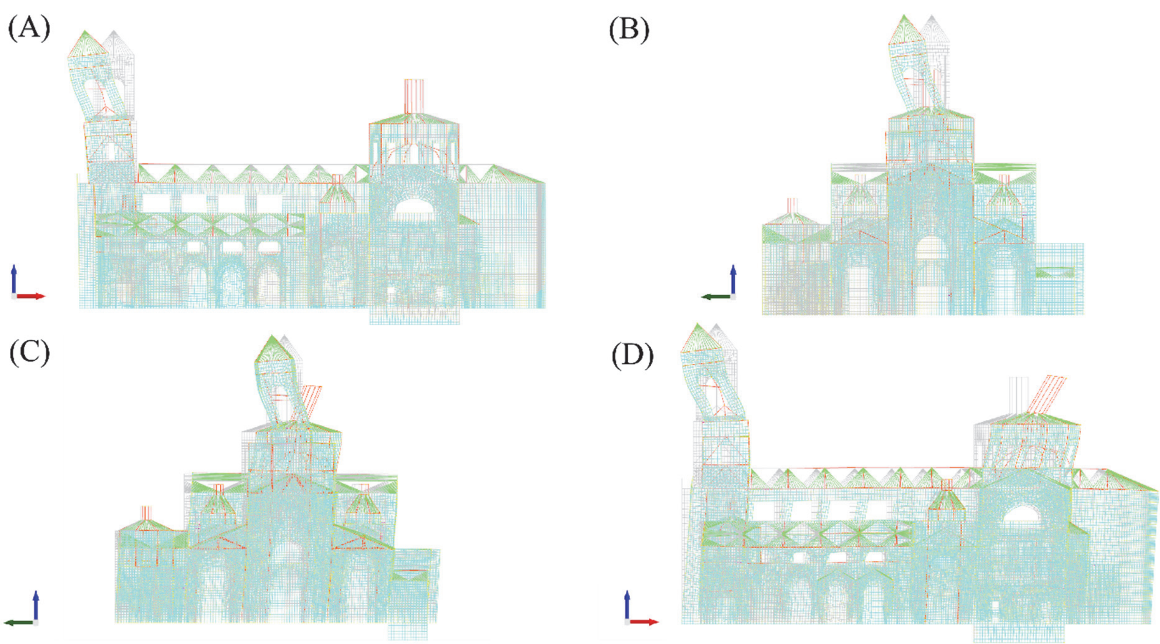

Figure 9: First (A), second (B), third (C) and fourth (D) mode shapes of the S.Maria Assunta Cathedral (axes X, Y and Z are indicated by colors red, green and blue, respectively).

Mode number	Period (s)	Mass participation ratio X	Y	Z	Mode number	Period (s)	Mass participation ratio X	Y	Z
1	0.4333	0.0763	0.0001	0.0000	6	0.2260	0.0000	0.0013	0.0000
2	0.4257	0.0001	0.0622	0.0000	7	0.2077	0.0314	0.0001	0.0001
3	0.2720	0.0003	0.5833	0.0000	8	0.2015	0.0000	0.0234	0.0000
4	0.2555	0.5013	0.0007	0.0001	9	0.1879	0.0011	0.0017	0.0001
5	0.2389	0.0000	0.0040	0.0000	10	0.1801	0.0000	0.0151	0.0000

Table 3: Modal information of the Cathedral's 3D global model for the first ten natural modes.

Nonlinear pushover analysis

By using the two models introduced in the previous section for the representation of material nonlinearities in masonry structures, i.e., the proposed refined diffuse interface model and the (reference) isotropic damage model, different nonlinear static analyses have been performed on the extracted transversal macro-element. Both the geometric configuration and the reference mesh arrangement of the analyzed macro-element considered for the subsequent structural analyses are reported in Fig. 10. It is worth noting that possible structural interactions of the modeled structural portion with the adjacent and incident ones have been neglected in favor of safety, neglecting the relevant contributions in terms of stiffness and mass but considering the effects in terms of transmitted loads. As a matter of fact, this assumption can be justified by the weak connection between the distinct portions of the Cathedral, which were built in different ages and without introducing construction details able to guarantee a global behavior against horizontal forces. Furthermore, also the soil/structure interactions have been neglected, so that fixed constraints have been considered as prescribed boundary conditions at the ground level.

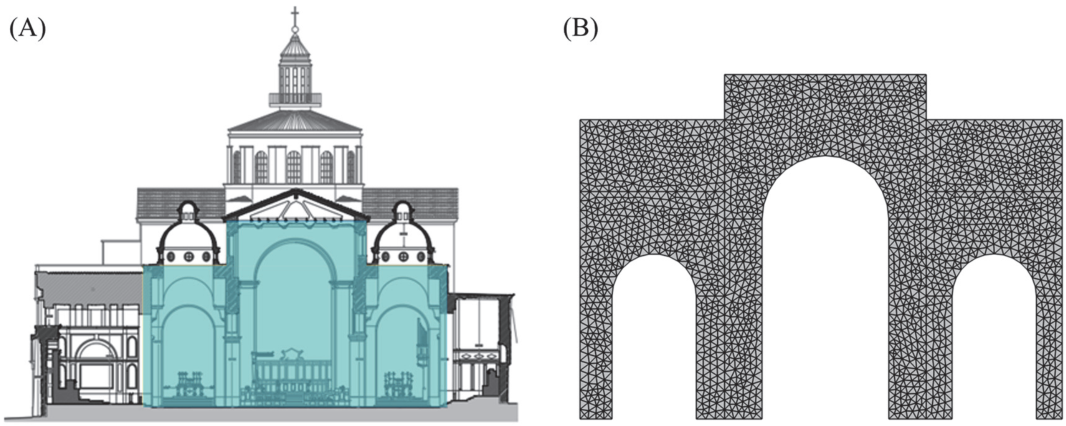

Figure 10: Transverse macro-element: identification in the transverse vertical section (A) and representation of the reference unstructured mesh (i.e. Delaunay) for the finite element model (B).

The nonlinear pushover analysis has been performed according to the following procedure:

- In the first step, a nonlinear analysis is performed, in which only the vertical loads (gravitational loads, vertical diffuse actions) are considered. Among these loads, the actions derived from the central dome and the two lateral gable roofs of the transept have been computed, including both the weight of all structural elements not explicitly modeled and the relevant applied loads;
- In the second step, starting from the configuration at the end of the first step, a nonlinear analysis is performed by applying monotonically increasing horizontal forces, obtained by assuming a uniform distribution of accelerations along the macro-element height, as prescribed by the Italian standards [8]. In particular, a lateral displacement control scheme has been adopted, with reference to the upper right corner of the central part of the macro-element (see Fig. 10B). Moreover, a Newton-Raphson method is employed to solve the nonlinear equilibrium equations. This step is aimed at the evaluation of the capacity curve of the given structure.

The reliability of the DIM approach for the failure analysis of masonry structures has been assessed by performing a comparison with the well-established isotropic damage model. As shown in Fig. 11, the results are in good agreement with each other in terms of damage maps. It is important to note that the cohesive/volumetric approach implemented in the DIM is more accurate in predicting the crack pattern during the damage evolution (potentially, also in terms of crack width and spacing). Such an advantage is provided by the superior ability of the proposed approach over the reference damage model to keep the discrete nature of fracture.

Moreover, a mesh sensitivity analysis has been conducted, both for damage and cohesive models, in order to investigate their mesh dependency properties.

In particular, three different meshes have been used for each model: mesh 1 (the coarsest, used as the reference mesh and shown in Fig. 11B), mesh 2 and mesh 3 (the finest), with an average element size of 0.6, 0.45 and 0.3 m, respectively. The results show that the proposed cohesive methodology is reliable in predicting the strongly nonlinear response of the given masonry macro-element. Firstly, the mesh convergence analysis presented here has demonstrated the small sensitivity of the peak and post-peak behavior to the mesh size, provided that a sufficiently refined discretization is used, in the spirit of a classical finite element setting. Secondly, the structural response predicted by the DIM (see Fig. 12(A)) are consistent with those arising from the well-established isotropic damage model (reported in Fig. 12(B)), thus further confirming the predictive capabilities of the novel approach.

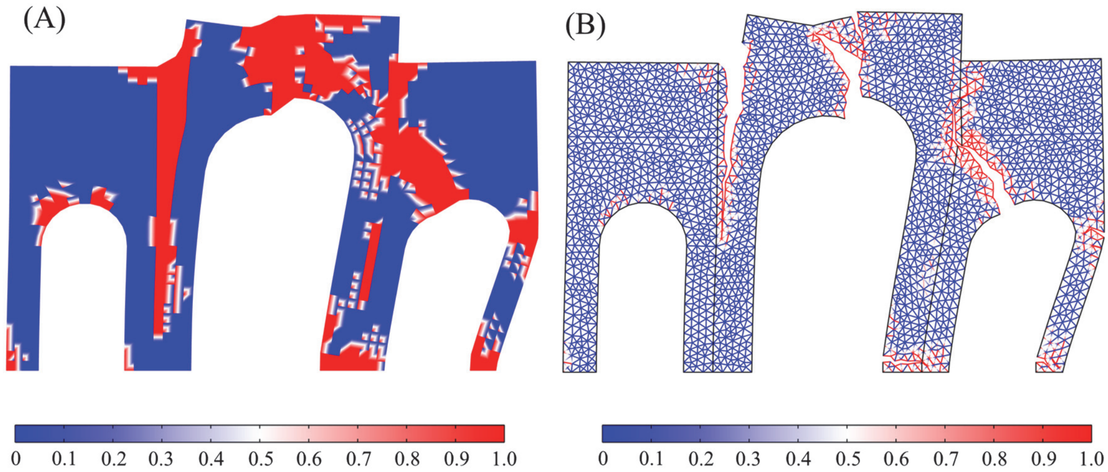

Figure 11: Damage maps: comparison between isotropic damage model (A) and DIM approach (B).

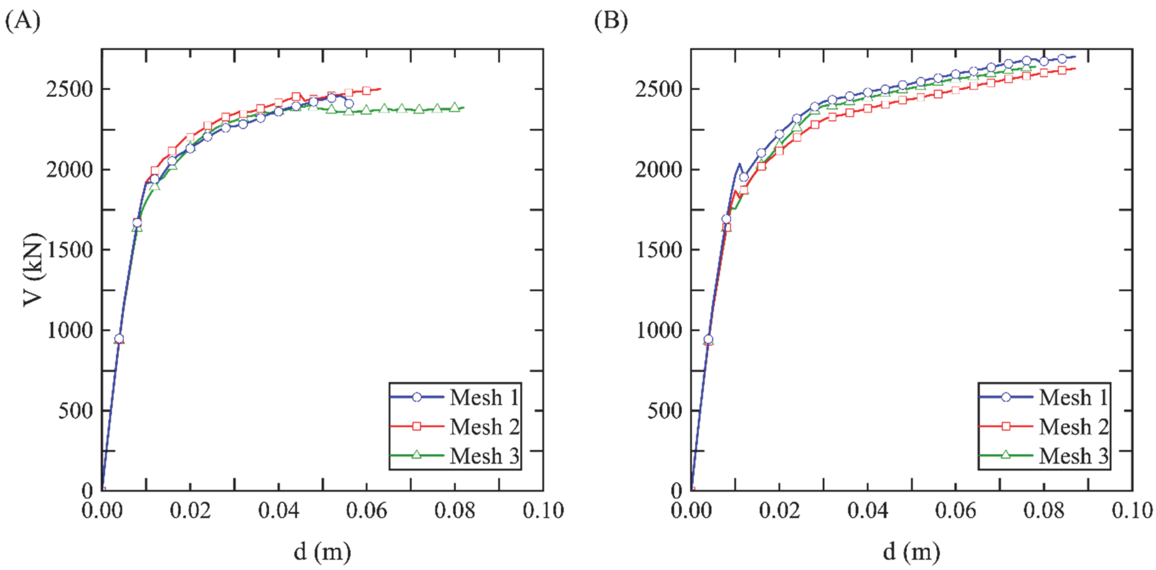

Figure 12: Pushover curves: comparison between DIM approach (A) and reference isotropic damage model (B).

Limit analysis of the local pre-assigned failure mechanisms in the masonry structures

The first step of the study of local collapse mechanisms for the masonry structure under investigation requires the identification of all vulnerable masonry portions susceptible to overturning risk. To support the identification of the vulnerable local portions, Annex C of the Italian guidelines [9] reports a set of 28 local collapse mechanisms that usually occur in masonry church structures. Such collapse mechanisms have been identified based on the inspections conducted in several church masonry structures damaged by seismic events.

Fig. 13 shows a schematic of the local collapse mechanisms identified for the masonry structure investigated in the present study. Although the present analysis investigated all such collapse mechanisms for assessing the seismic vulnerability of the Cathedral, in the sake of brevity here this paper focused the attention only on that involving the bell tower, depicted in the exploded scheme in Fig. 14. The bell tower is part of the main façade of the Cathedral and represents the highest sub-structure of the building. In particular, starting from the height of 17.70 m, the structure is alone from other masonry sub-structures. Hence, such a portion represents a soaring element ready to overturn under seismic action. Note that Annex C of the Italian Code does not report this mechanism explicitly. However, it is like the one causing the overturning of a common spire sub-structure, hence worth to be investigated. Besides, the fall of the bell tower represents a seriously unsafe condition because of its critical position up to the church's main entrance.

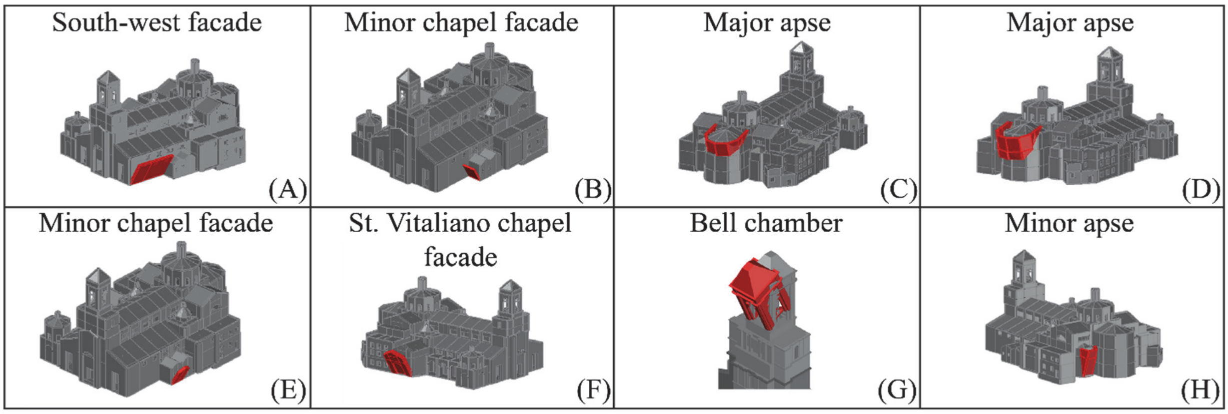

Figure 13: Schematic representation of pre-assigned failure mechanisms. A, B, E, F: façade overturning mechanism; C, D, H: apse overturning mechanism; G: belfry overturning mechanism.

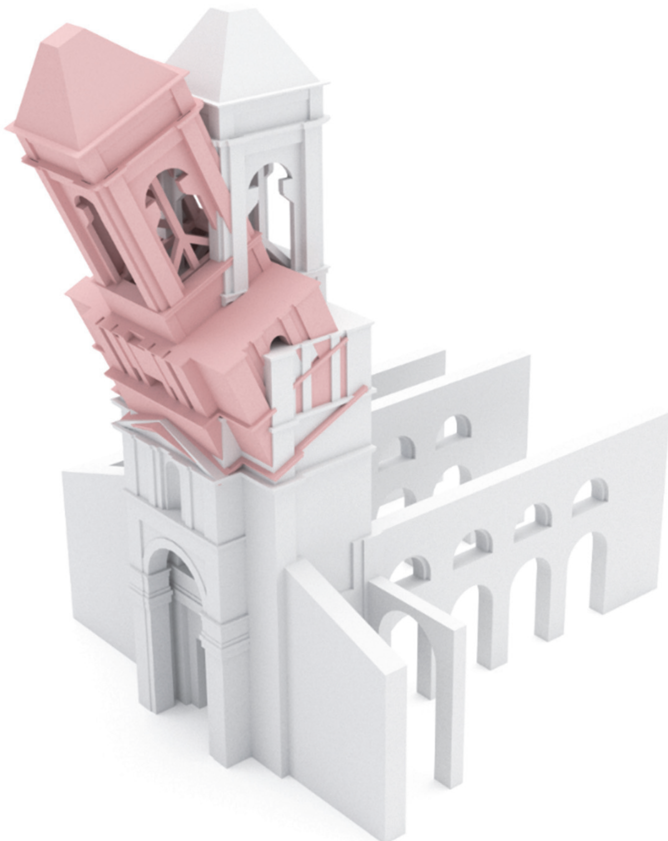

Figure 14: 3D representation of the bell tower pre-assigned failure mechanism.

COMPARATIVE ASSESSMENT AND STRUCTURAL SAFETY PREDICTION

As is well known, ancient buildings can usually bear only partially the entity of seismic accelerations provided by the present technical codes. Indeed, various retrofitting systems can be designed with the aim of improving the seismic response of such structures until that all the design seismic actions can be carried by the structural members. To better understand this aspect, the Italian standards introduce the notion of seismic vulnerability indicator, computed as the ratio between the actual capacity of the structure to carry horizontal loads, expressed in terms of seismic acceleration a_g, and the reference seismic acceleration of the construction site, $a_{g,\text{ref}}$, expressed by means of the design response spectrum provided by the Italian standards for construction. Moreover, the seismic vulnerability indicator can be also expressed in terms of return period, according to the expression reported in Ref. [8]. The expressions of these two indicators, denoted as ξ_E and ξ_{TR}, respectively, read as:

$$\xi_E = \frac{a_g}{a_{g,\text{ref}}} \qquad \xi_{TR} = \left(\frac{T_R}{T_{R,\text{ref}}}\right)^{\alpha} \tag{8}$$

In the second expression of Eq. (8) the exponent α is derived from the statistical analysis of the seismic hazard curves of the Italian territory. It is set equal to 0.41, in order to obtain results that are comparable with the value of the analogous seismic indicator expressed in terms of acceleration.

Fig. 15 show the average values of the return period seismic indicators for different groups of structural elements and loading types, as obtained by a linear dynamic analysis performed on the previously described global model. It is possible to note that the shear mechanisms, for both masonry and reinforced concrete elements, are the most critical while for the compression-bending actions, the various parts of the structure exhibit higher values of the seismic vulnerability indicators.

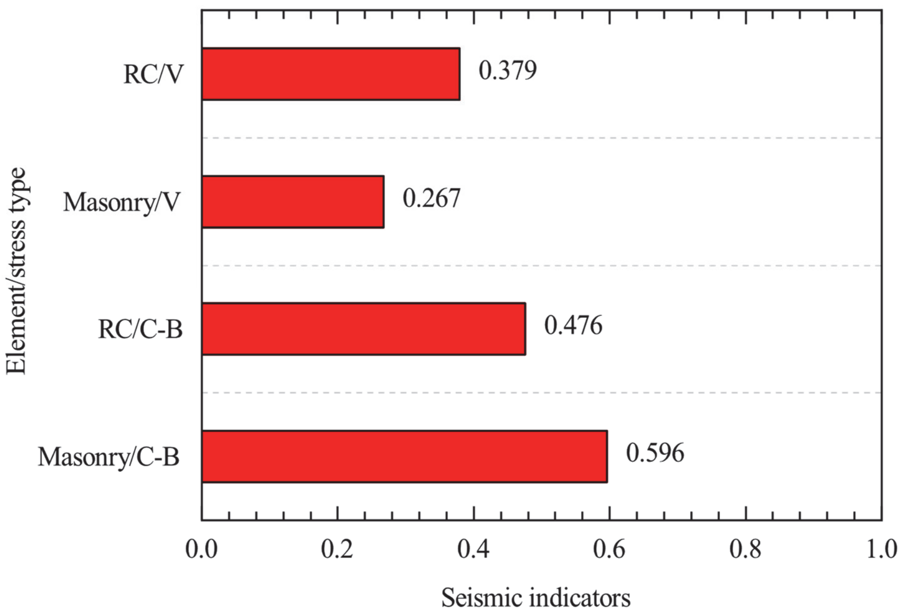

Figure 15: Average values of the seismic indicator in terms of return period for different element/stress type: RC/V=Reinforced concrete/shear force; Masonry/V=Masonry/shear force; RC/C-B=Reinforced concrete/compression-bending; Masonry/C-B=Masonry/compression-bending.

With regard to pre-assigned failure mechanism involving the bell tower, and depicted in Fig. 14, the seismic indicators in terms of both return period and seismic acceleration have been evaluated and compared to each other as illustrated in Fig. 16.

From the results reported in Fig. 16, it is important to note that the two seismic indicators defined in Eq. (8) are almost in agreement with each other.

Starting from the values of the seismic vulnerability indicators, Fig. 17 illustrates the number of structural members requiring seismic retrofitting as a function of the desired value of seismic indicator in terms of return period (T_R). For example, to achieve a value of $\xi_{TR} = 0.3$, corresponding to a seismic upgrade level of 30%, it is necessary to improve the seismic capacity of 184 masonry piers, 69 masonry spandrels, 44 reinforced concrete beams and 6 reinforced concrete columns. This scenario indicates that the reinforced concrete columns are in good conditions, after considering that only the 9% of the total number of RC columns require maintenance. On the contrary, the masonry spandrels are the most vulnerability elements of the whole structures, after considering that the 74% of total members requires maintenance.

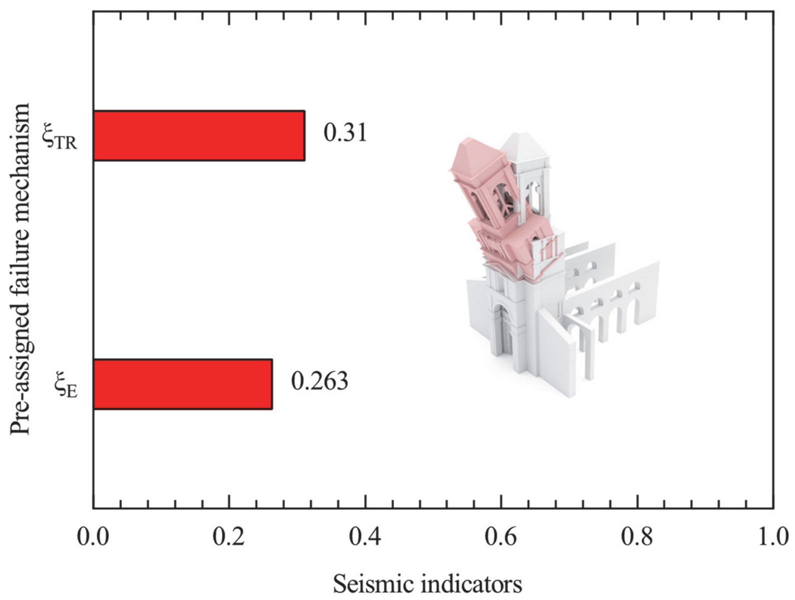

Figure 16: Comparison between the seismic indicators for the pre-assigned failure mechanism of the bell tower.

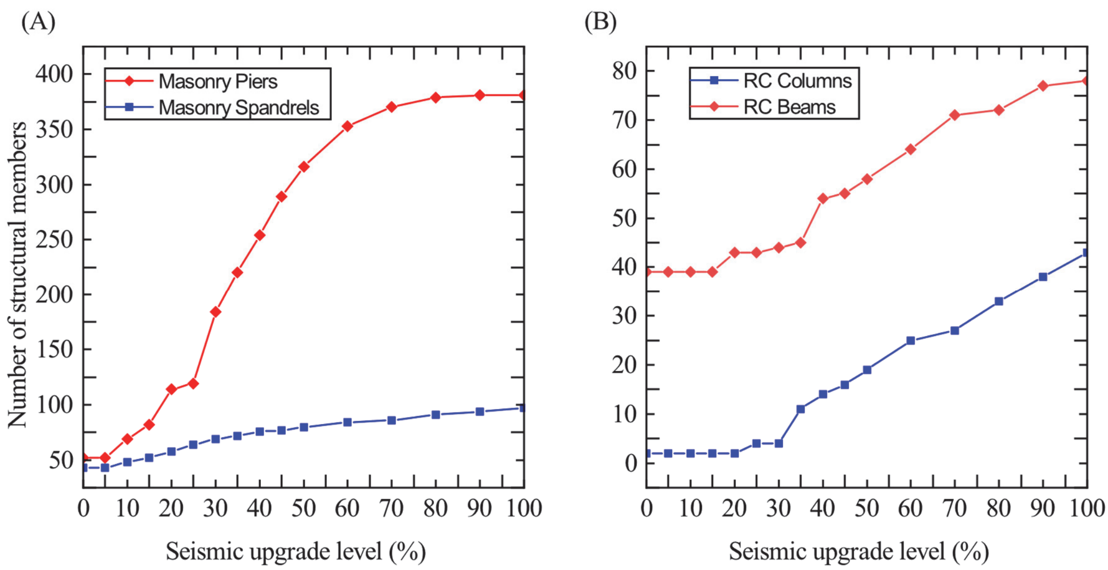

Figure 17: Number of structural elements requiring retrofitting interventions as a function of the required seismic upgrade level, for masonry elements (A) and reinforced concrete elements (B).

In order to obtain a more precise information about the seismic vulnerability of the Cathedral, the numerical results arising from the nonlinear static analysis performed on the structural macro-element are considered. In particular, the curve that provides the most conservative results in terms of collapse displacement has been chosen among the three curves depicted in Fig. 12(A), with the aim of deriving the related seismic vulnerability indicator. After the evaluation of the capacity curve of the structure, that represents the structural response of a multi-degree-of-freedom (MDOF) system [56], the curve of the single-degree-of-freedom (SDOF) system has been evaluated by dividing the horizontal displacement and base shear, i.e. the coordinates of each evaluation point, by the modal participation factor of the given macro-element evaluated during a preliminary linear dynamic analysis. Moreover, the equivalent bilinear curve needed for the evaluation of the seismic indicator has been also evaluated, as show in Fig. 18.

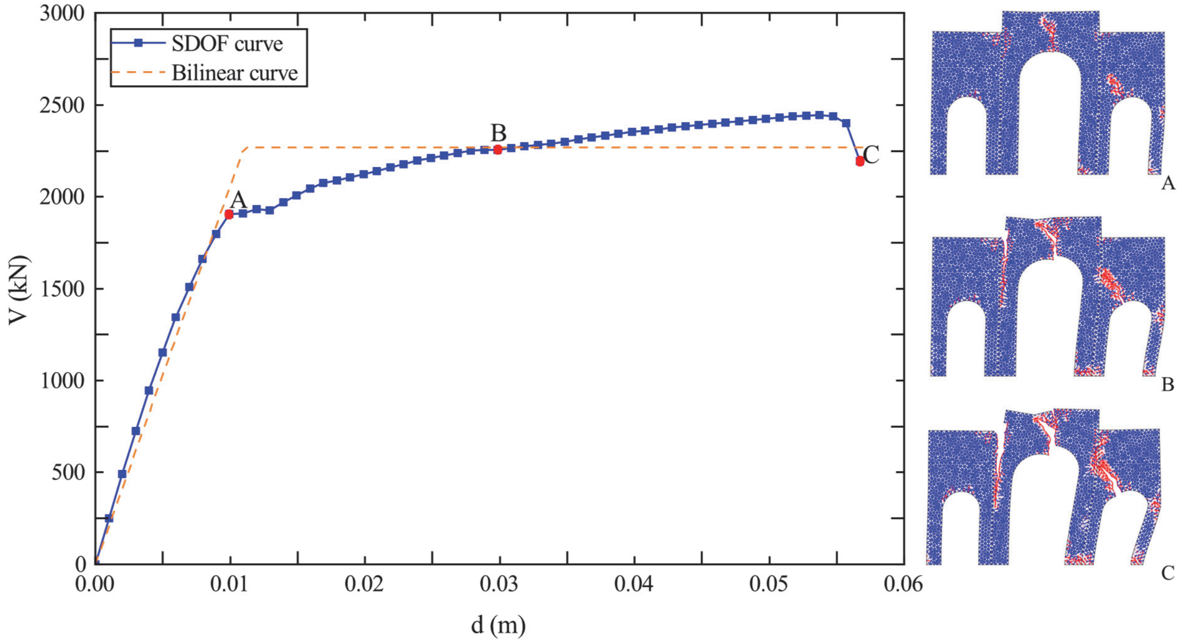

Figure 18: Capacity curve of the transverse macro-element and damage maps for three different points: (A) peak of the linear-elastic phase; (B) damage diffused in all the elements involved in the collapse mechanism; (C) collapse of the structure.

The seismic indicator associated with the capacity curve obtained through the above procedure has been evaluated for the life-safety limit state (SLV in Italian standard) [8].
In Tabs. 4 and 5, the complete results of this comparison are reported, showing that the seismic vulnerability indicators obtained by the linear dynamic analysis are lower than the analogous ones computed via the nonlinear static analysis. Such a difference can be explained by the fact that the structural safety assessment according to the linear dynamic analysis as prescribed by Italian standards is based on the evaluation of local failure mechanisms at the discrete element level rather than at the macro-element one, thus leading to more conservative results.

a_g / g	$a_{g,ref} / g$	T_R (Years)	$T_{R,ref}$ (Years)	ξ_E	ξ_{TR}
0.145	0.288	145	712	0.503	0.521

Table 4: Seismic vulnerability indicators for the macro-element obtained by performing the nonlinear static analysis on the local model.

a_g / g	$a_{g,ref} / g$	T_R (Years)	$T_{R,ref}$ (Years)	ξ_E	ξ_{TR}
0.128	0.288	82	712	0.445	0.412

Table 5: Average seismic vulnerability indicators for the macro-element obtained by performing the linear dynamic analysis on the global model.

Conclusions

In the present work, a detailed comparison between different analyses methodologies has been presented, in order to evaluate the seismic vulnerability of the Cathedral of S. Maria Assunta in Catanzaro (Italy). The classical limit and dynamic analyses admitted by the Italian code have been compared with the nonlinear static analysis performed using the diffuse interface model (DIM), an innovative research method based on the inter-element fracture approach for

simulating the material nonlinearity associated to damage. Starting from the definition of the knowledge level, different models have been developed and implemented within a finite element framework in order to perform advanced structural analyses.
The results illustrated in the previous two sections are suitable for a preliminary identification of the main structural deficiencies of the construction and to establish the necessity to increase the knowledge level. Moreover, the obtained outcomes allow to define a hierarchy between the different structural elements in terms of intervention priorities.
The analysis of local failure mechanisms has been performed by using a linear kinematic approach, while a linear dynamic analysis has been used to study the global behavior of the Cathedral under seismic loads. Moreover, the macro-elements extracted from the global model have been investigated via a pushover analysis using the force distributions provided from the Italian code. Two different models have been used to represent material nonlinearities, the former is based on a Rankine failure criterion, whereas the latter on a cohesive approach.
The study of the pre-assigned local failure mechanisms has pointed out the vulnerability of single parts of the building, especially the bell tower. Results obtained by using the cohesive model in conjunction with the DIM technique on macro-elements, show a reasonable agreement with those determined by means of classical damage models.
Despite the possible uncertainties, the proposed framework based on different types of structural analysis, including both classical and innovative approaches, can be regarded as a suitable tool to estimate the seismic vulnerability of historical masonry churches. More accurate investigations could be provided in the future taking into account the damage processes induced by nonlinear phenomena acting at the microstructural scale of the material (see for instance [57,58]) or by using the DIM model in combination with multiscale method [59,60], aimed at explicitly modeling the mechanical behavior of each masonry constituent.

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