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MODELLING AND EFFECTS OF SEISMIC IMPROVEMENT INTERVENTIONS ON CHURCH ROOFS

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Abstract: *The effectiveness of interventions for seismic improvement is a fundamental issue for the preservation of heritage churches, which are especially prone to earthquake damage. Consequently, a significant research effort is currently devoted to analyze observed damage cases for structures that had undergone different kinds of intervention before the earthquake, in order to confirm and optimize those with positive effect and rule out operations that may evolve into critical situations. To this purpose, the development of realistic but manageable numerical models must face issues related to the presence of different materials, complex geometries, partial and often uncertain constraint conditions, exceptional levels of action, and the resulting nonlinearities. This work focuses on repair and improvement interventions carried out on the roof of a church, specifically San Martino dei Gualdesi, proposed by the ReLUIS Consortium as case study. The church was seriously damaged by the 2016 Central Italy earthquake but had undergone damage and repair also in previous earthquakes. In a first set of interventions the original roof had been repaired by highly increasing its mass and stiffness, according to the technique adopted at the time. Subsequently, an improvement was performed by applying a lighter roof structure, which was the configuration that underwent the 2016 earthquake. A finite element model of the whole church has been developed and static and modal analyses have been performed to characterize its behaviour, before focusing on the interaction between the church and the bell tower, attached to the church and connected to the roof structure. The bell tower was highly damaged during the event of 2016. A separate model of the tower was then adopted, after calibrating specific boundary conditions that could represent satisfactorily the continuity with the main church body and the different interconnection with the roof structure in the two cases of massive and lighter roof. Linear and nonlinear structural analyses were performed on such bell tower model. Nonlinear dynamic analyses were carried out considering three-directional motion for the October 2016 event, which was most damaging for the church, with records available from a recording station in the area. Damage development and cumulation in time could be followed for the two roof configurations. The results are discussed also in comparison with other cases that had a similar history of interventions.*

1. Introduction

The frequent occurrence of damaging earthquakes in Italy threatens the survival of the Country's architectural heritage. Churches are the most numerous heritage assets, built over many centuries with a variety of typologies and styles. Within such variety, common structural characteristics (large and slender walls often weakened by large openings, wide-span halls, no diaphragms to foster a global collaborative response, etc.)

make churches particularly prone to seismic damage. Considerable research effort has been devoted to identify the main causes of vulnerability and to propose appropriate interventions for repairing damage and, in a perspective of prevention and conservation, to improve the capability to produce an adequate structural response. In many cases, the occurrence of a new earthquake after seismic damage had occurred and repairs and improvement had been performed has given the possibility to assess the validity of such interventions. As a result, new versions of the design code have included updated indications and procedures. Such calibration and interpretation work is still in progress due to the complexity of the problem when working with heritage structures. The need remains for examining in detail a large number of cases.

The work presented here is a contribution within a Task devoted to churches in a National Research Program on vulnerability reduction by the ReLUIS Consortium of Universities and DPC (Civil Protection Agency). The case study proposed is a church damaged during the 2016 Central Italy earthquake. The church had undergone previous earthquakes and had been subjected to interventions for repair and improvement. Different intervention strategies on the church roof and the effects on the adjacent bell tower are discussed based on damage observation and numerical simulations.

2. Repair and seismic improvement of church structures

The long series of earthquakes that have damaged churches since the second part of the 20th century have brought to identify and study the most frequent damage mechanisms (Doglioni et al. 1994; Lagomarsino & Podestà 2004 a, 2004b) and to propose interventions to reduce their occurrence. Improvement interventions, often performed while repairing previous damage, may amount to increase the shear strength of masonry which is often of poor quality in older structures, to avoid overtopping of loosely restrained elements and, mostly, to foster a better collaborative behavior of perimeter walls. This is often scarce in such buildings due to the lack of horizontal diaphragms and to poor interconnections of the main walls, particularly the façade), Interventions have often concerned the roof for the role that roofs have on the global response (e.g. Parisi et al. 2008). Roofs, if well connected, may offer a favorable link among vertical elements, reducing out-of-plane mechanisms of the external nave walls and overtopping of the façade. Moreover, the timber roof structure may be degraded or inadequately designed and may need upgrading operations. In such perspective, interventions on roofs have been widely implemented. Initially, the concept was to create a strong link often transforming the roof surface in a stiff slab, sometimes substituting the trusses underneath with reinforced concrete products of similar shape. The linking effect was created, but the excessive mass and stiffness of new elements and of the slab soon appeared incompatible with the underlying masonry. Damage and many cases of collapse during earthquakes occurred after the intervention, showing the inconsistency of such approach. (e.g. Lagomarsino & Podestà, 2004c, Binda et al. 2010; Parisi & Chesi, 2014, Parisi et al. 2016). Often, during new repairs excessive masses have been removed and substituted with lighter solutions; at times such reduction has been performed in a preventive approach, partially renouncing to the originally intended linking action.

3. The Centro-Italia earthquake of 2016 and the church of S. Martino dei Gualdesi

The long earthquake sequence that hit Central Italy in 2016 was particularly severe for the numerous churches of the territory (e.g., Cescatti et al. 2017; Borri et al. 2019; Penna et al. 2019; Jain et al. 2020; Acito et al. 2023). The case examined in this work was proposed as case study by ReLUIS Task WP5.3, dealing with interventions on worship buildings listed as heritage assets within a National Research program networking university research units and the Civil Protection Agency. The purpose was twofold: comparing different modeling approaches suitable for such structures and analyzing the effect of interventions on the response to the 2016 earthquake, the latter being the focus of this work.

3.1 Seismicity and the earthquake of 2016

The church of San Martino dei Gualdesi is located in Castelsantangelo sul Nera, Marche Region, about 15 km north of Norcia, Umbria Region, in a high seismicity zone according to the national hazard map, with an expected peak ground acceleration of 0.25 to 0.35 g for a return period of 475 years.

Figure 1 shows a timeline for severe earthquakes in the region since the second half of the 20th century and known to have damaged the church. The same timeline indicates interventions of repair and improvement of the structure.

Two important events that affected the church were the earthquake of Valnerina in 1979 with Mw 5.8, and the Umbria-Marche (Colfiorito) earthquake of 1997. The latter occurred on September 26th with two strong shocks, Mw 5.7 and Mw 6, followed by a period of lower intensity events until some strong ones arrived in March and April 1998. The earthquake of 2016 was also characterized by a high extension in time. The first shock arrived on August 24 near Accumoli, with magnitude Mw 6.0, followed by a long period of activity until the ensuing Spring, with some strong events. Particularly significant for Castelsantangelo have been the events of October 26 and of October 30, the former having epicenter within the municipal area. The shock of October 30, with Mw 6.5, has been the strongest in Italy after the1980 Irpinia earthquake.

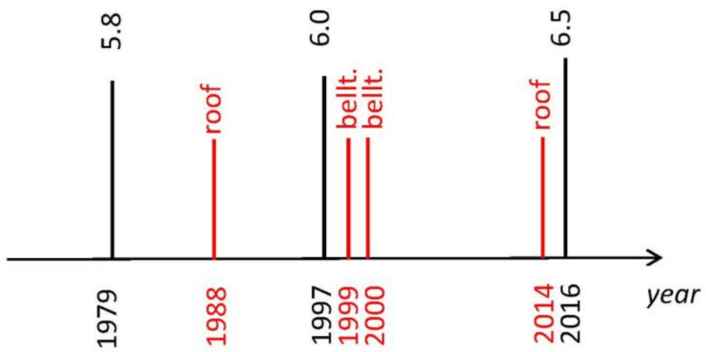

Figure 1. Timeline of severe earthquakes (in black) and of main interventions for the church (in red); the figure indicates the maximum Mw for each sequence and interventions on the roof and on the bell tower.

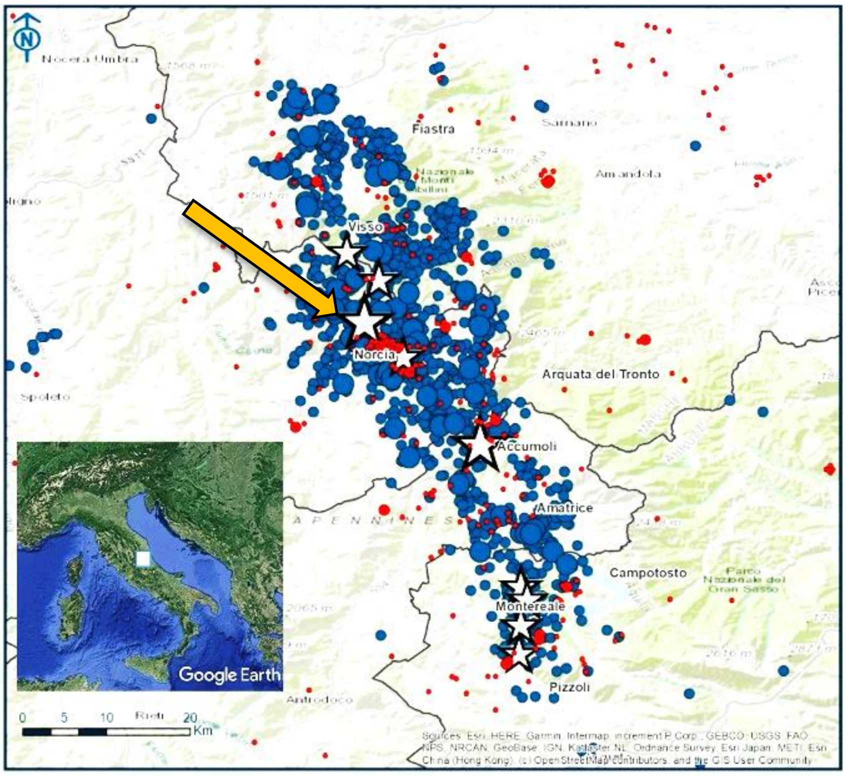

Figure 2. The 2016-17 earthquake sequence; stars indicate events with Mw> 5.0; the position of the third star from top coincides with the location of Castelsantangelo, see arrow (base picture from INGV)

The earthquake started on August 24 near Accumoli about 23 km south of Castelsantangelo with a shock of magnitude Mw 6.0, followed by others up to Mw 5.3 to the north near Norcia. From shake maps the acceleration at Castelsantangelo was 0.27 g.

The events of October 26 and 30 where the most significant for the church because they occurred at a short distance from the site. On the 26th of October two main shocks arrived with Mw 5.4 and 5.9 at 3 km and 5 km from Castelsantangelo, respectively. On October 30 a further strong event of Mw 6.5 occurred at about 7 km. A temporary recording station had been positioned in the town, at about 1 km from the church after the first shock of August 24 near Accumoli. The maximum recorded horizontal peak ground acceleration was 0.541g for October 26 and 0.447g for October 30. Other strong events occurred in the following Spring: on March 18 a Mw 6.5 was reached. The greater distance, however, attenuated the maximum acceleration at the Castelsantangelo recording station to 0.05 g.

Consequently, particularly fit records were available for the case study. Recordings were taken as usual in the North-South and East-West directions, plus the vertical one. Figure 3 reports the response spectra for the first shock of October 26, which was found to be the most consequential for the building.

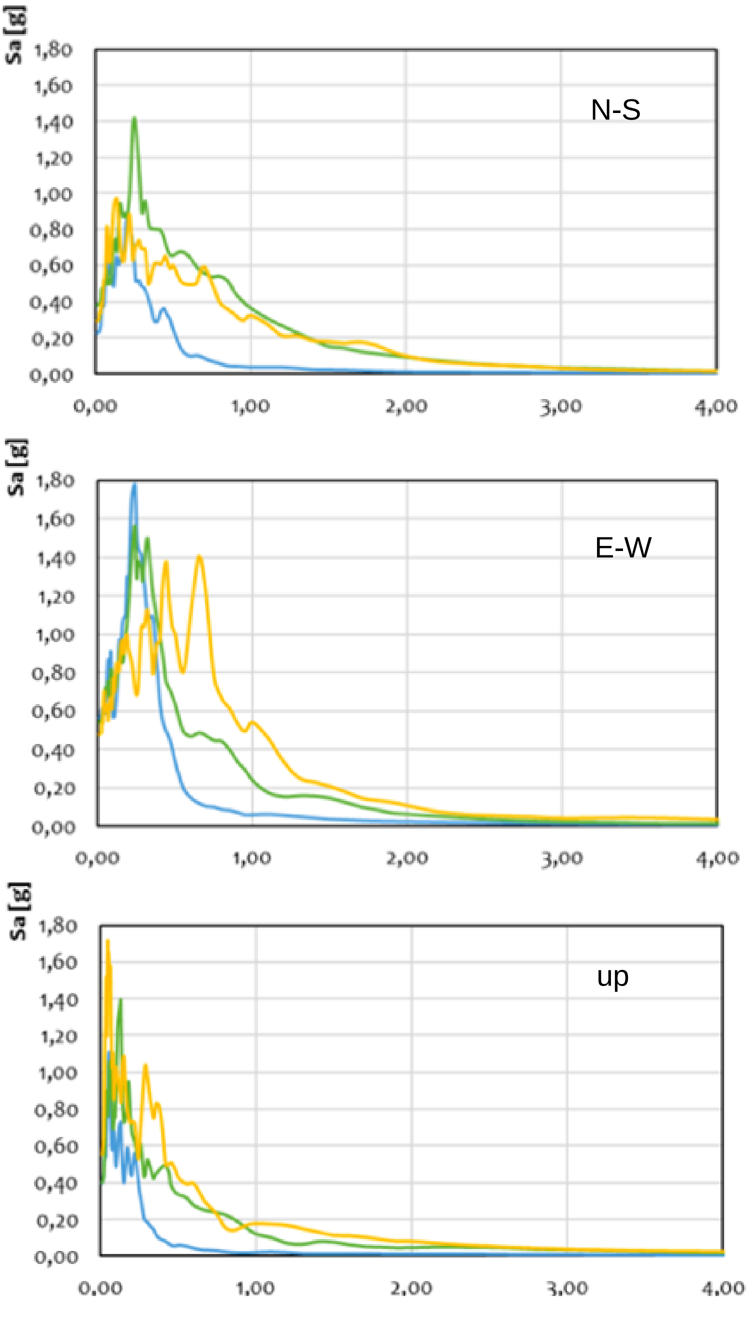

Figure 3. Pseudoacceleration response spectra for October 26 (blue for Mw 5.4, green for Mw 5.9) and October 30 (yellow, Mw 6.5)

Considering that the orientation of the church was at an angle of about 23° with respect to the East-West direction, a rotation has been applied to the records for use in the analysis. Figure 4 reports the acceleration spectrum in the longitudinal, along-nave, direction of the church. Compared to the code defined response spectrum (NTC2018) for a return period of 712 years, a reference for heritage buildings. The events of October 26 highly exceed the code spectrum values; in the orthogonal direction the difference is still evident but lower.

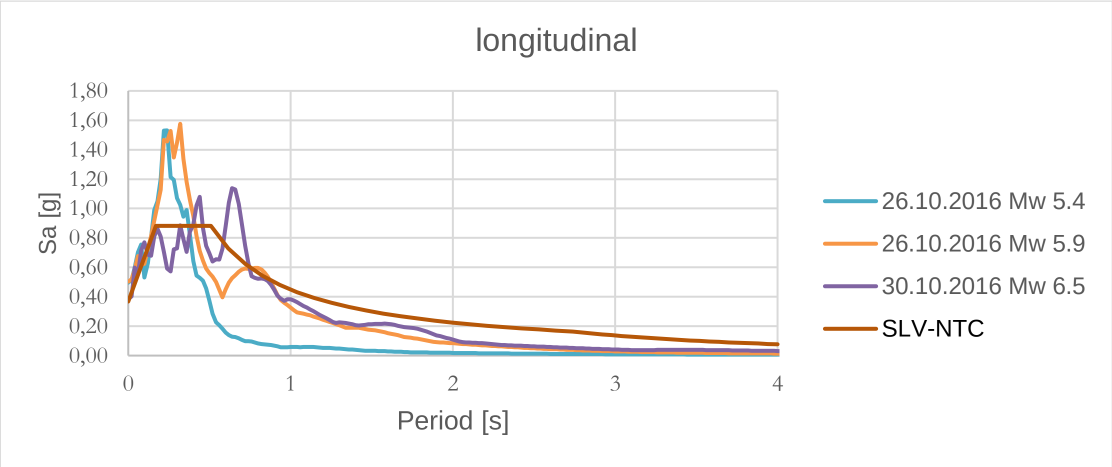

Figure 4. Pseudoacceleration response spectra compared with the code-defined spectrum.

3.2 The church

The church of San Martino dei Gualdesi was built in the 14th century; it has a simple, single-nave layout, with a rectangular plan of about 30.6 x 7.4 meters and an elevation of 6,9 m at the gable tip. The main façade is on the western side of the plan rectangle, while at the opposite side a bell tower 18.5 m tall is structurally connected to the apse area (fig. 5). The stone masonry walls have thicknesses from 70 to 120 centimeters.

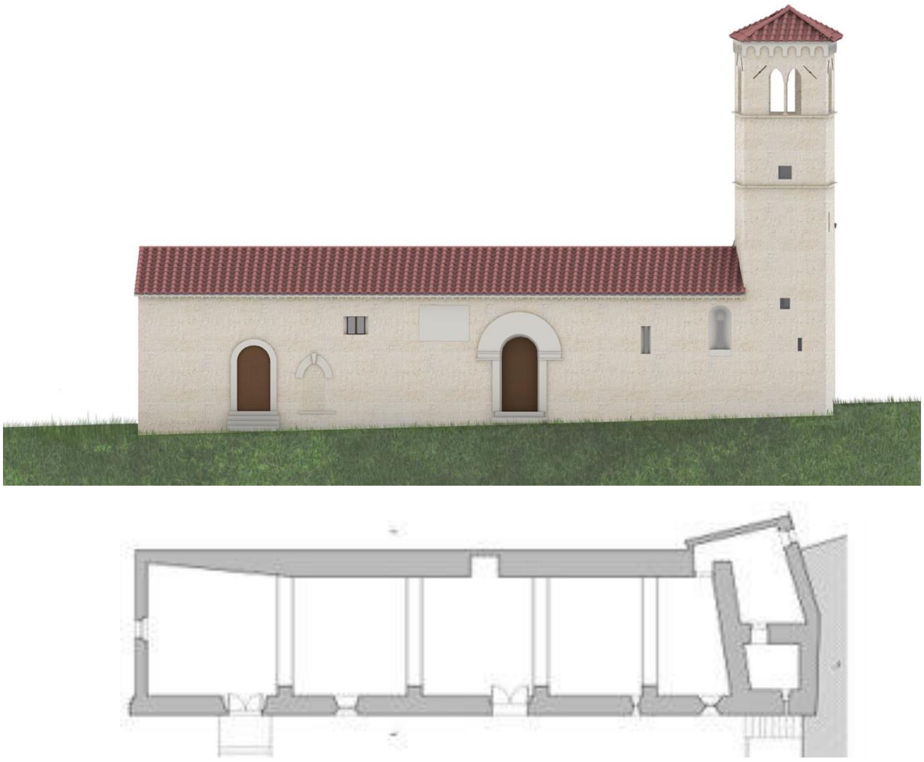

Figure 5. The church: rendering view from the side and plan

The building has a main façade on the western side and entrances laterally on the southern side. The nave is divided by a series of stone arches (fig. 6) and is covered by a timber roof structure of fairly recent fabrication, where 5 chestnut timber beams run along the nave and intersect with a series of secondary beams with brick cover. The bell tower is not aligned with the nave centerline. Data from an inspection after the seismic events of 2016 were made available, describing the main geometry and material characteristics as well as the damage occurred (Da Porto & Modena, 2020). More documentation was available describing the history of damage and interventions along time.

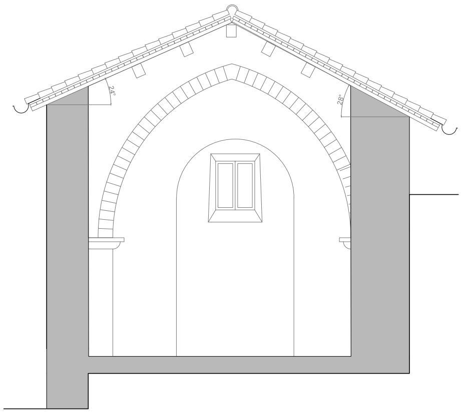

Figure 6. The church interior, toward west (Da Porto & Modena, 2020)

The church has undergone four campaigns of interventions for structural and architectural restoration after the earthquakes of 1979 and 1997. No detailed graphic documentation is available for the corresponding damage and interventions, but various documents allowed to outline their history. In 1988, as a consequence of damage occurred in 1979, bonding injections and some grouted steel-reinforcement of the nave walls and of the stone arches was performed in order to improve the strength of masonry; important works on the roof structure were also carried out. These last amounted to the substitution of the original timber structure with a stiffer lightweight concrete cover, as it was customary in central Italy at the time, together with the insertion of reinforced concrete ring beams at the top of masonry walls, connecting to the roof. In 1999, after the Umbria-Marche earthquake, interventions concerned mainly the bell tower that had been severely damaged: The masonry was repaired with local reconstruction of the cracked areas, removing damaged stones and inserting new elements; some grouted reinforcement was locally added; additionally, hoops were positioned to encircle the tower at different elevations.

In the year 2000, a new intervention was performed with a double stainless steel hooping at the tower top in correspondence of the belfry. The masonry of the nave was further consolidated, the connection of arches and nave walls strengthened with regularly spaced grouted reinforcement, and the roof structure was once more substituted using the timber solution that is present nowadays. A steel ring beam was introduced at the top of the walls to connect the roof and provide linking.

The arrival of the 2016 sequence, thus, found the presence of several different interventions on masonry walls, on arches and on the bell tower. Damage occurred during the 2016 sequence, consequent to the high level of acceleration reached, extended to different parts of the church, primarily the internal arches, the perimeter walls and the bell tower. Cracks were deep, but without loss of stability, contained by the interventions previously performed. The bell tower was cracked as in figure 7.

While damage from the 2016 events has been well documented, no detailed information is available about damage occurred in previous earthquakes. The question arises about the effect of the roof interventions that have been performed. Similar situations of roof interventions and subsequent modifications have been documented in other church buildings. The medieval church of S. Salvatore in Acquapagana, Marche, underwent the same series of earthquakes and roof interventions of San Martino dei Gualdesi. The single-nave church, which has a layout similar to San Martino with a series of arches, a bell tower connected asymmetrically to the apse and originally a timber roof, was also damaged in the 2016 earthquake series. The seismic behaviour under the 1997 and 2016 earthquakes has been analysed in Sferrazza Papa & Silva 2018 and Sferrazza Papa et al. 2021, focusing on the effects of interventions that had been performed.

The interest in the current case study is to point out the effect of the two different roof solutions, heavier and lighter, on the adjacent bell tower, having as a base the damage occurred in 2016 and the corresponding ground motion records. The numerical study developed to this purpose is described in the following sections.

Figure 7. Damage to the bell tower: west and east side (Da Porto & Modena, 2020).

4. Modeling and analyses

Two main models have been developed: first, a global finite element model of the whole church, in order to study the general behavior of the building in the elastic field and to set the bases for detailing the relationship between the church and the bell tower in the 2016 event. A second, more limited model concerned the bell tower and its constraints with the main church body. Its response to the recordings was analyzed in the nonlinear range. For each of the two models two versions were developed, representing the case of reinforced roof after the 1988 interventions and of the timber roof structure in the current situation (Cerreoni, 2021).

4.1 Linear analyses

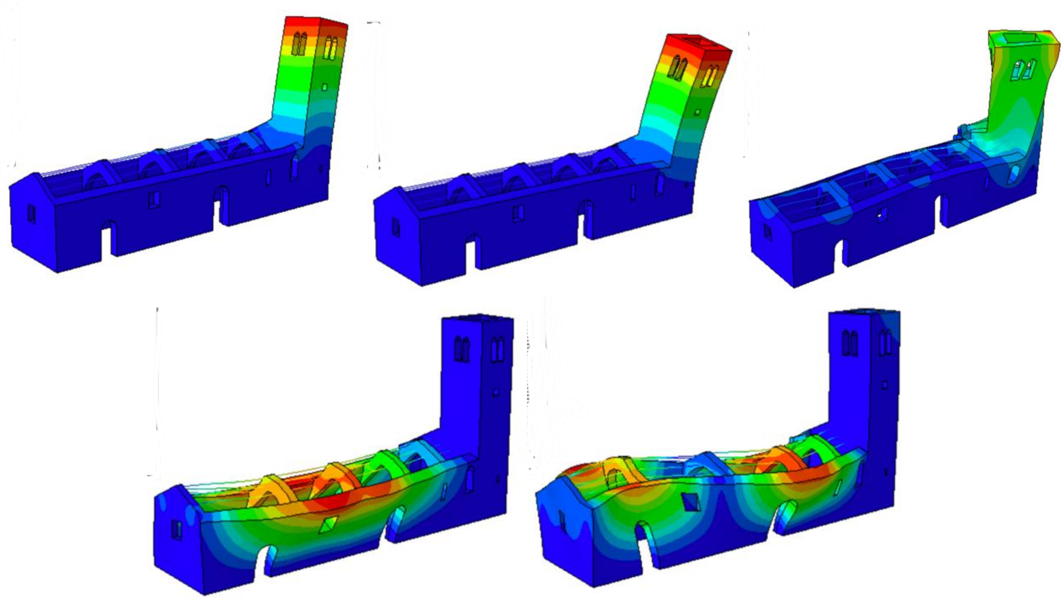

Figure 8. First five modal shapes for the case of timber roof structure; top line, from left: modes 1,2 and 5; bottom line from left, modes 3 and 4.

Table 1. Timber roof, modal values

	Mode1	Mode 2	Mode 3	Mode 4	Mode 5
Period T (s)	0.320	0.286	0.159	0.134	0.119
Mass(longitudinal) (%)	17.87	2.36	0.	0.	1.08
Mass (transversal) (%)	2.53	17.94	30.62	1.02	0.33

Figure 8 reports the first five modes for the roof structure in the current situation. The bell tower and the main body of the church have very distinct modes. The first two modes are translational for the bell tower, in the nave direction and across it respectively, while the first torsional mode is the 5th to appear. The 3rd and 4th mode concern the lateral deformation of the nave walls, which recalls a typical failure mechanism occurred in many churches. The participating masses are accordingly distributed in the main directions, and periods are those typically found in this church typology, as in Table 1.

When the massive roof cover is present, the same basic sequence of modes appears (fig. 9). The first two modes are still bell tower modes, but with a significant rotation; in the 3rd mode the stronger top link restrains the lateral deformation and involves the bell tower in the deformation. Periods are lower (Table 2).

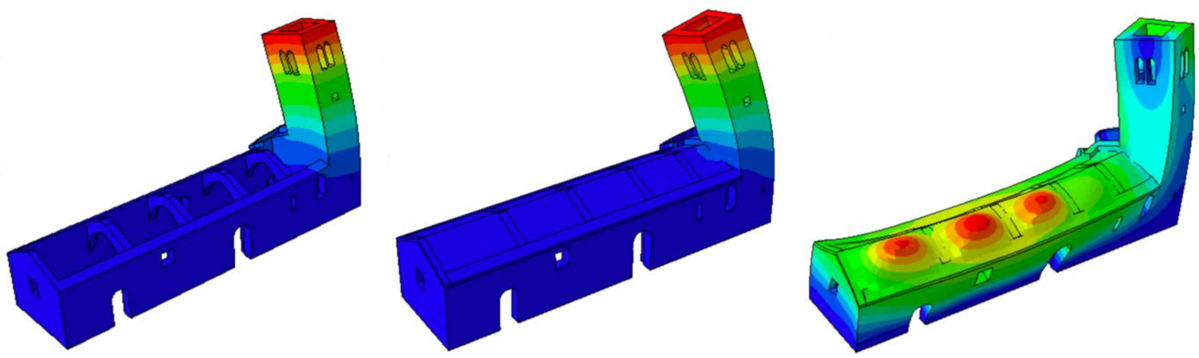

Figure 9. First three modal shapes for the case of massive roof structure; from left: modes 1,2 and 3.

Table 2. Concrete roof, modal values.

	Mode1	Mode 2	Mode 3
Period T (s)	0.293	0.271	0.129
Mass (longitudinal) (%)	9.44	8.01	0.09
Mass (transversal) (%)	10.43	12.37	39.48

The first two modes are particularly interesting to the purpose of studying further the bell tower. They practically do not involve the rest of the structure and have a significant participating mass ratio, around 18% for the first case examined. This mass fraction is reported to the entire church mass but, indeed, belongs totally to the tower. Considering that the mass of the bell tower is approximately 35% the total mass of the system, proportionally the participating mass in the two modes, when referred only to the mass of the bell tower, is about 52%, indicating that the mode represents a large fraction of the tower response.

4.2 Nonlinear analyses

To assess more in detail the relationship between the tower and the roof structure in their different configurations, nonlinear dynamic analyses have been performed. For manageability reasons, a reduced model limited to the tower with boundaries simulating the connection to the main church body has been adopted, based on observation of stiffnesses and displacements in modal analysis. Figure 10 shows restrains for the case of lighter and of heavier roof. Longitudinally, fixed hinges have been used to represent the presence of the church on the bell tower, considering the great stiffness of the main church body in such direction. The seismic motion has been applied Inserting at restraint positions the recorded value of acceleration by means of suitable boundaries. The record used corresponds to the first event of October 26, which had caused the main damage to the church. All three components of the acceleration were applied.

The figure shows also the two first modes of the bell tower model. An initial modal analysis has shown periods of 0,39 and 0,31 seconds for the lighter roof and of 0.31 and 0.29 seconds for the concrete cover case. Comparing such values with those from the full model, the concrete roof case matches values more precisely, with a 6.5 % approximation. Yet, all values appear reasonable for an indicative analysis.

The nonlinear behavior of masonry has been described with the Concrete Damage Plasticity model, CDP, (Lee & Fenves 1985; Lubliner et al. 1989), originally developed for concrete but subsequently extended and largely adopted for masonry (e.g., Acito et al. 2014; Silva et al. 2017; Valente et al. 2017, Valente & Milani, 2018, Rainone et al. 2023). In this study, parameters have been calibrated on the stone masonry of the bell tower. The CDP allows to model brittle behavior and the decay of stiffness consequent to damage.

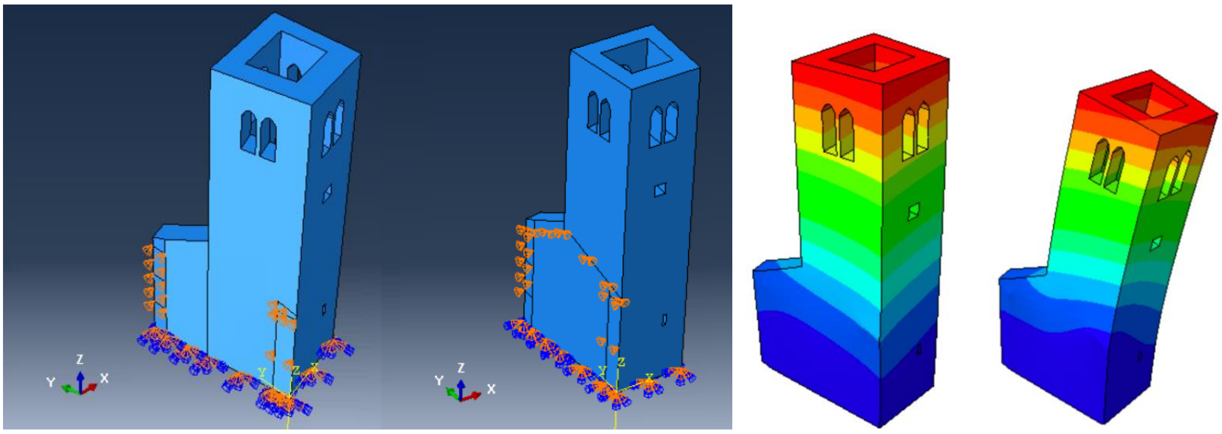

Figure 10. (left) Restraints simulating connection with the church: case of timber roof (left) and concrete cover (right); (right): first two modes of the bell tower model.

Results may be seen in the damage maps in figure 11. For the lighter roof system, model 1, damage has spread on most of the connecting wall between church and bell tower, in correspondence of the apse and at the intersection with the nave walls. Cracks propagate on the south side diagonally, reaching the opening in the wall, while on the north side a vertical crack runs along the wall separating the church and the bell tower from the presbytery. On the same side a diagonal crack extends from the base to the opening in the east wall. Results for model 2 indicate diffused damage at the connection areas between roof cover and west wall and again at the junction with the nave. In both cases, no damage appears at the bell tower top, whereas it has occurred during the earthquake. This may derive from the situation of the masonry in the bell tower, previously damaged, repaired and strengthened with reinforcement bars, which could not be detailed in this model.

For further verifying the validity of results based on the partial model, a nonlinear dynamic analysis has been performed with the same recordings for a short extension of time, 8 seconds, also on the full structure. Figure 12 shows a favorable comparison of displacements in the two cases.

The crack pattern obtained numerically and that observed for the present roof case are compatible. Yet, for a more precise assessment of effects, a quantitative measure of damage has been developed making reference to the displacement of the tower top relative to the base during motion and to the residual displacement after motion. The amount of residual displacement may be assumed as a measure of plasticization. Figures 13 and 14 report the development of displacements for model 1 (lighter roof), and model 2 (heavier roof), in the longitudinal and transversal direction. The pictures show an important difference in the two results: the lighter roof develops a milder plasticization in the bell tower, with a particularly low residual displacement in the transversal direction. The heavier roof clearly corresponds to a greater permanent damage of the structure. The result confirms the negative influence of heavy roof solutions observed in several damage situations. A similar analysis performed on the previously mentioned church of S. Salvatore, which had undergone the same history of earthquakes and a comparable series of interventions, yielded corresponding results.

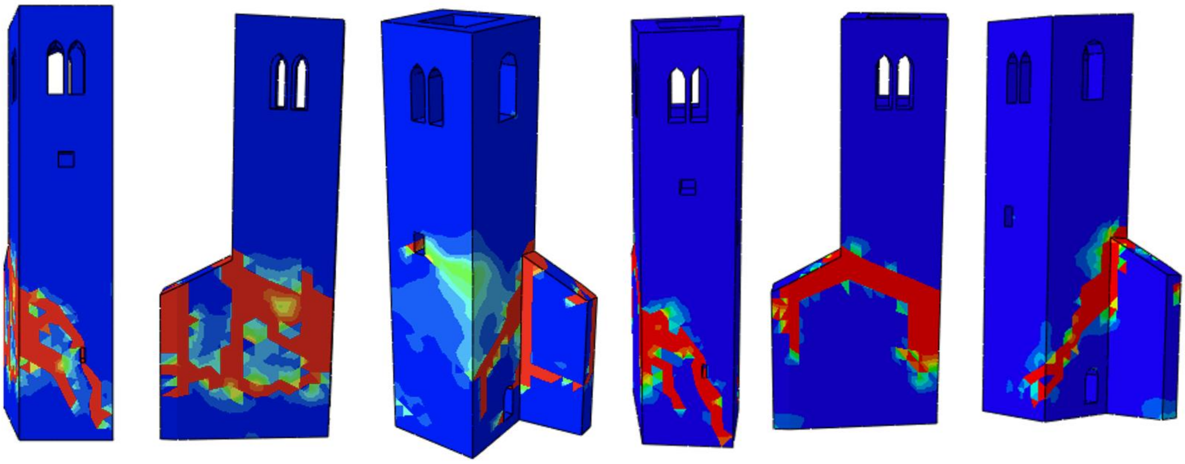

Figure 11. Damage development and distribution in model 1 (timber roof), first three images from left and model 2 (concrete slab) second three from left; for each case, South, West and North-East views are shown.

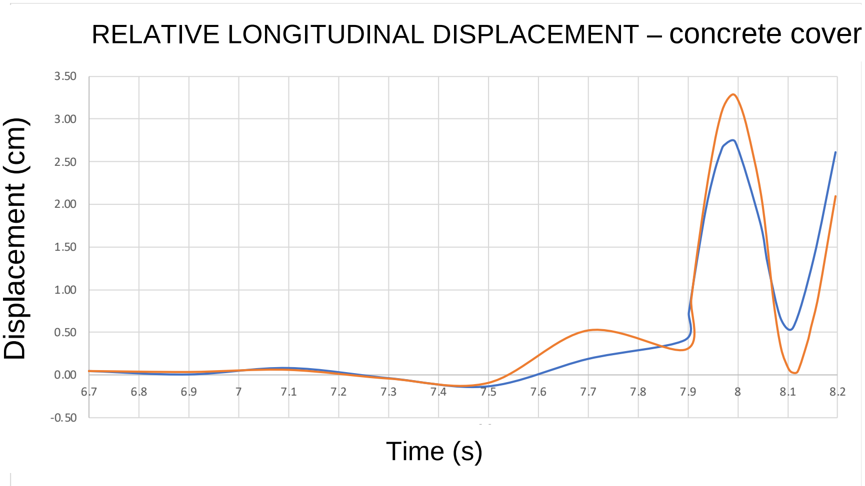

Figure 12. Displacements of the bell tower top relative to the ground in the longitudinal direction in the first 8.2 seconds of motion (partial model in red; full model in blue).

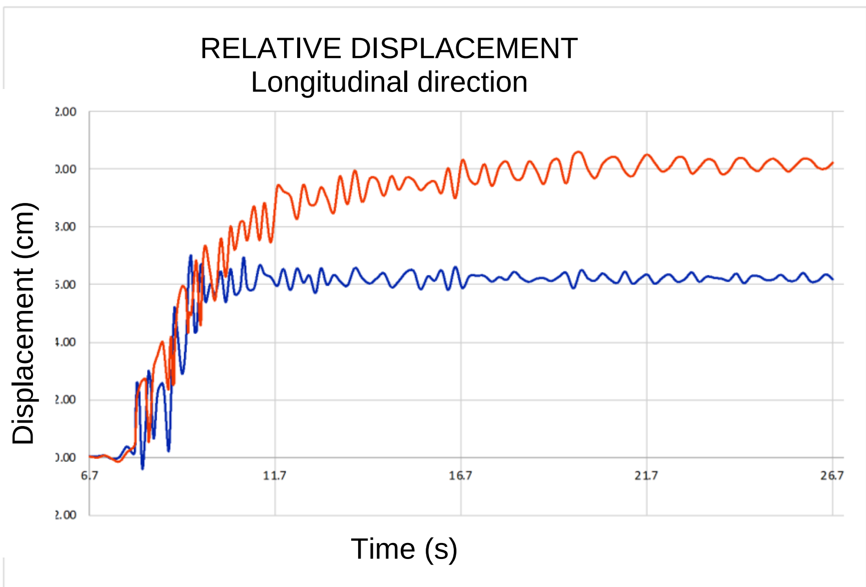

Figure 13. Longitudinal relative displacement of the bell tower top for models 1 (blue) and 2 (red)

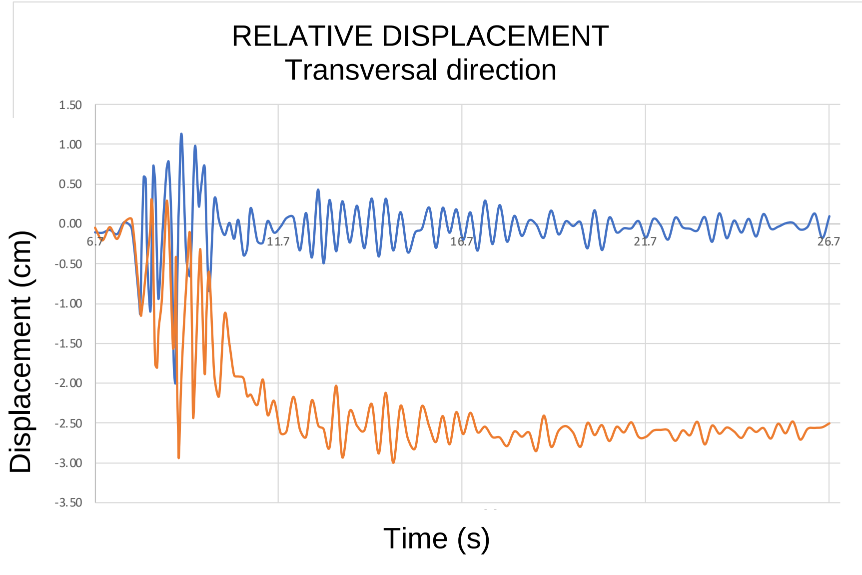

Figure 14. Transversal relative displacement of the bell tower top for models 1 (blue) and 2 (red)

5. Conclusions

Interventions devised in the last part of the 20th century to improve the seismic response of church buildings by strengthening and stiffening the roof structure with concrete elements and slabs had shown inconveniences in subsequent earthquakes, causing damage due to the increase of mass and stiffness often incompatible with weak masonry walls underneath. Subsequently, many such structures had been substituted with lighter solutions, often in timber. In the case studied, where a concrete roof slab had been substituted by a timber beam system, the 2016 earthquake caused various types of damage to the main church body and to the attached bell tower. In the study, the effect of the two roof types on the behavior and damage of the bell tower are investigated numerically, with reference to acceleration time histories recorded in a nearby station.

From modal analysis, the stiffer roof shows the positive linking effect for which it was originally conceived, restricting and regulating the lateral deformations of the nave walls, but it also modifies significantly the dynamic behavior of the adjacent bell tower. From nonlinear dynamic time history analysis, cracking was widely spread in all the bell tower walls for both roof types, yet for the case of concrete roof a strong concentration of post-elastic deformations appeared along the lines of connection between tower and roof. By comparing the top sway history and the final permanent displacement, which is a measure of the depth of penetration in the post elastic field, the higher impact of the stronger church roof on the adjoining bell tower was shown and quantified: the effect of the type of roof on the seismic response and on the damage potential not only on perimetral walls but also of adjacent elements, like the bell tower, has been put into evidence.

6. Acknowledgements

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