Damage survey, simplified assessment and advanced seismic analyses of two masonry churches after the 2012 Emilia earthquake

by
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Abstract

Recent Italian earthquakes (L'Aquila 2009, Emilia 2012, Central Italy 2016-2017) showed the high vulnerability of existing and historical masonry constructions to seismic actions. This paper investigates the damage distribution and the seismic response of two masonry churches located in the province of Ferrara (Northern Italy) and severely damaged by the 2012 Emilia earthquake. A proper knowledge of the geometry and structural details of the two churches was achieved through accurate on-site surveys and documentary research conducted after the earthquake. Moreover, the surveys provided detailed information about the crack patterns, highlighting the presence of significant damage in several parts of the churches. The seismic vulnerability assessment of the two churches was conducted on the basis of both the simplified approach given by Italian Guidelines on the Built Heritage and sophisticated non-linear dynamic analyses performed on detailed finite element models assuming an elasto-plastic damage behavior for masonry. Limit analysis with pre-assigned failure mechanisms was carried out to preliminarily identify the most probable active failure mechanisms. Non-linear dynamic analyses with different PGA levels allowed for a deep insight into the damage distribution in the churches: moreover, the energy density dissipated by tensile damage and the maximum normalized displacements were computed for the different macro-elements. A comparison between the numerical results and the damage observed during detailed field surveys was also provided. The results obtained in this study may represent a useful step to improve the knowledge of the seismic behavior of similar masonry churches located in the same region.

Keywords: masonry church, crack pattern, simplified approach, FE model, non-linear dynamic analysis, damage distribution.

1. Introduction

In May-June 2012 a seismic sequence struck a wide portion of the Emilia-Romagna Region (Northern Italy), mainly the provinces of Modena and Ferrara. The first mainshock (ML 5.9) occurred on May 20 and the epicenter was located a few kilometers north of the municipality of Finale Emilia. A second main shock (ML 5.8) occurred on May 29, about 12 km west of the first earthquake, with the epicenter near the municipality of Medolla. Industrial precast buildings and historical heritage, including castles, fortresses, palaces, churches, bell towers and towers, particularly common in the area, were heavily damaged by the seismic sequence (Artioli, Battaglia and Tralli 2013; D'Altri et al. 2017; Cattari et al. 2014; Castellazzi et al. 2017; Castellazzi et al. 2018; Andreini et al. 2014; Penna et al. 2014): masonry churches, in particular, were subjected to extensive damage (Sorrentino et al. 2013; Barbieri et al. 2017; Valente et al 2017a; Valente et al. 2017b). In fact, masonry churches are not conceived to properly withstand horizontal loads and may be susceptible to damage and prone to partial or total collapse even under small-to-moderate seismic actions (Doglioni, Moretti and Petrini 1994; Brandonisio et al. 2013; Foraboschi 2013; Lagomarsino 2012; Milani, Sheu and Valente 2017a). Their high seismic vulnerability is mainly due to the particular structural and architectonical features, such as high and slender perimeter walls, absence of adequate connections between the various parts, presence of flexible wooden roofs, and the specific mechanical properties of the masonry material, such as a very small tensile strength.

The seismic vulnerability assessment of masonry churches is an articulated problem due to the complexity of the structures and requires an integrated approach with a deep knowledge of the history, geometry, materials and structure. The observation of damage occurred in past earthquakes is of paramount importance to understand the seismic performance of churches and highlight their main structural deficiencies: post-earthquake surveys have demonstrated that damage mechanisms have certain recurring characteristics and collapse usually occurs locally and in function of the macro-element typology. Several numerical studies have been carried out to assess the seismic vulnerability of churches and to understand their structural behavior under seismic actions. Different methods and procedures, characterized by different levels of accuracy and applicability, have been employed, including elastic eigen-frequency analysis, elastic response spectrum analysis, pushover analysis, limit analysis with preassigned failure mechanisms or with finite elements (FE) and sophisticated non-linear dynamic analysis (Aras et al. 2011; De Matteis and Mazzolani 2010; Castellazzi, Gentilini and Nobile 2013; Brando, Criber and De Matteis 2015). Moreover, large-scale studies have been also conducted by some researchers (Marotta et al. 2017; De Matteis, Criber and Brando 2016; Da Porto et al 2012) to provide a territorial scale assessment of the seismic vulnerability of churches, which can be used to identify the most critical macro-elements and to predict the damage level that could be expected on similar churches.

This paper presents the results of an extensive investigation program, including visual inspection and on-site survey, documentary research, structural modelling, application of simplified approach and advanced numerical simulations, to assess the damage state and the seismic response of two masonry churches hit by the 2012 Emilia earthquake.

In the first phase, several accurate on-site surveys of the churches were performed with the aim of obtaining the main details on the geometry of the structure and identifying critical areas. The post-earthquake damage assessment provided useful information on the seismic response of the structures: as a result of the field survey, the damage observed for the churches was also interpreted in terms of collapse mechanisms.

In the second phase, a simplified approach directly based on the limit analysis theorems and full three-dimensional non-linear dynamic analyses were used to identify the most vulnerable elements of the churches under study.

First, a macro-element approach based on the limit analysis theorems was used complementarily to the non-linear dynamic analyses. The Italian Guidelines on the Built Heritage (DPCM 2011) suggest

a quantitative assessment of the seismic vulnerability of churches based on the upper bound theorem of limit analysis on 28 pre-assigned failure mechanisms, assuming masonry unable to withstand tensile stresses. In the framework of the kinematic theorem, the failure mechanism activating in reality is that associated to the lowest multiplier.
For the evaluation of the seismic vulnerability of historical masonry constructions, the use of appropriate and advanced numerical tools is essential (Barbieri et al. 2013; Betti and Vignoli 2008; Clementi et al. 2017; Formisano et al. 2015; Clementi et al. 2016; Milani, Shehu and Valente 2017b; Valente and Milani 2017; Milani, Valente and Alessandri 2017; Milani, Shehu and Valente 2018).
This study shows the importance of reliable finite element (FE) models in order to identify the potential collapse mechanisms of the churches and the structural weaknesses significantly affecting their seismic vulnerability. Compared to the previous approach, the FE method, based on a global analysis of the whole building under seismic loading through suitable numerical models, requires a large amount of input data and a great computational effort; on the other hand, it provides an accurate and comprehensive description of the seismic response of the churches. The seismic performance of the two churches was analyzed in terms of damage distribution, maximum normalized displacements and energy density dissipated by tensile damage for the main macro-elements under different PGA levels. A correlation between the damage detected by in-situ inspections and the numerical results obtained by using advanced numerical simulations was also provided.

2. Description of the churches under study

In this section a concise description of the main historical, geometrical and constructive features of the two churches under study is provided. The data collected from existing available documentations and from the survey phase were employed for the seismic vulnerability assessment of the two churches through simplified and refined analysis approaches.

2.1. San Paolo church in Porporana

San Paolo church is located in Porporana, a small hamlet in the north-west of the municipality of Ferrara. The pre-existing church dates back to the fourteenth century. The limited information about the church is collected in historical documents and maps that are present in the archives of the Municipality of Ferrara. In 1590 the church was rebuilt after the earthquake that hit the region in 1570. Various structural changes and restoration works were carried out over the centuries: in particular, in the eighteenth century the bell tower, the large presbytery zone and the side chapels were added. The first documents providing the plan and elevation drawings of the structure date back to the end of the nineteenth century.
The masonry church, which is about 29 m long and 14 m wide, consists of a single nave with lateral chapels: a small winter chapel and sacristy are adjacent to the apse area. The triumphal arch divides the nave from the presbytery. The structure is made of exposed bricks, except for the main façade that is plastered.
The main façade (about 17 m high) has a gable on the top and is characterized by recesses, lesenes and an infilled large window. Five pinnacles are present on the top of the façade. The nave is covered by a wooden double-pitched roof, whereas the side chapels, the winter chapel and the sacristy are covered by a single-pitched roof. Inside, the main hall presents ribs and vaults *in incannucciato*; the side chapels are covered by barrel vaults made of *mattoni in foglio*. The winter chapel presents a cloister vault *in incannucciato*, the sacristy and the apse have a sand ceiling with beams. It is worth mentioning that the side chapels, the winter chapel and the sacristy were added later and therefore the interlocking may result not perfectly effective.
Figure 1 shows the drawings of the plan, façade, longitudinal and transversal sections of the church with indication of the main dimensions; some general views of the church are also provided.

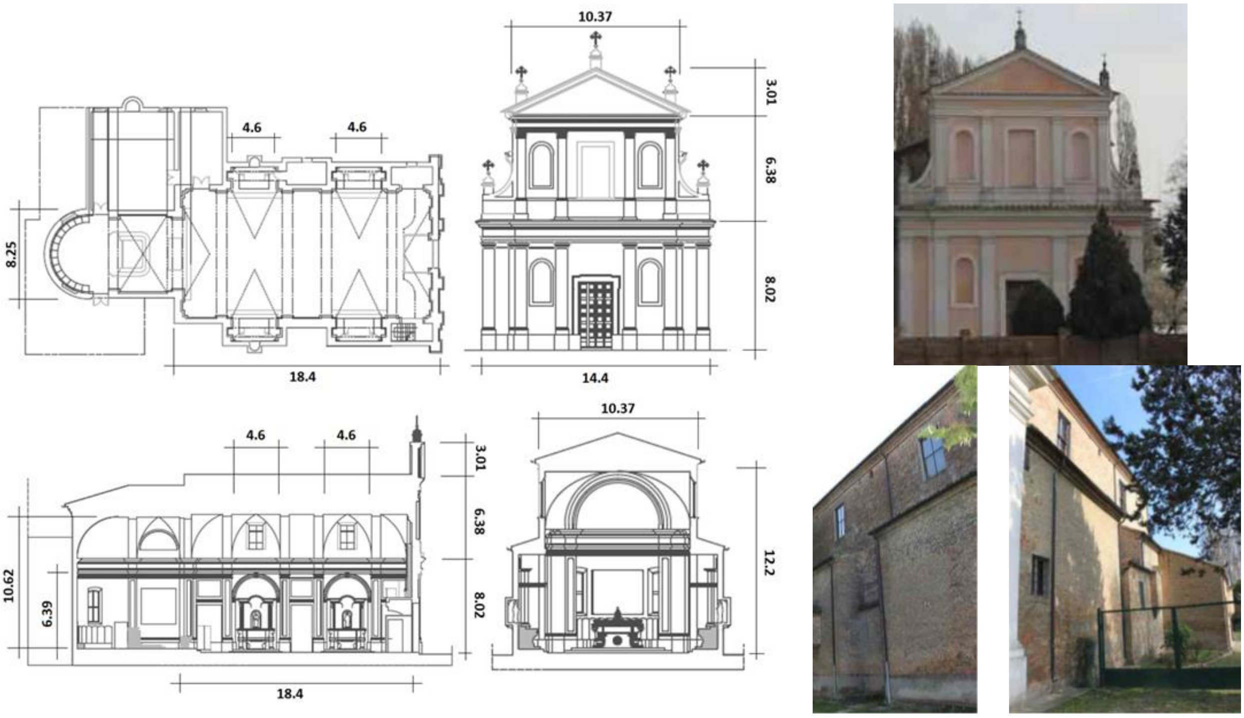


Figure 1. San Paolo church in Porporana. Drawings of the plan, façade, longitudinal and transversal sections along with some general views. The main geometrical dimensions of the church are also reported (in meters).

2.2. San Pietro Apostolo church in Bondeno

San Pietro Apostolo church is located in the small hamlet of Santa Bianca in the municipality of Bondeno, in the province of Ferrara. The first documents dealing with the existence of the church date back to the end of the fourteenth century. In 1680 the church was severely damaged by a devastating flood and it was demolished and rebuilt, with the exception of the bell tower, which remained the original one. In the first half of the twentieth century some interventions enlarged the original structure. In the last decades some small structural changes and restoration works were carried out.

The masonry church is about 28 m long and 9.5 m wide and it is oriented northwest. The church has a tripartite façade, which is about 13 m high and is decorated by lesenes. The façade, surmounted by a triangular pediment with decorative pinnacles, has a single entrance and two symmetrical recesses in the upper part. The plan of the church consists of a single nave with side chapels. The access to the side chapels is possible through portals of equal size surmounted by round arches. The central nave and the side chapels are covered by cross and round arched vaults *in incannucciato*. The presbytery zone, elevated by some steps and preceded by a triumphal arch, is 3.8 m long and covered by a false dome. The church ends with a circular apse presenting a diameter equal to about 4.5 m. The bell tower, on the left side of the church, is connected to the presbytery zone: it exhibits a square section and is about 15 m high. The church is characterized by the presence of adjacent buildings (rectory, sacristy and minor constructions).

Figure 2 shows the drawings of the plan and the elevations with indication of the main dimensions of the church; some general views of the church are also provided.

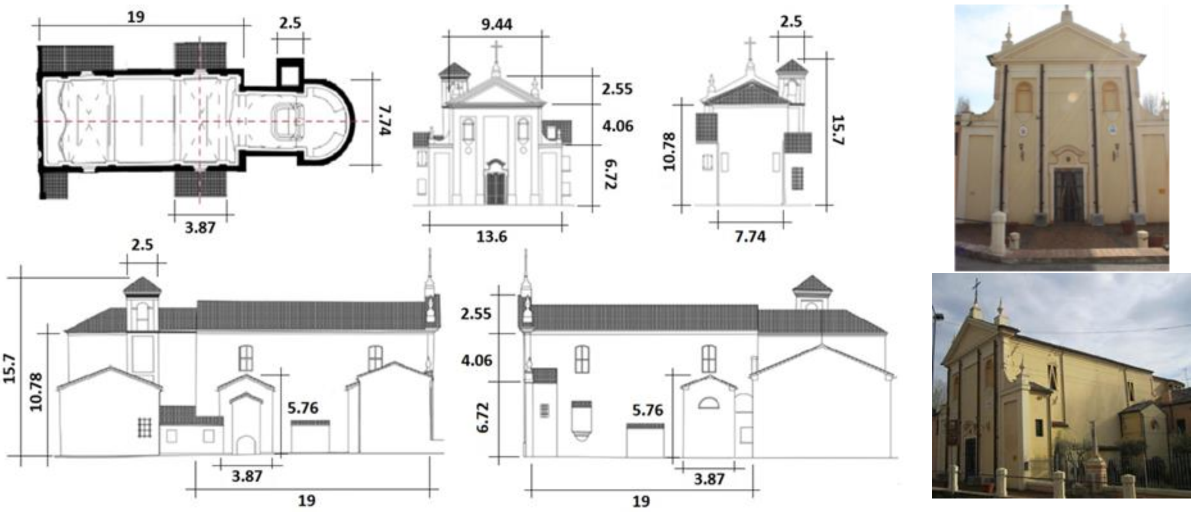


Figure 2. San Pietro Apostolo church in Bondeno. Drawings of the plan and elevations along with some general views. The main geometrical dimensions of the church are also reported (in meters).

3. Damage survey

The 2012 Emilia earthquake caused widespread damage to both the churches, in particular to San Pietro Apostolo church. Detailed field surveys of the cracks patterns were conducted after the seismic sequence and, based on different visual inspections, the presence of several cracks was detected. In the following, a short description of a photographic collection of the main cracks is provided for both the churches.

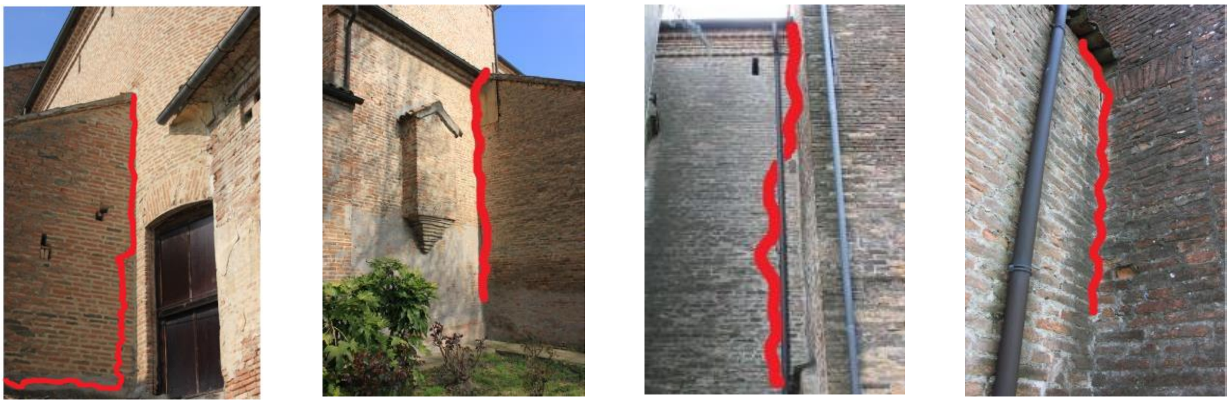
3.1. San Paolo church

The façade presents severe vertical cracks that propagate from the entablature to the main door, Figure 3 (photo a); an important crack on the façade is also visible inside the church, in correspondence with the arch above the organ. Significant vertical cracks can be observed in the upper part of the nave walls in correspondence with the façade, Figure 3 (photos b-c). Internal and external cracks patterns observed suggest the activation of a partial failure mechanism involving the façade. As a matter of fact, it should be also pointed out that the presence of a meaningful vertical crack near the main opening of the façade is probably an indication of a relatively good interlocking between façade and nave lateral walls, which is also expected to be associated to a higher load carrying capacity. Since the most vulnerable part of the structure seems to be the façade under out-of-plane actions, it appears reasonable to conclude that the main direction of the shake might be roughly parallel to the longitudinal direction of the structure. To corroborate such a conclusion, façade cracks pattern should be always read cross-checking information with surveys conducted on the rest of the church, especially on longitudinal walls of the nave and near the triumphal arch.

Figure 4 collects the results of such a cross-checking and synoptically shows severe vertical cracks in the connection regions between the side chapels and the nave walls (photos a-b) and between the church and the annexes (photos c-d). Such cracks suggest a differential deformation of the structure along the longitudinal direction, particularly among the central nave, the lateral annexes and the triumphal arch, which is obviously a consequence of an earthquake shake directed longitudinally. Inside the church, many cracks are visible in the upper part of the building, mainly near the arches and openings, Figure 5 (photos a-b). The vertical cracks along the connection regions between the nave walls and the adjacent buildings (side chapel and sacristy) are considerable also inside the church, Figure 5 (photos c-d), an information which once again confirms the idea of a shake acting along the main direction of the nave.

(a) (b) (c)

Figure 3. San Paolo church. (a) Vertical cracks in the façade; (b)-(c) vertical cracks in the connection region between the façade and the upper part of the nave walls.



(a) (b) (c) (d)

Figure 4. San Paolo church. (a)-(b) Vertical cracks between the side chapels and the nave walls; (c)-(d) vertical cracks between the church and the annexes.

(a) (b) (c) (d)

Figure 5. San Paolo church. (a)-(b) Cracks in correspondence with the side chapels; (c)-(d) vertical cracks between the nave walls and the side chapels and sacristy.

Again in agreement with the conclusions already raised, significant cracks are observed in the triumphal arch, where the effective contribution of the steel tie-road is also evident, Figure 6 (photo a). The apse presents clear vertical and diagonal cracks in the upper part of the right wall, Figure 6 (photo b), and in correspondence with a small door. Such a crack pattern indicates an overturning mechanism not yet fully developed; it is again consistent with what has already been discussed, i.e. that partial collapses experienced are associated to a shake directed mainly along the longitudinal direction, but also suggests that the load carrying capacity of the apse seems still slightly higher than that of the façade, despite the good interlocking of the façade with the nave lateral walls.

(a) (b)

Figure 6. San Paolo church. (a) Cracks in the triumphal arch; (b) crack in the apse

3.2. San Pietro Apostolo church

A comprehensive survey of the most important damages suffered by the church was carried out directly by the authors with repeated in-situ inspections, immediately after the 2012 seismic sequence (July and August 2012). As a matter of fact, the façade presents visible damage in the upper part and clear signs of detachment from the nave walls, Figure 7 (photos a-c). Such a damage may indicate the onset of an overturning mechanism of the façade, probably due to the not fully effective connection with the nave longitudinal walls. In addition, the presence of openings in the upper part may undermine the interlocking region. The clearly visible overturning of the upper part of the façade indicates that the seismic excitation was probably directed roughly along the longitudinal axis of the church. A collapse of some pinnacles can be observed in Figure 7 (photos a-b), which is again typical in such typology of structures; it occurs indeed far before the activation of a façade mechanism and is promoted both by the absent bracing of such architectural elements with contiguous structures and the amplification of the excitation at the roof level. The nave walls present visible shear cracks caused by the seismic excitation acting along the in-plane direction of the wall, see Figure 7 (photos c-d). This was not surprising, because it was already deduced by the façade damage state, considering that the seismic excitation was directed mainly along the longitudinal axis of the church. Cracks were found to be located mainly near the openings, in correspondence of the spandrels, with diagonal cross cracks typically ascribed to an in-plane shear failure under cyclic loads. The damage of the façade inside the church is illustrated in Figure 8 (photos a-b): significant cracks are visible in correspondence with the arches connecting the main nave and the side chapels and near the recesses located under the openings. Both crack patterns are compatible with a longitudinal deformation of the central nave, which tended to push the façade out-of-plane and detached from laterally located walls, e.g. chapels, as clearly visible from Figures 8 and 9.

(a) (b) (c) (d)

Figure 7. San Pietro Apostolo church. (a) Main cracks in the façade and collapse of the pinnacles; (b)-(c) cracks in the connection region between the façade and the nave walls; (d) cracks in the nave wall near the opening.

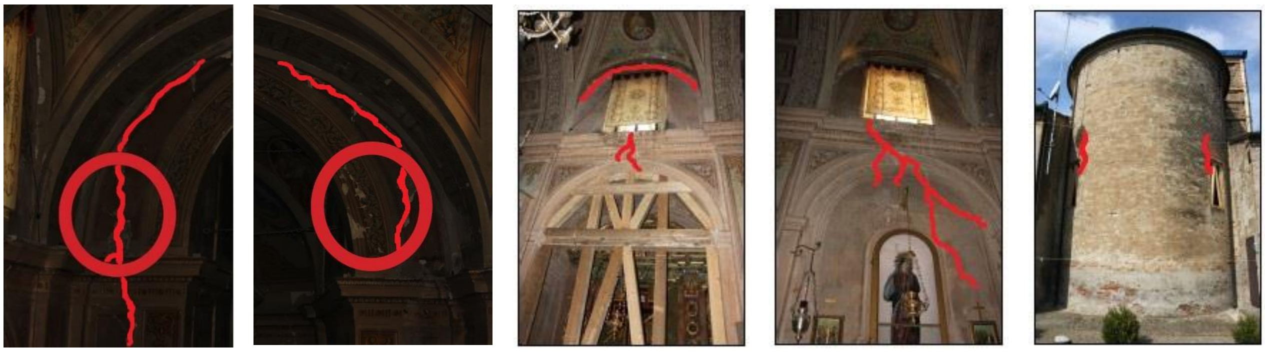
(a) (b)

Figure 8. San Pietro Apostolo church. (a)-(b) Cracks along the arch of the façade (inner view).

(a) (b) (c) (d)

Figure 9. San Pietro Apostolo church. (a)-(b) Cracks of the side chapels and nave walls (external view); (c)-(d) cracks of the side chapels (inner view).

Consistently with such a deduction, important cracks are indeed observed in the side chapels and near the connection region between the side chapels and the nave walls, promoted also by the quite ineffective interlocking between perpendicular walls observed. Figure 9 (photos a-d) shows the signs of detachment of the side chapels (outer and inner views), creating a possible overturning mechanism. Severe cracks are detected at the imposts of the triumphal arch, Figure 10 (photos a-b), due to the probably poor interlocking with the supporting nave walls and the longitudinal motion of the nave. Damage is more significant on the left side, probably influenced by the presence of the bell tower overloading the arch during the earthquake. Other cracks can be observed in the arch and in the recesses starting from the openings, as shown in Figure 10 (photos c-d). The apse does not present widespread damage or overturning mechanisms, meaning that the collapse activation acceleration was higher than that of the façade. Some typical diagonal cracks are observed near the openings, Figure 10 (photo e), which however can be justified by an in-plane shear stress concentration induced by the presence of the transept, which interconnects with the presbytery roughly in that location.



(a) (b) (c) (d) (e)

Figure 10. San Pietro Apostolo church. (a)-(b) Cracks at the imposts of the triumphal arch; (c)-(d) cracks starting from the openings and involving the arches and recesses; (e) cracks in the apse near the openings.

4. Limit analysis with pre-assigned mechanisms

A preliminary assessment based on limit analysis with pre-assigned failure mechanisms is conducted on the two churches according to Italian Guidelines on the Built Heritage (DPCM 2011). The partial failure mechanisms of the two churches are evaluated by means of a manual approach assisted with a common 3D CAD software. Such a simplified approach is based on the following assumptions: (1) rigid bodies forming a failure mechanism; (2) masonry unable to withstand tensile stresses; (3) the application of the classic upper bound theorem of limit analysis. It is worth mentioning that some failure mechanisms taken into account by the code are not considered in this study due to the simplicity of the church under consideration and the absence of several architectural elements. The selected macro-elements of the two churches under study are shown in Figure 11. The meaning of the labels is the following:

-F stands for façade overturning. Two collapse mechanisms are investigated, namely overturning with a horizontal yield line positioned on the base (F1) and at a middle height (F2). The conservative hypothesis of bad interlocking between façade and perpendicular walls is made, obviously resulting in very low horizontal accelerations associated with the activation of the failure mechanism.

-T stands for tympanum overturning. Two possible failures may occur; the first (T1) with horizontal hinge at the base of the tympanum, the second (T2) with inclined yield lines.

-SC stands for side chapel overturning.

-NW stands for nave wall overturning.

-TA refers to the triumphal arch behavior, with the in-plane failure for the formation of a four hinges mechanism.

-A refers to the apse overturning.

-BT stands for the belfry of the bell tower.

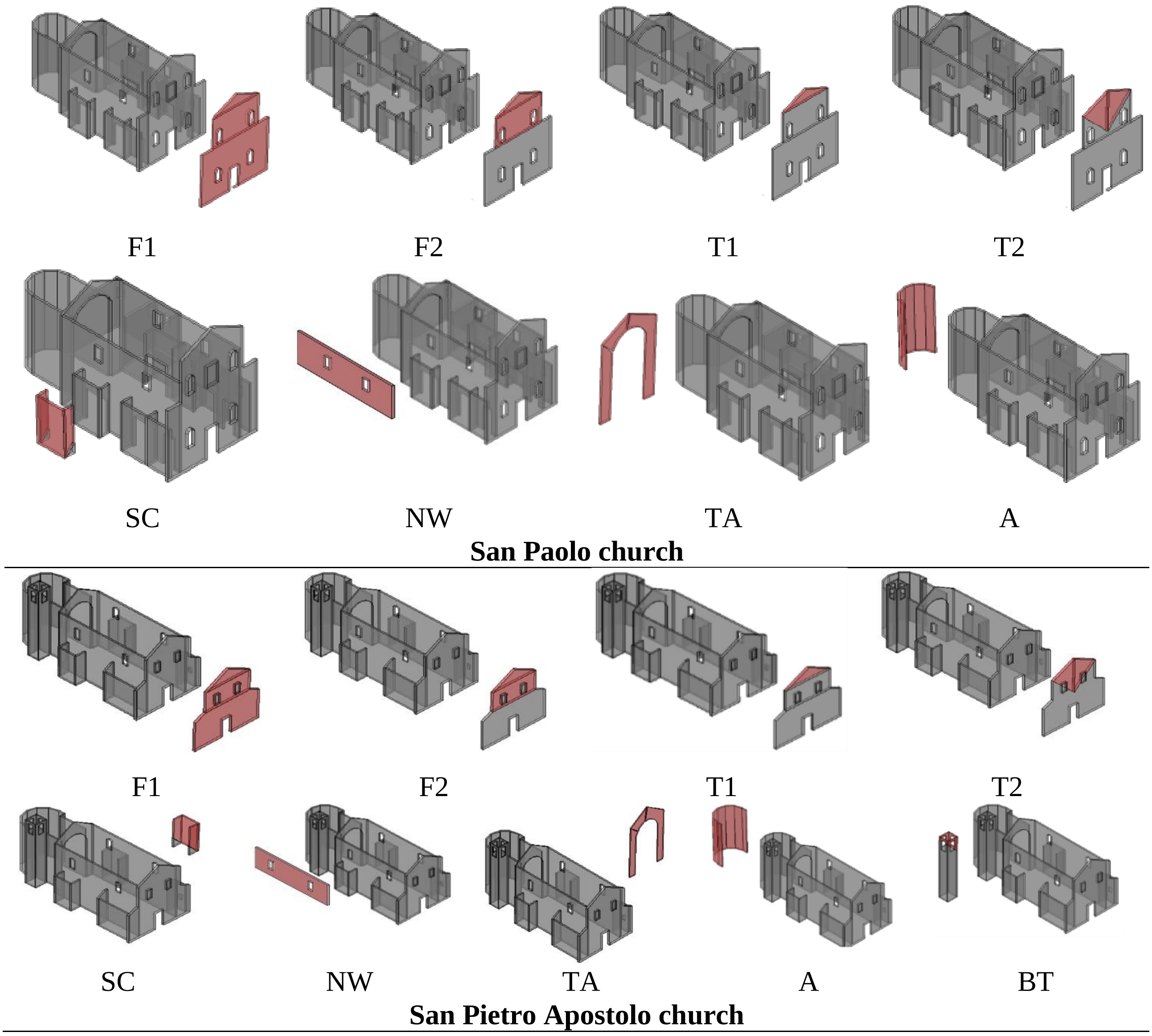


Figure 11. Selected macro-elements investigated through the application of the limit analysis with pre-assigned failure mechanisms for the two churches under study. Notation: F1=Overturning of the façade, F2=Overturning of the upper part of the façade, T1= Overturning of the tympanum (horizontal hinge), T2= Overturning of the tympanum (inclined yield lines), SC=Overturning of the side chapel, NW=Overturning of the nave wall, TA=Triumphal arch, A=Overturning of the apse, BT= Belfry of the bell tower.

The resultant collapse multiplier referring to partial failure mechanisms evaluated through the application of the upper bound theorem of limit analysis are summarized for both the churches in Figure 12. It can be observed that at-hand kinematic limit analysis based on failure mechanisms provides low values of accelerations at failure, especially in the case of façade (F) and nave wall (NW) overturning. This result is a consequence of the assumptions made for the masonry constitutive behavior (no-tension material) as well as of the conservative hypothesis of bad interlocking between perpendicular walls (especially for the façade).

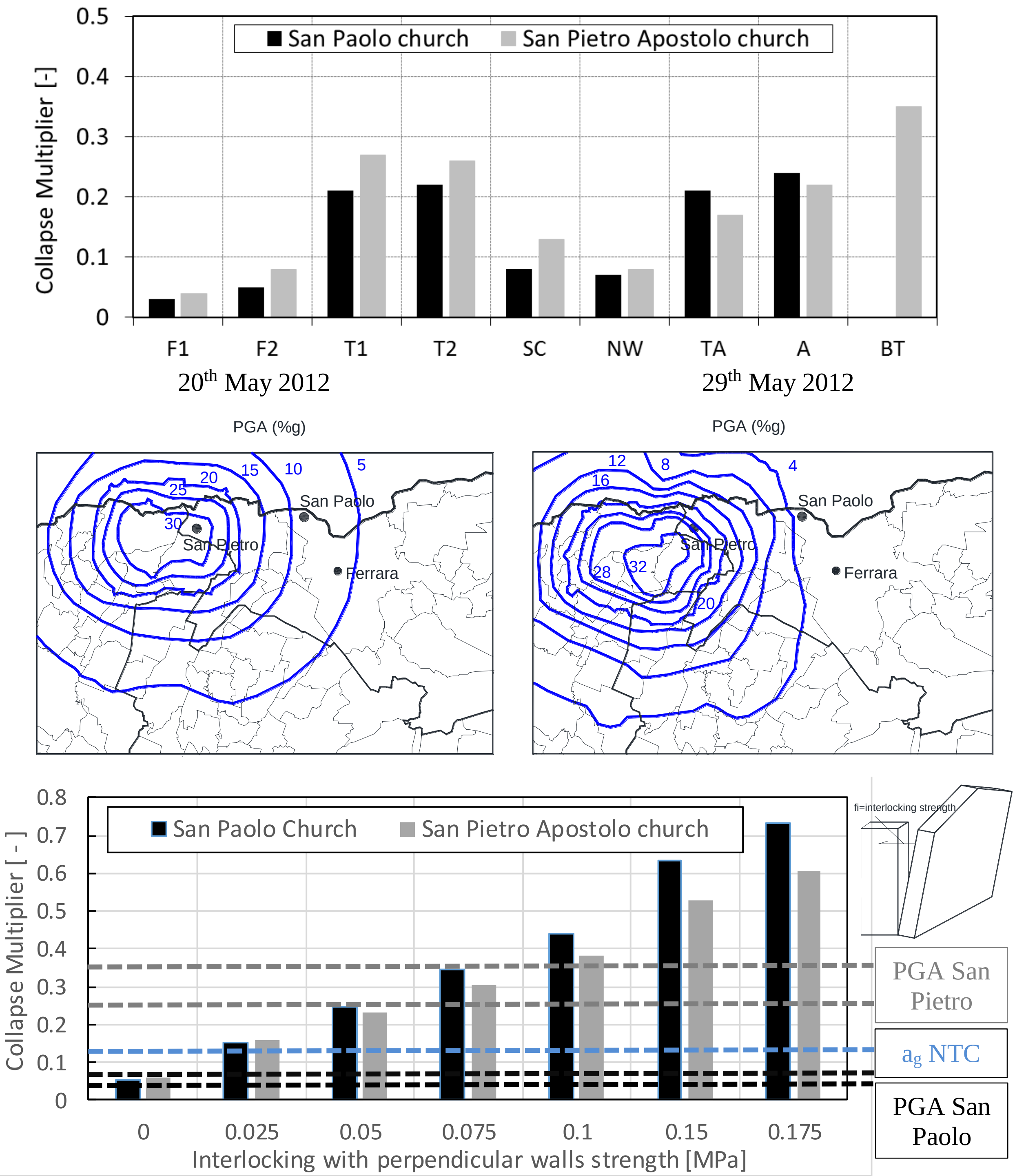


Figure 12. Top: Values of the collapse multiplier obtained through at-hand kinematic analysis for the different macro-elements of the two churches. Middle: PGAs registered during the 20th and 29th May seismic events. Bottom: Values of the collapse multipliers for the overturning of the two façades assuming a non zero interlocking strength with perpendicular walls.

From an analysis of Figure 12, it appears therefore clear that the most vulnerable macro-element is the façade for both the churches, a result consistent with the crack patterns observed after the seismic events. As can be seen again in Figure 12 (middle sub-figure), the PGAs registered during the 20th and 29th May seismic events in the region where the two churches are located were quite low for San Paolo church and moderately high for San Pietro church: the maximum horizontal acceleration a_g provided by the Italian code at the Life Safety Limit State (SLV) stands roughly in between. Without taking into account any interlocking of the façade with perpendicular walls, a prompt collapse should be expected for San Pietro church and an almost full activation of the overturning for San Paolo church. Fortunately, such events did not occur, since both façades are still standing. A crucial issue to understand such extra-resistance is to properly evaluate, in the application of the partial failure mechanisms provided by the Italian Guidelines for the built heritage, a fictitious strength f_i at the interface between perpendicular walls representing interlocking between the façade and the nave walls, as sketched in Figure 12. Such strength is not easily identifiable in practice and would require detailed in-situ surveys with non-destructive tests, but it could give a preliminary indication of the effect of a partial interlocking existing on walls crossing on corners. Adding such extra-resistance to the overturning mechanisms at increased values of f_i, the results depicted in Figure 12 are found (bottom sub-figure), where it is clearly understandable that (i) a moderate increase of the interlocking turns out into a considerable increase of the load carrying capacity, (ii) the hypothesis of zero interlocking is over-conservative in the majority of the cases and hence not consistent with reality and (iii) the determination of the real interlocking between the walls is a crucial issue for any realistic evaluation of safety, which is the base of any planned seismic upgrading.

Whilst not recommended by the Italian Guidelines for the Built Heritage (DPCM 2011), non-linear kinematic analyses can be also performed, in agreement with the Italian Code (DM 2008; Circolare 2009) and following available specialized literature (see for instance Criber, Brando and De Matteis 2015).

In Figure 13, the results of such analyses performed on both the churches are summarized. The Acceleration-Displacement (AD) format of the elastic spectrum is represented as blue curve; the Italian code allows a graphical evaluation of the demand and capacity displacements, plotting first the red segment shown in Figure 13, which represents the collapse acceleration of the macro-block (in this case the façade) at varied (deformed) configurations. The point of intersection between the red segment and the horizontal axis (acceleration equal to zero) corresponds to a varied configuration with a displacement of the control node equal to d_0*, for which the stabilizing bending moment is zero. Once d_0* is known, it is possible to determine d_u*=0.4 d_0* and d_s*=0.4 d_u*. Then, it is necessary to plot the straight line passing through the origin and the point (d_s* a_s*) lying on the red segment. Such line intersects the elastic AD spectrum in correspondence with the purple point, which represents the displacement demand: the displacement capacity is d_u*. It is also interesting to notice that for the churches under consideration, there are minimal differences on the spectrum considered, with a maximum horizontal acceleration (a_g) at the Life Safety Limit State (SLV) equal to 0.131g and 0.132g, respectively, for San Pietro church and San Paolo church, see Figure 12.

From Figure 13, it is clear that disregarding totally the interlocking strength between perpendicular walls leads to a largely unsuccessful safety assessment of the macro-block under the assigned seismic loads, whereas a small but non-zero interlocking is sufficient to increase enough the capacity.

The state of damage observed after the seismic events and the limit analysis calculations carried out (linear and non-linear) lead to conclude that the interconnection between the façade and the nave walls (for both the churches) is relatively effective, probably with a much higher quality of the masonry material for San Pietro church when compared to San Paolo church (subjected during the seismic events to PGA much lower than that predicted by the Italian Code).

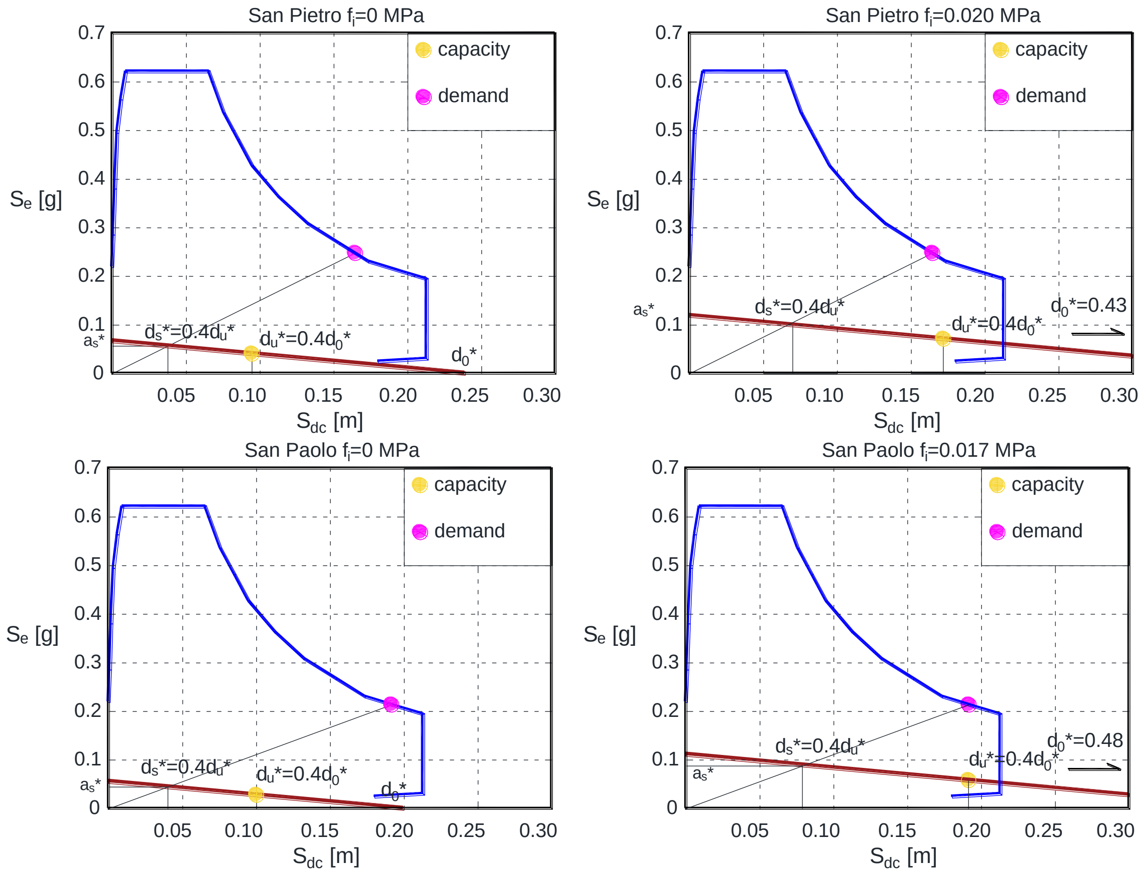


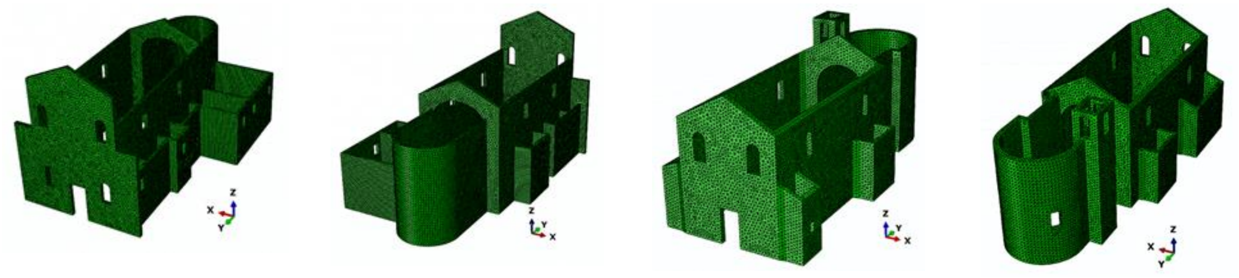
Figure 13: Results of the non-linear kinematic analyses performed on both the façades of the churches for zero interlocking (left column) and non-null interlocking sufficiently high to make the analyses satisfied.

5. FE models and material model adopted

Detailed three-dimensional FE models of the two churches were created by using the Abaqus software code (Abaqus 2014): the FE discretizations, based on tetrahedron elements, are shown in Figure 14. The FE model of San Paolo church was composed of about 258000 elements, while the FE model of San Pietro Apostolo church consisted of about 210000 elements.

It is worth noting that the dimensions of the mesh elements may play a certain role in non-linear dynamic analyses. In order to have an insight into such an issue for the cases under study, comprehensive sensitivity analyses have been carried out for a reliable investigation of the structural behavior of the churches in the non-linear dynamic range. The mesh was defined as a good compromise to perform sufficiently reliable analyses with a reduced computational effort.

It is important to highlight that some simplifications were made in the geometry of the FE models: masonry vaults and timber roofs were not modelled. Their membrane stiffness is safely assumed negligible for horizontal loads, as well as the box behavior induced by their presence in the FE models. For this reason, only vertical loads transferred by the roof to the top of perimeter walls are introduced in the models.



(a) (b)

Figure 14. FE models of the two churches: (a) San Paolo church, (b) San Pietro Apostolo church.

The Concrete Damaged Plasticity (CDP) model implemented in Abaqus was used to simulate the inelastic behavior of masonry. Such a mechanical model, developed by Lubliner et al. (1989) for concrete and further elaborated by Lee and Fenves (1998), may be applied to materials with quasi-brittle behavior such as masonry. Such an approach can be used to effectively model two typical failure mechanisms, namely tensile cracking and compressive crushing, and also to capture the material degradation under cyclic loading. The mechanical properties of masonry can be implemented in the model through uniaxial tensile and compressive constitutive laws, Figure 15. In compression, the response is linear up to the yield stress σ_{co}: the post-elastic compression phase is described by an inelastic part with hardening up to the peak stress σ_{cu} and softening behavior. In tension, the response is linear up to the peak stress σ_{to}; then it is described by a brittle behavior with a softening curve.

The degradation of the elastic stiffness is given by two scalar damage variables, d_t (tensile) and d_c (compressive), which are functions of plastic strains. The following standard relationships define the uniaxial tensile σ_t and compressive σ_c stresses:

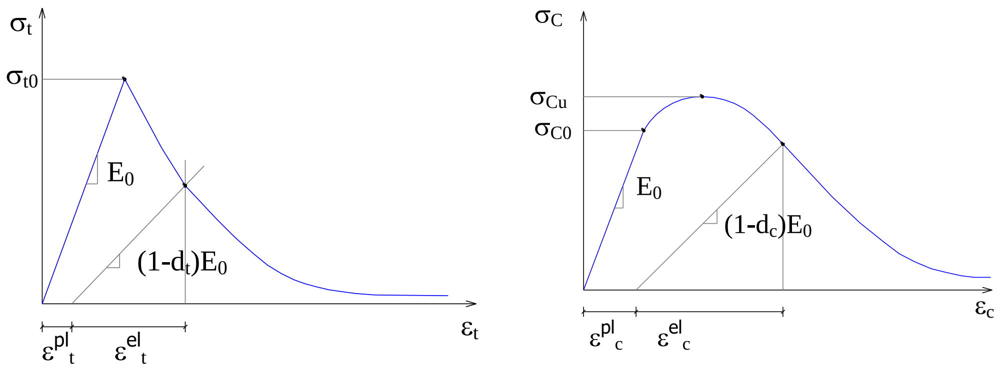
$$\begin{aligned} \sigma_t &= (1-d_t)E_0\left(\varepsilon_t - \varepsilon_t^{pl}\right) \\ \sigma_c &= (1-d_c)E_0\left(\varepsilon_c - \varepsilon_c^{pl}\right) \end{aligned} \qquad (1)$$

where E_0 is the initial elastic modulus, ε_t and ε_c are the total strain in tension and in compression, ε_t^{pl} and ε_c^{pl} are the equivalent plastic strain in tension and in compression.

The values of the main parameters adopted in the non-linear dynamic simulations for masonry with the Concrete Damage Plasticity model are the following: i) the dilation angle ψ (angle due to a variation in volume of the material following the application of a shear force) is equal to 10°, which seems reasonable for masonry subjected to a moderate-to-low level of vertical compression and is in agreement with experimental evidences available in the literature (Van der Pluijim 1993). ii) the strength ratio (ratio between the biaxial and uniaxial compression strength) is equal to 1.16, in agreement with experimental results reported in (Page 1981); iii) the correction parameter of the eccentricity ε is assumed equal to the default value 0.1 (Abaqus 2014), which implies that the material has almost the same dilation angle over a wide range of confining pressure stress values; iv) the Kc parameter (ratio between the second stress invariant on the tensile meridian and the one on the compressive meridian) is set equal to 0.666 to well approximate the Mohr-Coulomb failure criterion, as suggested by the user's Guide (Abaqus 2014); v) the viscosity parameter (numerical parameter which allows reaching convergence in softening without affecting the accuracy of the results) is set equal to 0.002 to avoid convergence problems related to softening and stiffness degradation. It is worth mentioning that a fracture energy-based regularization is used to describe the softening in tension and compression: in this way, the response of the model is not significantly influenced by the mesh size. The scalar damage variable in tension (d_t) is assumed to follow a linear variation from zero (for plastic strain corresponding to the stress peak) to 0.95 (for plastic strain equal to 0.003).

The same material was used for both the churches because they are made of a very similar masonry typology, as confirmed by on-site surveys: moreover, this assumption could be reasonably adopted because the two churches date back to a similar period and are located in the same region. Due to the lack of precise experimental data, the following mechanical properties have been assumed for a

masonry made of clay bricks and mortar, according to the indications provided in the Italian recommendations for existing buildings and built heritage (DPCM 2011; DM 2008; Circolare 2009): (i) the density and the elastic modulus are equal to 1800 kg/m^3 and 1500 MPa, respectively; (ii) the compressive strength is equal to σ_{cu}=2.4 MPa. The tensile strength is assumed equal to σ_{to}=0.17 MPa, obtaining a ratio between the tensile and compressive strength equal to about 0.07. The values adopted for the main mechanical properties of masonry seem to be reasonable taking into account the results of experimental investigations performed on masonry of churches located in the same area (Valente, Barbieri and Biolzi 2017b).
In each FE model, the main macro-elements, indicated in Figures 16-17, were highlighted and investigated in detail in the following sections.



(a) (b)

Figure 15. Representation of the masonry constitutive behavior in (a) tension and (b) compression.

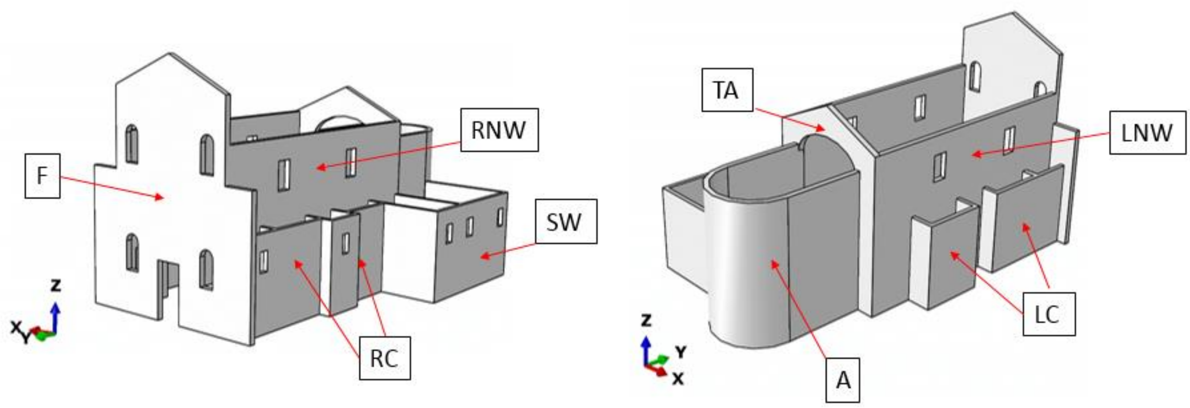


Figure 16. San Paolo Church. Indication of the different macro-elements in the FE model. Notation: F=Façade, LNW=Left nave wall, RNW=Right nave wall, TA=Triumphal arch, A=Apse, LC=Left chapels, RC=Right chapels, SW=Sacristy wall.

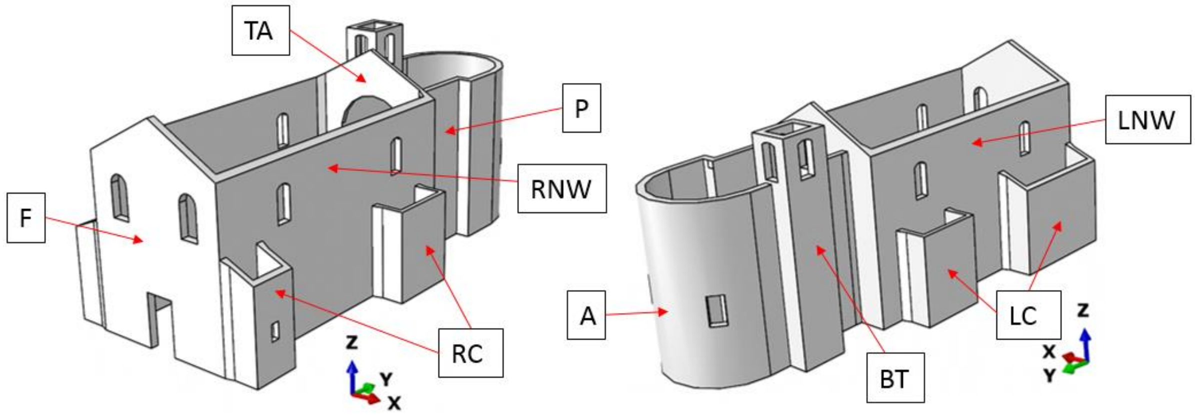


Figure 17. San Pietro Apostolo church. Indication of the different macro-elements in the FE model. Notation: F=Façade, LNW=Left nave wall, RNW=Right nave wall, TA=Triumphal arch, A=Apse, P= Presbytery, LC=Left chapels, RC=Right chapels, BT=Bell tower.

6. Modal Analysis

An eigen-frequency analysis is carried out on the three-dimensional FE models in order to obtain a preliminary assessment of the dynamic behavior of the churches under study. Figures 18-19 show the modal deformed shapes of the first main modes, the corresponding periods and the participating mass ratio along the two main directions for the two churches. The first main modes characterized by a participating mass ratio larger than about 4% are considered.

6.1. San Paolo church

The first mode (T=0.483 s) involves the upper part of the façade with a participating mass ratio of about 5% in the longitudinal direction. The second (T=0.378 s) and third (T=0.360 s) modes involve mainly the two nave walls with a participating mass ratio of 11.25% and 6.41%, respectively, in the transversal direction: it is worth noting that the side chapels and annexes are not the same for the two lateral walls. The fourth mode (T=0.273 s) concerns mainly the nave walls, the triumphal arch and the apse, with a participating mass ratio of 7.25% in the transversal direction. The twenty-third mode (T=0.103 s) exhibits the maximum value (22%) of participating mass ratio in the longitudinal direction. The first one hundred modes correspond to a total participating mass ratio of 84% in the longitudinal direction and 85% in the transversal direction.

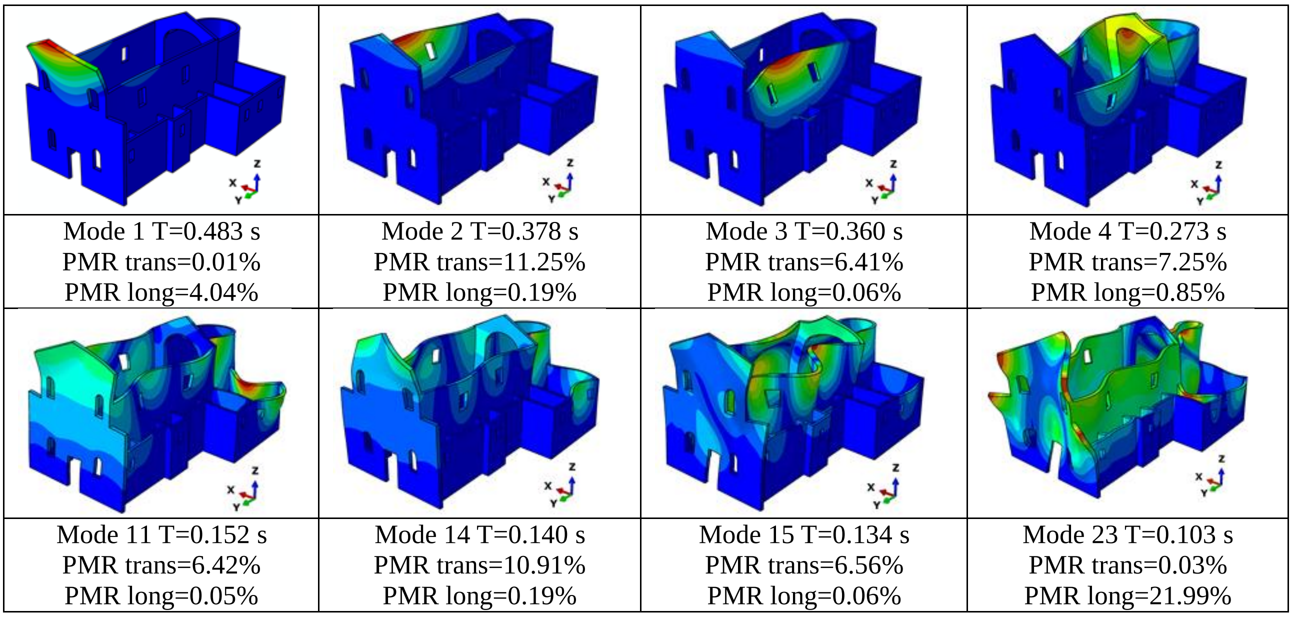


Figure 18. San Paolo Church. Deformed shapes of the first eight main modes, corresponding periods and participating mass ratio in the transversal and longitudinal directions.

6.2. San Pietro Apostolo church

The first (T=0.358 s) and second (T=0.296 s) modes involve the right and left walls of the nave with a participating mass ratio of 8.59% and 13.42% in the transversal direction, respectively. The third mode (T=0.261 s) concerns the left wall of the nave, the presbytery zone and the bell tower with a participating mass ratio of about 11.1% in the transversal direction. The fifth mode (T=0.218 s) involves the façade with a participating mass ratio of about 4.2% in the longitudinal direction. Among the first modes, the fourteenth mode (T=0.112 s) exhibits the highest value (7%) of participating mass ratio in the longitudinal direction. The first one hundred modes correspond to a total participating mass ratio of 83% in the longitudinal direction and 86% in the transversal direction.

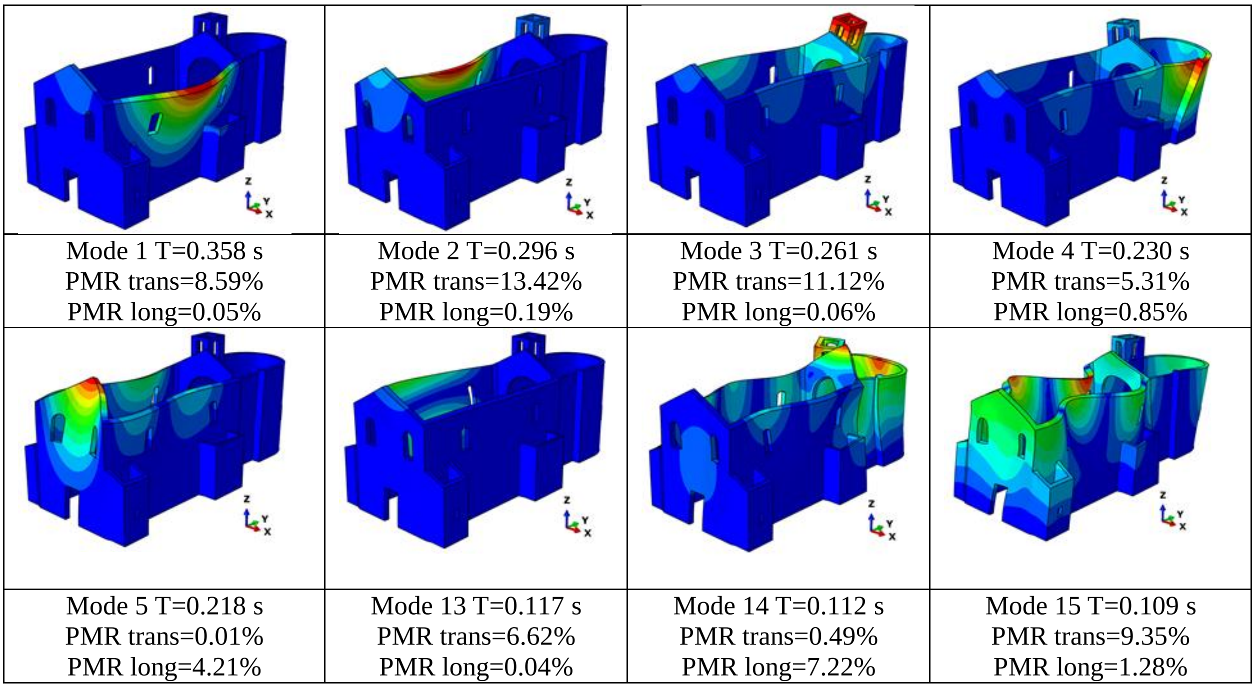


Figure 19. San Pietro Apostolo church. Deformed shapes of the first eight main modes, corresponding periods and participating mass ratio in the transversal and longitudinal directions.

Figure 20 shows the distribution of the first one hundred modes in the longitudinal and transversal directions for the churches under study. The results indicate that the transversal direction is the critical direction for both the churches; moreover, San Paolo church presents a local weakness in the upper part of the façade. It is worth mentioning that modal analysis can represent a fast and useful tool to provide a preliminary explanation of the damage occurred during the earthquake, taking into account the extended damage observed mainly in the nave walls of both the churches.
Moreover, it can be noted that low values of participating mass ratios are associated with the first main modes, highlighting that the dynamic response of the churches is characterized by the local behavior of the different macro-elements: several modes are necessary to describe the behavior of the churches. It is important to observe that, neglecting the first mode that involves the upper part of the façade of San Paolo church, quite similar values of periods can be observed for the first main modes of the two churches. Moreover, it is worth mentioning that only the third mode involves the bell tower of San Pietro Apostolo church, because it is small and embedded in the structure.

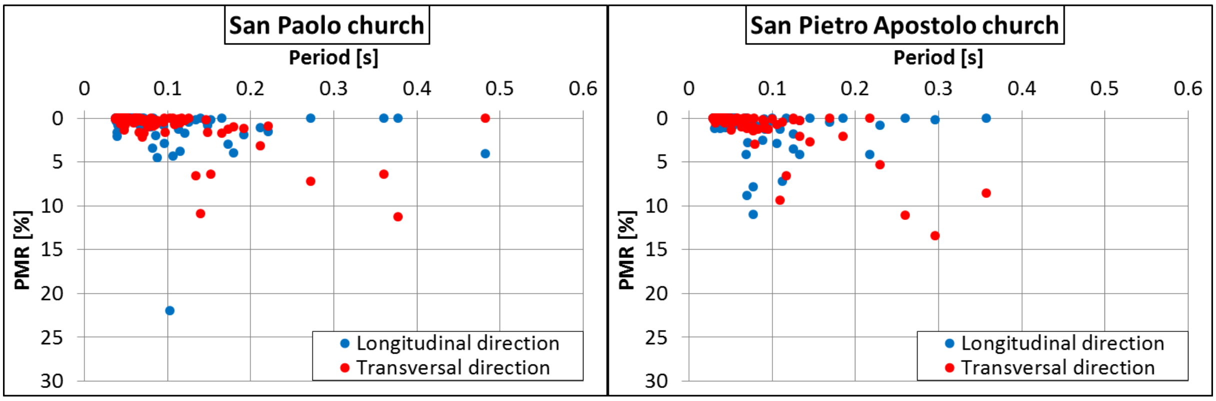


Figure 20. Distribution of the first one hundred modes in the longitudinal and transversal directions for the churches under study: modal participating mass ratio as a function of the corresponding vibration period.

7. Non-linear dynamic analysis

The seismic response of the two churches is studied through non-linear dynamic analyses by assuming as seismic input the real accelerogram registered during the first main shock on May 29 and recorded in the municipality of Mirandola. Due to the high computational efforts required by the analyses, the numerical simulations are conducted by selecting a part (10 s) of the record. Figure 21 shows the horizontal components of the accelerogram used in the non-linear dynamic analyses, adopting equal intensity (PGA) in the two orthogonal directions. Non-linear time history analyses are conducted using implicit integration method: a Rayleigh damping model is adopted assuming that the global damping matrix C is obtained as a linear combination of the global stiffness K and mass M matrices.
The tensile damage contour plots, obtained at the end of the numerical simulations with three different peak ground accelerations ranging between PGA=0.1g and PGA=0.2g, are reported for the two churches, providing a deep insight into the evolution of tensile damage during the numerical simulations. The seismic response is also evaluated in terms of maximum normalized displacements and EDDTD for the main macro-elements composing the churches under different PGA values: it is useful to explain that EDDTD is an acronym for Energy Density Dissipated by Tensile Damage and it measures the sum of the areas inside a load-unload cycle of a Gauss Point limited to the tension-tension region.

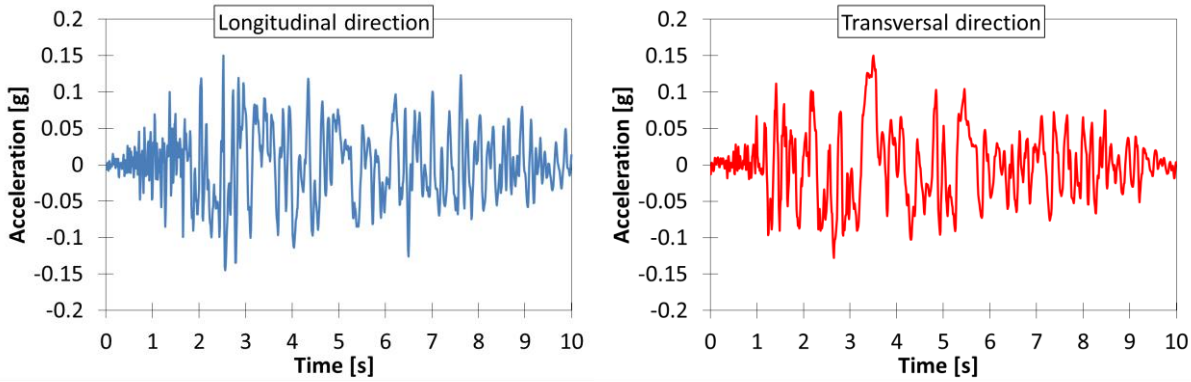


Figure 21. Horizontal components of the real accelerogram used in the non-linear dynamic analyses: longitudinal direction (left) and transversal direction (right).

7.1. San Paolo church

Figure 22 shows the tensile damage contour plots of San Paolo church at the end of the non-linear dynamic analyses for three different PGA values. It is worth noting that a significant increase of damage can be observed when the seismic intensity level varies from PGA=0.1g to PGA=0.2g.

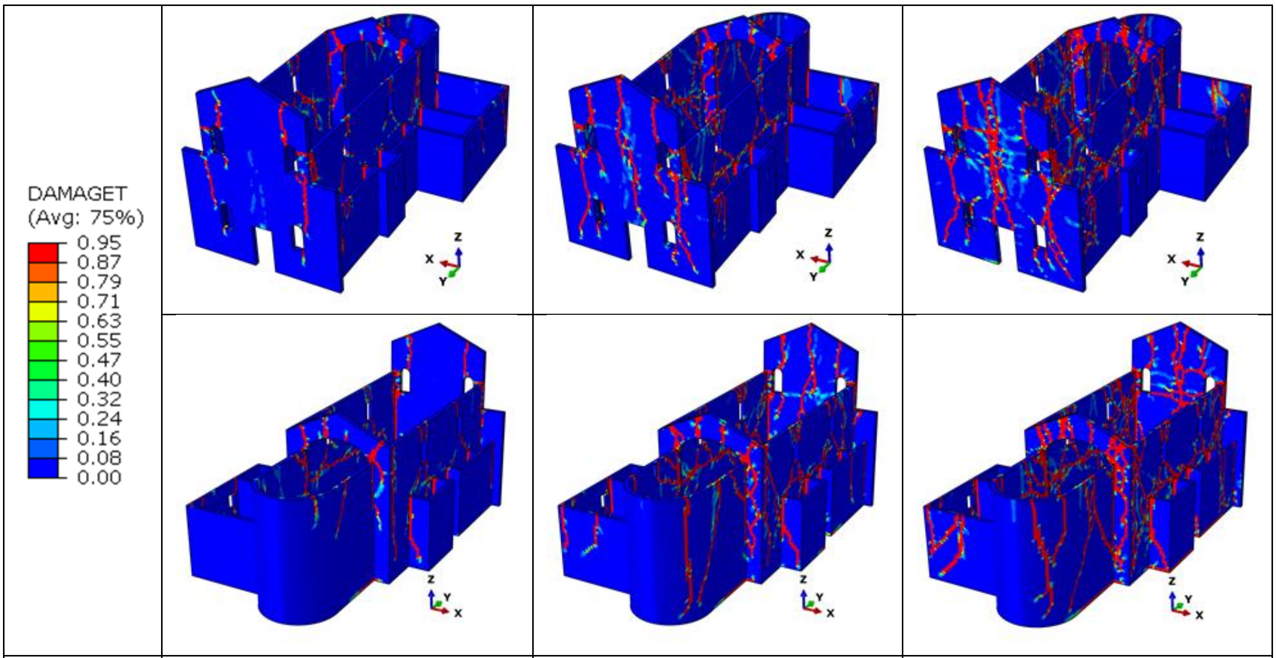
-Under PGA=0.1g the façade exhibits some vertical cracks involving the four openings. The analysis under PGA=0.15g shows the formation of a large vertical crack in the central part (from the top of the tympanum to the door) and two lateral cracks in the edge parts. Under PGA=0.2g damage increases and extends considerably all over the façade. It is important to highlight that the façade of the church is rather slender and there is a significant height difference (17.4 m vs 12.1 m) between the top of the tympanum and the nave walls. Moreover, it can be noted that the numerical model shows a more widespread damage than the real case, particularly with regard to the edge parts.

-The nave walls exhibit diagonal and vertical cracks near the openings, even under PGA=0.1g, in agreement with the on-site survey; evident horizontal damage is present along the walls at the height of the side chapels. Severe vertical cracks can be observed in the connection regions between the nave walls and the façade, even under PGA=0.1g, as emerged from the on-site survey.

-A considerable damage is registered in the triumphal arch, which exhibits vertical cracks in the central part and diagonal cracks at the edges: moreover, the vertical elements are significantly damaged. These results are reflected in the main outcomes derived from the on-site survey.

-The side chapels exhibit clear vertical cracks close to the connection regions with the nave walls, even under PGA=0.1g: it is pretty similar to what has been described in Section 3.1.

-The apse presents negligible damage under PGA=0.1g: on the contrary, several vertical cracks starting from the top of the walls can be observed for higher PGA. These results are consistent with what has been highlighted in the direct damage survey.



	PGA=0.1g	PGA=0.15g	PGA=0.2g

Figure 22. San Paolo church. Tensile damage contour plots at the end of the non-linear dynamic analyses with three different PGA.

Figure 23 shows the maximum normalized displacement (displacement divided by height) registered for the main macro-elements of the church in the two orthogonal directions during the non-linear dynamic analyses with different PGA values.

-Under PGA=0.1g, the maximum normalized displacement is computed for the façade (0.3%) in the longitudinal direction, and for the nave walls, the triumphal arch and the apse (around 0.2%) in the transversal direction.

-Under PGA=0.2g, the façade presents very high normalized displacements (larger than 1%) in the longitudinal direction. In the transversal direction the highest normalized displacement (0.7%) is registered for the left nave wall, which is less restrained than the right one; slightly smaller normalized displacements (higher than 0.6%) are observed for the triumphal arch and the apse.

Figure 24 shows the displacement time-history registered for the façade and the left nave wall in the longitudinal and transversal directions, respectively, during the non-linear dynamic analyses with two different PGA values (PGA=0.1g and PGA=0.2g). It has to be noted that a large increase of residual displacements of the façade occurs in the case of PGA=0.2g, indicating an onset of collapse mechanism of the tympanum: a remarkable increase of the residual displacement can be observed also for the left nave wall. It is also important to remark that in the global FE model there is a non zero interlocking between perpendicular walls, because corners are modeled with elements perfectly bonded that behave exactly in the same manner as the selected masonry material (i.e. with small peak tensile strength and softening). Obviously, such hypothesis has the effect to slightly increase the load carrying capacity and hence decrease residual displacements. As far as the activation of a mechanism is concerned, the consequence is that macro-blocks are retained –at least partially- against overturning under pure mode I, especially when the results are compared with the failure mechanisms provided by the Italian Guidelines for the Built Heritage without accounting for interlocking.

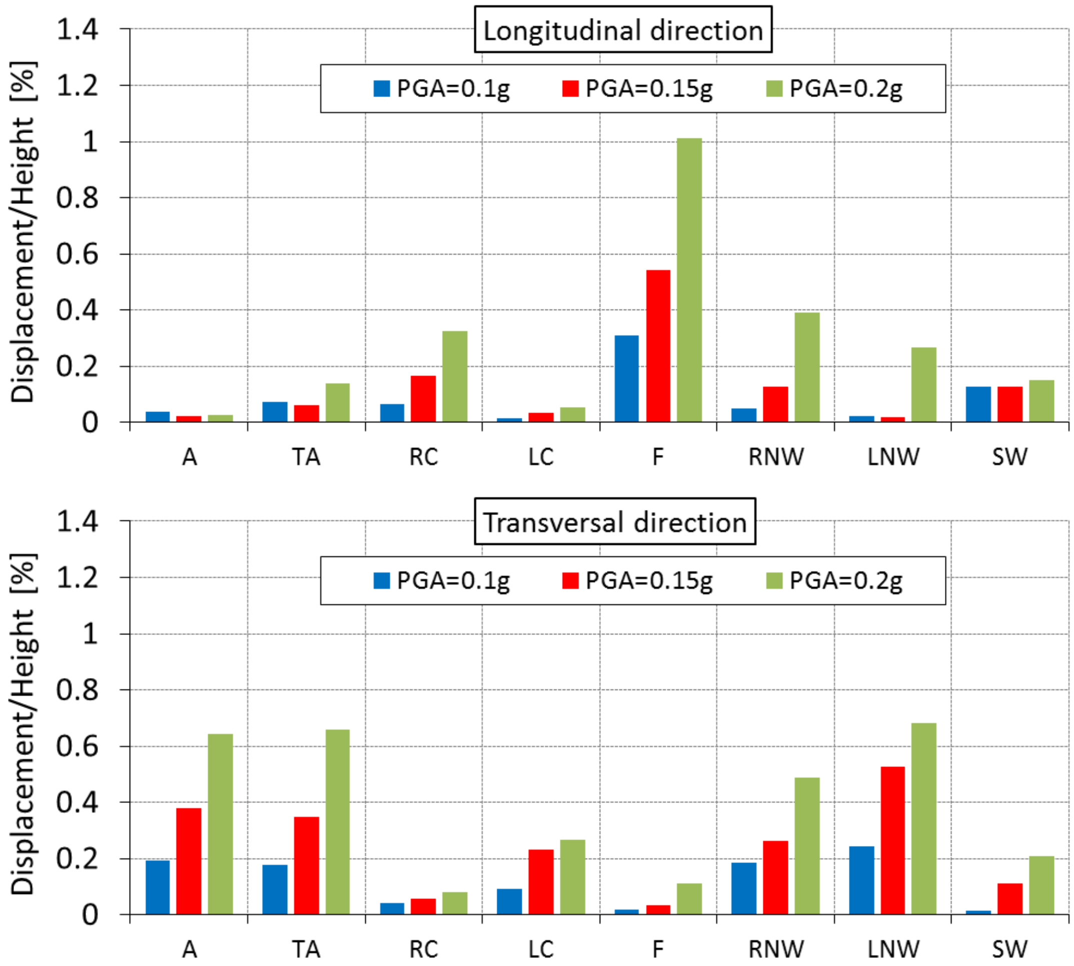


Figure 23. San Paolo church. Maximum normalized displacement registered for the main macro-elements in the longitudinal and transversal directions during the non-linear dynamic analyses with different PGA.

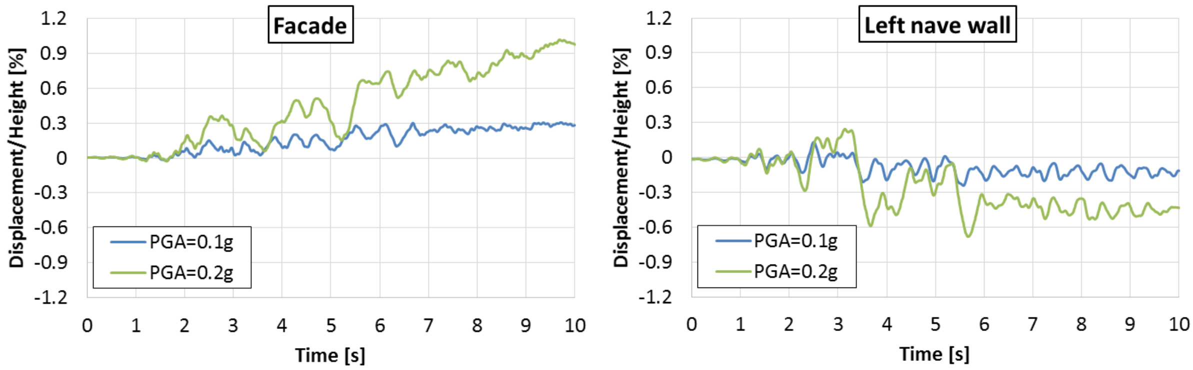
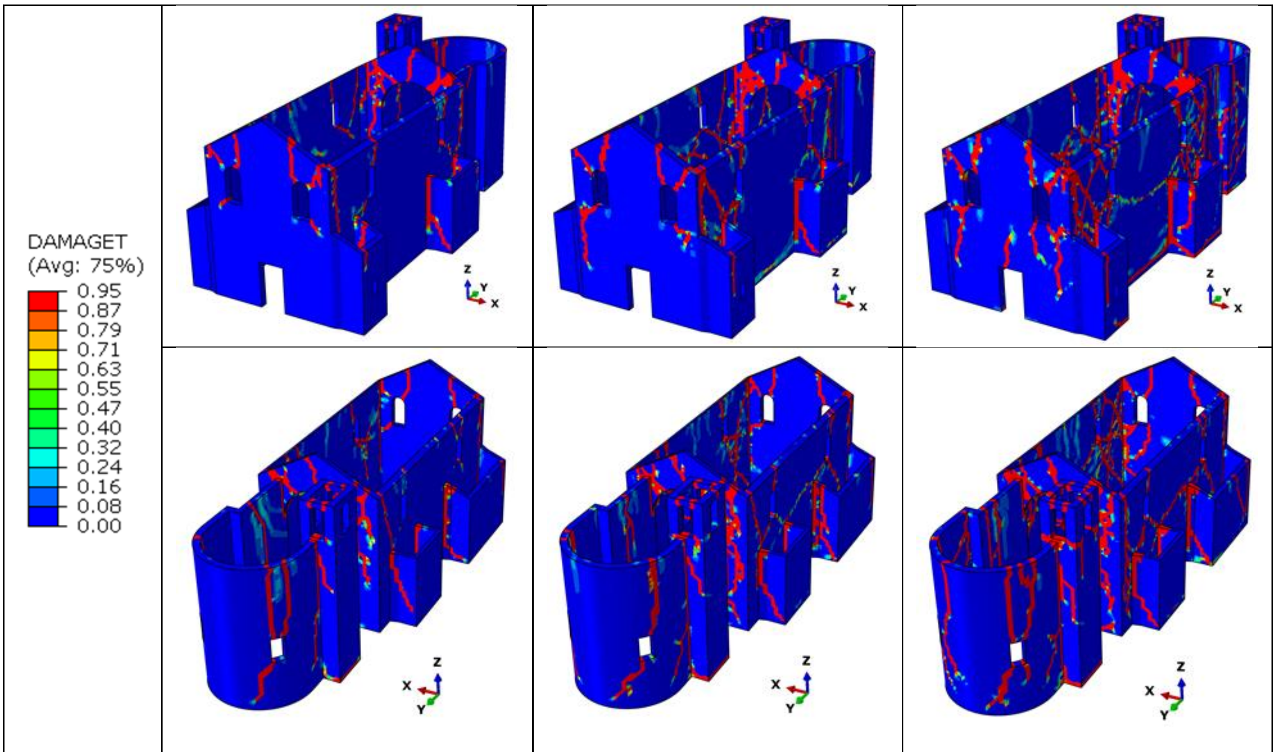


Figure 24. San Paolo church. Displacement time-history registered for the façade and the left nave wall in the longitudinal and transversal directions, respectively, during the non-linear dynamic analyses with two different PGA.

7.2. San Pietro Apostolo church

Figure 25 shows the tensile damage contour plots of San Pietro Apostolo church at the end of the non-linear dynamic analyses for three different PGA levels. It is worth noting that widespread damage can be observed even for PGA=0.1g.

-Under PGA=0.1g, the façade presents some important cracks in the upper part, near the two openings. For higher PGA, damage increases and extends to the bottom part, showing two main cracks in correspondence with the nave walls; moreover, an evident crack can be observed above the main door under PGA=0.2g. It can be noted that damage is more evident than that observed during the on-site survey.
-The nave walls exhibit significant damage under PGA=0.2g. A horizontal crack is visible mainly in the left wall at the height of the side chapel: it may be probably due to the out-of-plane deflection of the upper part of the wall. Severe vertical cracks, which are consistent with the real damage, can be observed in the connection regions with the façade, even under PGA=0.1g,
-The triumphal arch shows extensive damage with a distribution that is comparable with the crack pattern observed during the field survey: vertical cracks can be observed in the central part and diagonal cracks are registered at the edges.
-The apse exhibits marked vertical cracks near the openings and at the edges, near the bell tower, even under PGA=0.1g, as confirmed by the field survey.
-It can be noted that the bell tower presents damage in the upper part, near the openings, and along the body, even under PGA=0.1g: important vertical cracks can be observed near the connection regions with the walls of the apse and the presbytery.
-Severe vertical cracks, which are in a good agreement with the surveyed damage, are registered in the connection regions between the side chapels and the nave walls, even under PGA=0.1g.



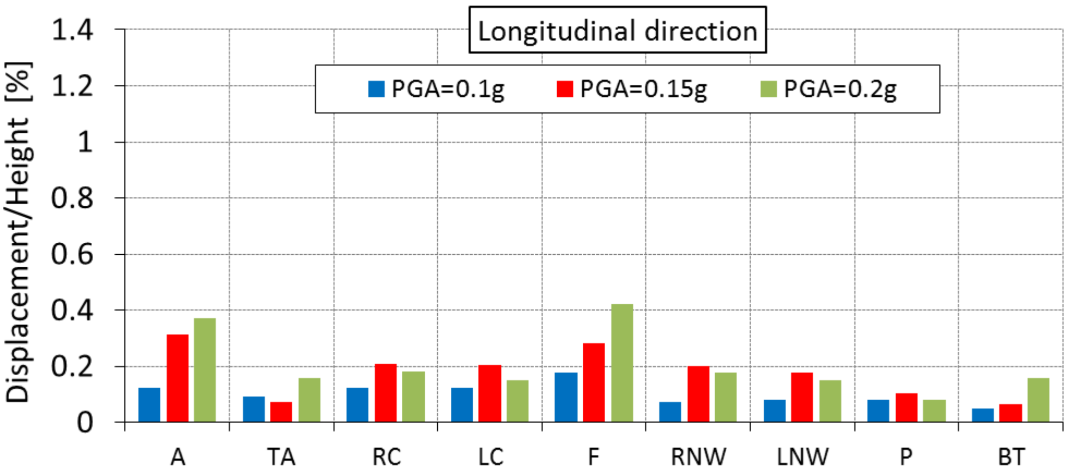
PGA=0.1g PGA=0.15g PGA=0.2g

Figure 25. San Pietro Apostolo church. Tensile damage contour plots at the end of the non-linear dynamic analyses with three different PGA.

Figure 26 shows the maximum normalized displacement registered for the main macro-elements of San Pietro Apostolo church in the two orthogonal directions during the non-linear dynamic analyses with different PGA.
-Under PGA=0.1g, the maximum normalized displacement is observed for the façade (0.18%) in the longitudinal direction and for the right nave wall and the side chapel (about 0.2%) in the transversal direction.

-Under PGA=0.2g, in the transversal direction the maximum normalized displacement is much larger (0.61%) for the right nave wall, which is less restrained by the side chapels when compared with the left nave wall. The normalized displacement of the bell tower is limited and smaller than 0.2% in both the directions. The façade presents normalized displacements larger than 0.4% in the longitudinal direction.

Figure 27 shows the displacement time-history registered for the façade and the right nave wall in the longitudinal and transversal directions, respectively, during the non-linear dynamic analyses with two different PGA values (PGA=0.1g and PGA=0.2g). It has to be noted that the residual displacements are small for both the macro-elements in the case of PGA=0.1g; conversely, under PGA=0.2g a clear increase of the residual displacements can be observed.



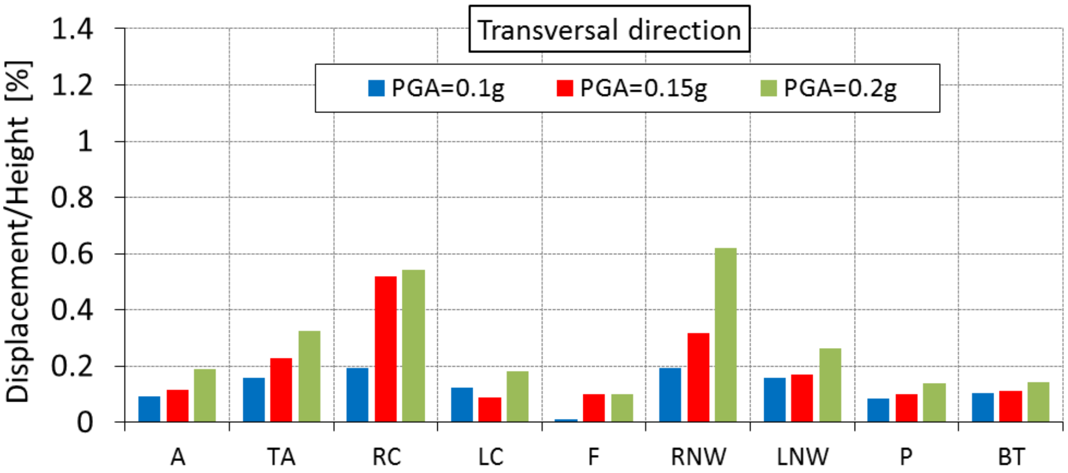


Figure 26. San Pietro Apostolo church. Maximum normalized displacement registered for the main macro-elements in the longitudinal and transversal directions during the non-linear dynamic analyses with different PGA.

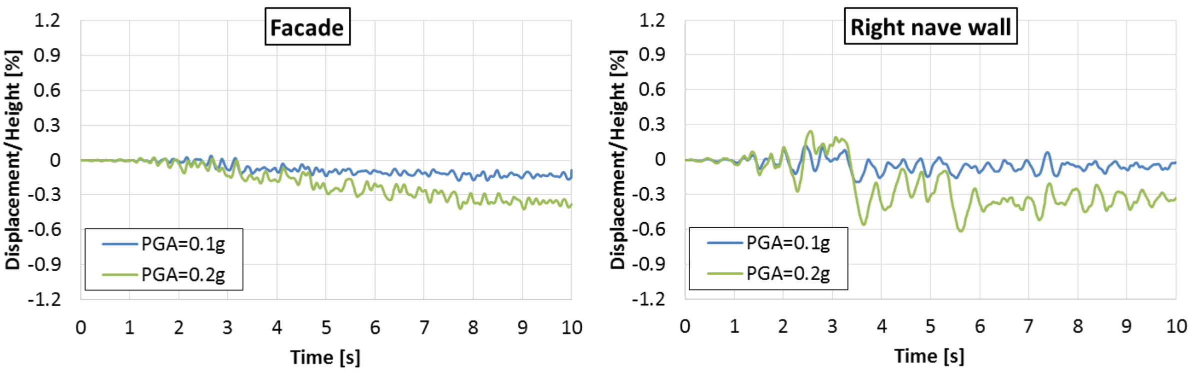


Figure 27. San Pietro Apostolo church. Displacement time-history registered for the façade and the right nave wall in the longitudinal and transversal directions, respectively, during the non-linear dynamic analyses with two different PGA.

7.3. Normalized base shear and energy density dissipated by tensile damage

Figure 28 shows the maximum value of the base shear normalized by the weight of the church in the two orthogonal directions during the non-linear dynamic analyses with different PGA values. It can be noted that the values of the normalized base shear are similar in the transversal direction and slightly larger for San Paolo church in the longitudinal direction under PGA=0.2g. It is important to observe that the highest values of the normalized base shear are registered in the longitudinal direction for both the churches, as could be derived from the preliminary results of the modal analysis indicating a lower stiffness in the transversal direction.

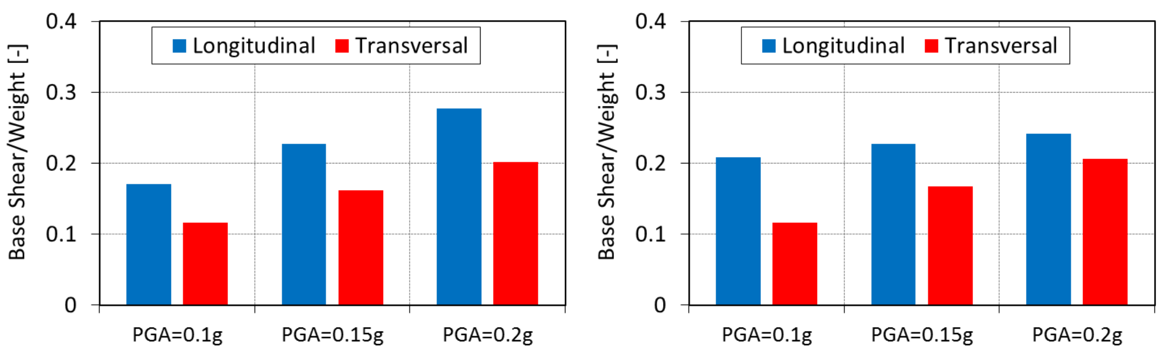


Figure 28. Maximum base shear/weight ratio in the two orthogonal directions during the non-linear dynamic analyses with different PGA for San Paolo church (left) and San Pietro Apostolo church (right).

Figure 29 shows the evolution of the global energy density dissipated by tensile damage (EDDTD) for the two churches under different PGA. The values of the EDDTD are similar for both the churches, with slightly larger values for San Paolo church. A large increase of the EDDTD can be observed in the case of PGA=0.2g for both the churches.

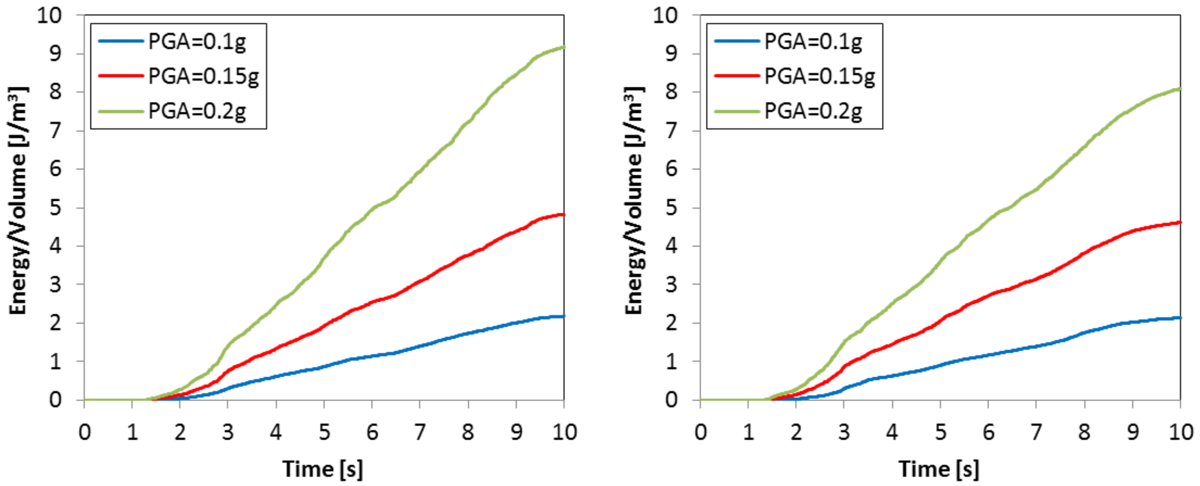


Figure 29. Energy density dissipated by tensile damage for the two churches during the non-linear dynamic analyses with different PGA: San Paolo church (left) and San Pietro Apostolo church (right).

Figure 30 shows the energy density dissipated by tensile damage (EDDTD) for the main macro-elements of the two churches under different PGA. The highest value of the EDDTD is computed for the triumphal arch of both the churches, for all the PGA values. In the case of San Paolo church, high values of the EDDTD are also registered for the façade and the nave walls. As regards San Pietro Apostolo church, excluding the triumphal arch, a more uniform distribution of the EDDTD can be observed, with slightly larger values registered for the apse, the presbytery and the right nave wall.

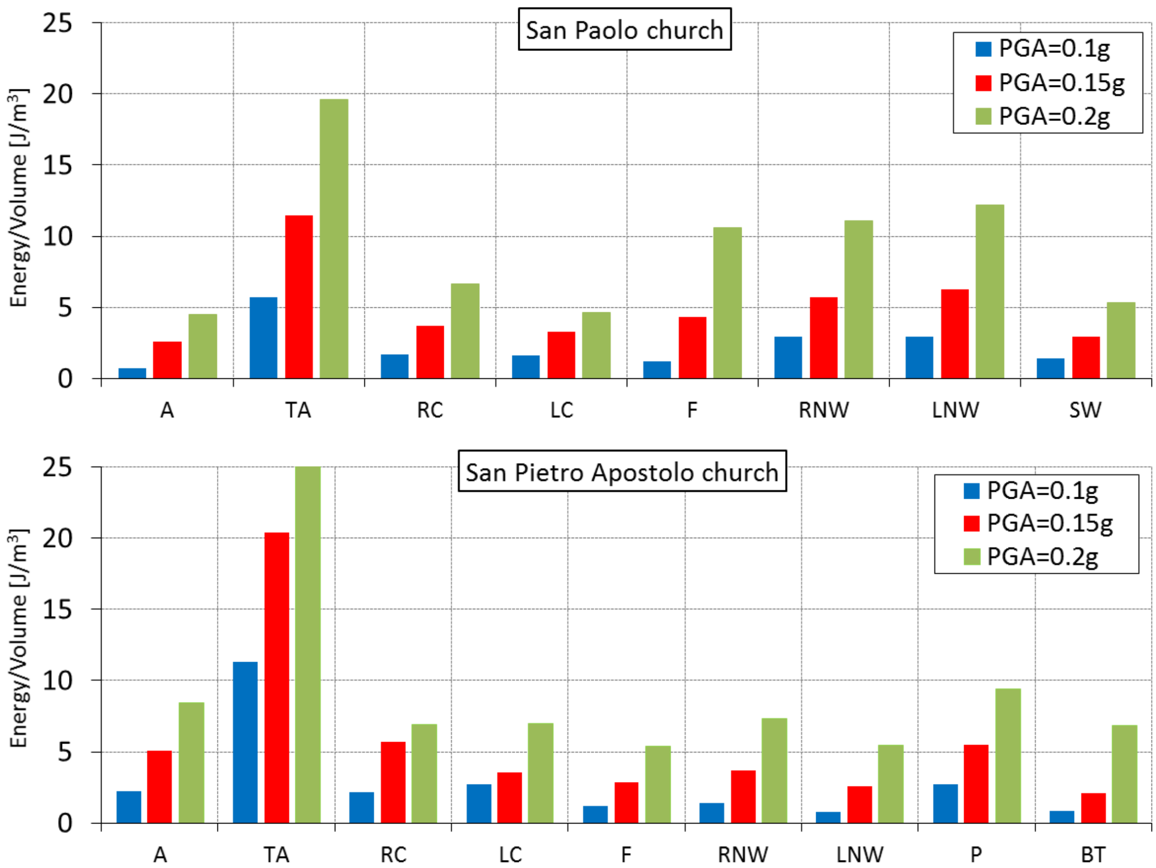


Figure 30. Energy density dissipated by tensile damage for the main macro-elements of the two churches at the end of the non-linear dynamic analyses with different PGA.

8. Discussion and comparison of the results

It is important to notice that, on the whole, a similar behavior can be observed for the two churches and a good agreement between numerical results and real damage can be registered. The post-earthquake damage surveys provided valuable information on the crack patterns and the non-linear dynamic analyses conducted on detailed FE models made it possible to simulate the damage distribution of both the churches for different PGA values.

- A simplified procedure based on a kinematic approach was performed for a preliminary assessment of the local collapse mechanisms that can develop on the main macro-elements of the two churches. It is important to highlight that the approach is straightforward, simple and directly usable without using any FE code. However, it is affected by possible inaccuracies: 1) the real geometry of the church is accounted for only in an approximate way; 2) the assumption of a no tension material model for masonry may result in an underestimation of the load carrying capacity, leading to collapse multipliers that are extremely low. In this study the critical elements identified by such a simplified approach were the façade and the nave walls, characterized by low values of the resultant collapse multipliers for both the churches.

-The non-linear dynamic analyses provided a deep insight into the seismic behavior of the two churches and allowed for a comparison with the real damage observed after the earthquake.

-The highest values of the normalized displacement in the longitudinal direction were registered for the façade of both the churches. It has to be noticed that the maximum normalized displacement was much larger for San Paolo church than for San Pietro Apostolo church. Moreover, the façade of San Paolo church exhibited more significant damage than that of San Pietro Apostolo church, as evidenced by the cracks observed during the field survey.

-For both the churches extensive vertical damage was registered in the connection regions between the façade and the nave walls, indicating the possibility of an overturning mechanism in case of bad interlocking. Such a damage was clearly observed during the on-site survey for both the churches.

-The highest values of normalized displacement in the transversal direction were registered for the nave walls of both the churches: it can be noted that the values were similar for the two churches. Marked cracks starting from the openings of the nave walls were directly observed for both the churches during the field survey.

-The maximum value of the energy density dissipated by tensile damage was registered for the triumphal arch of both the churches, as also confirmed by the considerable damage observed in the central part and at the edges of the triumphal arch in the contour plots. It is worth mentioning that the triumphal arch of San Paolo church exhibited high normalized displacements in the transversal direction. It can be noted that the field survey showed significant cracks in the triumphal arch of both the churches.

-The side chapels presented severe damage in the connection regions with the nave walls for both the churches, as emerged by the field survey. The maximum normalized displacements in the transversal direction were slightly larger for the side chapels of San Pietro Apostolo church than those of San Paolo church.

-The apse of both the churches showed widespread vertical cracks at the end of the non-linear dynamic analysis with PGA=0.2g. Damage was particularly significant for the apse of San Pietro Apostolo church, showing severe cracks in correspondence with the openings, as confirmed by the field survey. In San Paolo church, the apse exhibited some small cracks under PGA=0.1g and widespread damage under PGA=0.2g.

-The bell tower of San Pietro Apostolo church showed significant damage in the belfry and in the connection regions with the church: such a result was consistent with the findings of the field survey.

9. Conclusions

This paper has investigated the cracks patterns and the seismic response of two churches damaged during the 2012 Emilia earthquake through detailed field surveys and different methods of analysis. A simplified approach directly based on the limit analysis theorems, modal analyses and sophisticated three-dimensional non-linear dynamic analyses adopting an elasto-plastic damage model for masonry have been used to identify the most vulnerable elements of the churches under study.

-The seismic vulnerability assessment of the churches carried out through the simplified approach given by Italian Guidelines on the Built Heritage, together with the visual observations of the widespread damage during several field surveys, have demonstrated that the examined structures are highly vulnerable to seismic actions. Limit analysis with pre-assigned failure mechanisms is a simple and straightforward method and may provide a preliminary estimation of the collapse loads and the active partial failure mechanisms. However, the results are approximate because the method is intrinsically affected by possible inaccuracies, leading to resultant collapse multipliers that are extremely low. The macro-elements with higher vulnerability were found to be the overturning of the façade and the nave walls.

-The results of the modal analysis have shown a low stiffness in the transversal direction for both the churches. Neglecting the first mode that involves the upper part of the façade of San Paolo church, similar values of periods can be observed for the first main modes of the two churches.

-Refined non-linear dynamic analyses performed on detailed FE models were fundamental tools to simulate the damage distribution of both the churches under different PGA values. Numerical results have indicated that widespread damage can be registered even for PGA=0.1g, in particular for San Pietro Apostolo church, as emerged from post-earthquake surveys. In the case of San Paolo church, the façade exhibits extensive damage with incipient overturning and the triumphal arch is strongly damaged. As regards San Pietro Apostolo church, widespread damage is observed in the facade, in the nave walls and in the triumphal arch.

-It can be noted that similar results are obtained for the two churches. The triumphal arches present the highest values of energy density dissipated by tensile damage: moreover, the maximum displacements are registered for the façade in the longitudinal direction and for the nave walls and the triumphal arch in the transversal direction.

-A good agreement between numerical results and real damage can be observed, demonstrating the effectiveness of the numerical approach adopted. The results obtained in this study can be used to better understand the seismic performance of other masonry churches with similar characteristics located in the same region.

Acknowledgements

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