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Combining Architectural Conservation and Seismic Strengthening in the Wood-Based Retrofitting of a Monumental Timber Roof: The Case Study of St. Andrew's Church in Ceto, Brescia, Italy

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ABSTRACT

This work presents the application of timber-based retrofitting techniques to a case-study stone masonry church featuring a wooden roof from 18th century. From the static point of view, the original roof structure presented a number of undersized structural elements, and its members were poorly or not connected among each other and to the masonry, making the church vulnerable to seismic loads as well. Thus, the roof was retrofitted with wood-based techniques, including an overlay of plywood panels, against seismic actions. These affordable, rapid, easily realizable interventions enabled both the conservation and seismic retrofitting of the roof, providing an adequate load-carrying capacity for static loads, and an effective diaphragm action against seismic loads. The conducted numerical analyses showed that the realized interventions greatly improve the seismic behaviour of the building. Besides, when the additional energy dissipation provided by the plywood panels overlay is taken into account in the numerical model, the church would even potentially be able to fully withstand the expected seismic action of the site.

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Architectural conservation; masonry buildings; plywood panels; seismic retrofitting; timber floors

1. Introduction

Historical and monumental buildings that are part of the architectural heritage of several countries, often feature masonry walls as vertical loadbearing structural elements, and timber floors or roofs as horizontal components. Taking into consideration the Italian heritage-building stock, a large part of it consists of masonry churches, thus several research studies investigated their seismic vulnerability and the impact that earthquakes can have on these specific structures. Post-event observations and evaluations have been conducted after the main earthquakes occurred in Italy in the last decades, such as 1976 Friuli Earthquake (Doglioni et al. 1994), 1997 Umbria-Marche Earthquakes (Lagomarsino and Podestà 2004b, 2004c), 2002 Molise Earthquake (Lagomarsino and Podestà 2004a), 2003 Valle Scrivia Earthquake (Ruggieri et al. 2022), 2009 L'Aquila Earthquake sequences (da Porto et al. 2012; De Matteis, Brando, and Corlito 2019; De Matteis, Criber, and Brando 2016; Lagomarsino 2012), 2012 Emilia Earthquake (Indirli, Marghella, and Marzo 2012; Taffarel et al. 2016), 2016–17 Central Italy Earthquake sequences (Canuti et al. 2019; Cescatti et al. 2020; De Matteis and Zizi 2019; Penna et al. 2019), and 2017 Ischia Island Earthquake (Salzano et al. 2020). In light of these seismic events, vulnerability analyses and evaluation of collapse mechanisms have been carried out, focusing on either single monuments (e.g. Boscato et al. 2014; Brando, Criber, and De Matteis 2013, 2015; Criber, Brando, and De Matteis 2015; Sorrentino et al. 2009), or large samples of masonry churches (e.g. 90 single-nave churches in Piemonte in Ruggieri et al. 2022, 64 three-nave churches in Abruzzo in De Matteis, Criber, and Brando 2016).

Damage surveys on churches are performed by identifying potential collapse mechanisms involving single portions of the buildingfor instance, façade, nave, transept, dome, bell tower, and so forth. This methodology was first introduced in Doglioni et al. (1994) and formed the basis for specific guidelines for masonry churches (Doglioni 2000). The approach was then refined in Lagomarsino and Podestà (2004b, 2004c), with the definition of 28 collapse mechanisms to which the most recurrent damage failures can be associated. The outcomes from these studies were then implemented in the

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Italian Guidelines (Ministero per i Beni e le Attività Culturali 2011) with regard to the seismic risk assessment of architectural heritage and included in the post-earthquake management procedures by Ministry of Cultural Heritage and Activities (MiBACT guidelines 2015), and Italian Department of Civil Protection. Examples of investigations on such collapse mechanisms can be found in (e.g. Lagomarsino 1998; Brando, Criber, and De Matteis 2015; Casolo 2017; Casolo et al. 2000; Casolo and Uva 2013; Criber, Brando, and De Matteis 2015; D'ayala and Speranza 2003). These structural failures have often been caused by the poor characteristics of masonry walls, the lack of adequate connections among vertical and horizontal structural components, as well as the flexibility and insufficient capability of as-built timber floors to transfer and redistribute seismic loads. Hence, the improvement of these characteristics is crucial for preserving such monumental constructions and the architectural heritage in general, by preventing as much as possible the structural damage caused by earthquakes.

However, when designing seismic retrofitting interventions on existing churches, their historical and architectural values has also to be considered and preserved. Hence, seismic strengthening solutions have to be designed in such a way that they are reversible, not invasive, and enable the conservation of the buildings. Yet, inappropriate modifications and structural interventions, such as the substitution of timber diaphragms with concrete slabs, have frequently been applied to the existing (wooden) roof structures of masonry churches, even worsening their seismic response (Binda, Chesi, and Parisi 2010; Lagomarsino 2012; Tobriner, Comerio, and Green 1997). In general, churches with extended interventions that abnormally increased their local mass and stiffness, often reported severe damage, whose location, pattern, and extent can be greatly influenced by the presence of a rigid or deformable roof (Sferrazza Papa et al. 2021).

In this context, recent investigations highlighted the potential of timber-based seismic strengthening techniques for existing masonry buildings (Gubana 2015; Mirra and Ravenshorst 2021; Mirra et al. 2022; Mirra, Ravenshorst, and van de Kuilen 2021b; Pozza et al. 2021). More specifically, with reference to the improvement of the in-plane response of timber diaphragms, experimental studies on wood-based retrofitting techniques, such as the overlay of planks (Corradi et al. 2006; Piazza, Baldessari, and Tomasi 2008; Valluzzi et al. 2010), cross-laminated timber panels (Branco, Kekeliak, and Lourenço 2015), oriented strand board panels (Gubana and Melotto 2018), or plywood panels (Brignola, Pampanin, and Podestà 2012; Giongo et al. 2013; Mirra, Ravenshorst, and van de Kuilen 2020; Peralta, Bracci, and Hueste 2004; Wilson, Quenneville, and Ingham 2014), demonstrated the excellent performance and high potential of these strengthening methods. In particular, an overlay of plywood panels fastened around their perimeter to the existing sheathing can greatly increase not only the in-plane strength and stiffness of a wooden floor, but also its energy dissipation, providing additional benefits for the whole masonry building (Mirra, Ravenshorst, and van de Kuilen 2021a, 2021c, 2021d; Mirra and Ravenshorst 2022).

This work presents the application of the aforementioned concepts to a case-study building, namely St. Andrew's Church (Figure 1) in Ceto (Province of Brescia, Italy). This existing stone masonry construction features a monumental timber roof, which has been retrofitted by means of wood-based seismic strengthening techniques. The main objective of this work is to highlight, aided by a specific case study, the advantages of timber-based solutions for both seismic improvement and architectural conservation. These benefits are not only presented from an academic perspective, but also from the point of view of professional engineering, in order to contribute to the framework supporting both further research on wood-based seismic retrofitting techniques, and their actual, practical application on monumental buildings. Hence, the present article starts from a frequent scenario in engineering practice, where both the allocated budget and the execution time are limited. This led to conservative design and modelling assumptions, since only visual inspections on the existing structure were conducted, and a more in-depth structural assessment was not possible. Yet, even in such situation, the adopted wood-based techniques allowed to meet the requirements in terms of realization time and cost, and to provide the church with additional energy dissipation sources, as will be presented in the following.

Firstly, a detailed description of the church and its vulnerabilities is provided in Section 2. Subsequently, Section 3 presents the applied reversible, wood-based retrofitting interventions and their design. In order to assess the impact of these strengthening solutions on the global seismic performance of the building, two numerical models were constructed (Section 4). The first model, created by the engineering firm responsible for the design, was realized in the commercial software Aedes.PCM (AEDES 2022): in this model, besides evaluating local collapse mechanisms in the as-built state, the improvement in seismic capacity after the retrofitting was quantified with pushover analyses, in agreement with the Italian Building Code (NTC 2018). In the second model, realized in DIANA FEA (Ferreira

Figure 1. (a) Location of the analysed church in the province of Brescia; (b) position of the church in the town of Ceto; (c) case-study building.

2020) by Delft University of Technology, nonlinear incremental dynamic analyses (IDA) were conducted. In this case, the targets were the assessment and validation of the outcomes of the first model from engineering practice, and the investigation of the dissipative contribution of the roof retrofitted with plywood panels. The results from the analyses are discussed and compared in Section 5, focusing also in detail on the advantages and benefits of the use of wood-based techniques in this case study. Finally, in Section 6 the main conclusions and recommendations for future works are summarized.

2. Case-study church

2.1. Brief historical background and description of main architectural features

The Church of St. Andrew (Figure 1) in Ceto (Province of Brescia, Italy) was built between 1708 and 1732 on the location of a pre-existing 11th century chapel, and consecrated on 1st September 1743. According to the detailed information available in the local parish archives, the construction of the masonry structure was realized with local stones, and was completed in 1713, including the timber roof. From 1713 to 1726, mainly finishing works took place, enriching the church with altars, statues, frescos and some valuable paintings, realized by, among others, two very active and respected North-Italian masters of the time, Giacomo Ceruti and Bartolomeo Litterini. These finishing works provided the church with the present sober, but valuable appearance from both the artistic and the architectural point of view (Figure 2). The late-baroque façade features a stringcourse separating it into two portions, both embellished with pilasters; in the lower portion the entrance is present, surmounted by a semi-circular tympanum, while the upper one has a rectangular window and a curved pediment with acroteria. The 28-m-high bell tower is adjacent to the church but separated from its lateral buttresses and is made of granite blocks. The interior of the church has surfaces decorated in relief, wall paintings, and frescos, and consists of a single nave with four side chapels. The walls are marked by pilasters supporting a projecting cornice, above which a barrel vault with lunettes is found, having four openings in correspondence to the side chapels.

2.2. Description of the structure

The case-study church is a relatively compact stone masonry building (Figure 3a,b), consisting of a single nave measuring 21 × 14 m, with an average height of 14.5 m, and covered with a barrel vault. The apse, where the presbytery and choir are located, is covered with a cross vault and separated from the hall by a triumphal arch, and has dimensions of 8.2 × 8.2 m, with an average height of 13 m. The building rests on

Figure 2. (a) View of the interior of the church; (b) particular of the frescoed triumphal arch; (c) high altar with polychrome marbles; (d) original walnut choir; (e) organ from 1779.

sloping rocky ground and is located on top of a retaining wall (Figure 1).

With regard to the main structural components of the church, all walls are composed of stone masonry (Figure 3c), reaching an average thickness of about 220 cm in the location of the buttresses, which also feature existing metal ties at the vault height (Figure 3d). Elsewhere, the thickness of the walls varies from 70 cm to 100 cm; both vaults are 10–15 cm thick. From the conducted in-situ visual inspections, the structure appeared to be well realized, but the limited budget available (see Section 3) did not allow more detailed investigations on materials and structural components.

The original monumental roof (Figures 3–4), dated back to the beginning of 18th century, entirely consists of wooden structural elements, arranged as follows:

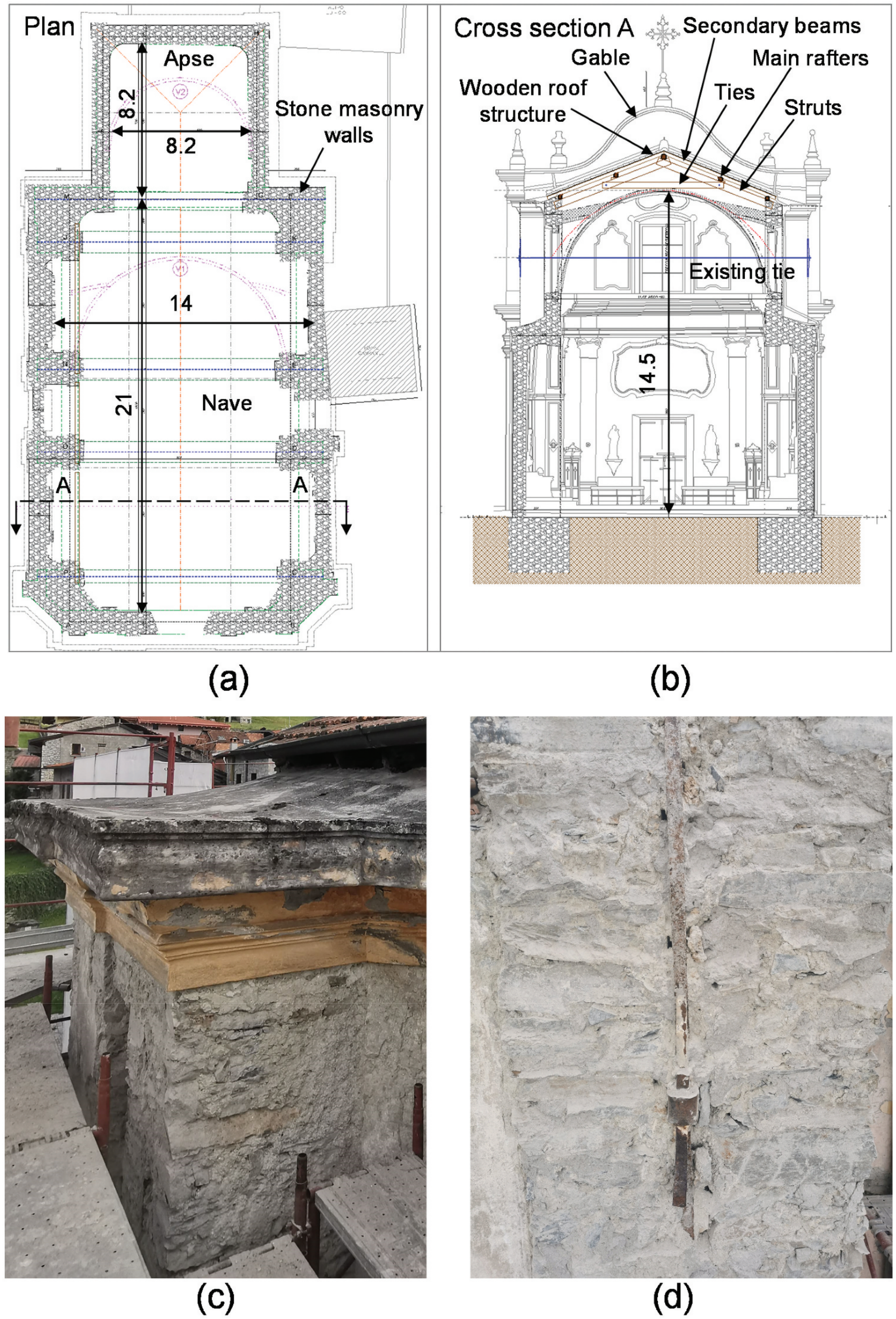

Figure 3. Plan (a) and cross section (b) of the case-study church (dimensions in m); particulars of a stone masonry wall (c) and of an existing metal tie (d).

- 38×38 cm struts, resting on the masonry buttresses, and connected by two 20×20 cm wooden ties fastened to the struts by means of a metal pin. Only in the location of the apse, these struts measure 32×32 cm.
- Primary 26×26 cm longitudinal rafters, and a 32×32 cm ridge beam; at the supports a 24×24 cm wooden wall plate is also present, resting for 75% of its thickness on the masonry walls. For the roof portion covering the apse, 24×24 cm rafters and a 28×28 cm ridge beam are present.
- Secondary 16×16 cm (15×15 cm above the apse) joists at 80 cm spacing.

Figure 4. (a) Existing wooden roof structure; (b) existing metal pin between timber struts and ties; (c) ridge beam undergoing excessive deflection.

- Existing sheathing, having a thickness of 20 mm, surfaced with traditional Marseille roof tiles.

All structural elements of the roof are made of spruce (*Picea abies*), with the exception of the wooden struts and ties, made of larch (*Larix decidua*). Also for the case of the roof structure, it was not possible to conduct more detailed investigations than visual inspections due to budget limitations.

2.3. Background of the retrofitting interventions and main vulnerabilities of the building

Given the relevance of the church for the local territory, as well as its historical and architectural value, a restoration intervention was commissioned by the Curia of Brescia. The amount allocated for the execution of the whole restoration work, excluding technical expenses and taxes, was 280 000 €, out of which 50 000 € for structural and seismic upgrading. The main reason behind this conservation intervention was the bad condition of the plasters of the facades, which were detached in several locations and initially led to concerns about structural problems of the church as well. After several inspections, it was ascertained that the observable cracks and detachments of material only involved the finishing layers and had been caused by the chemical and thermo-hygrometric incompatibility between stone masonry and cement plaster, here improperly applied in past restoration works. On the contrary, from the conducted visual inspections, the underlying masonry structural elements appeared to be well dimensioned and constructed, and also the existing metal ties were in good state and well restrained to the walls (see again Figure 3d). Yet, even if globally the church did not present structural issues, some local vulnerabilities from both static and seismic point of view were identified.

The wooden roof structure was found in fair state of conservation but was very difficult to inspect, as the attic of the nave and the apse is narrow and normally not accessible. The main rafters, including the ridge beam (Figure 4c), appeared to be very undersized, also considering the spans involved, up to 8 m: large deflections could be observed, which had been compensated over time by additional wooden blocks, to keep the support for the secondary beams as horizontal as possible. These secondary elements seemed better dimensioned and sound, but a small number of joists (approximately 15% of the total) were locally affected by biological degradation, caused by slight water infiltrations. The main wooden struts and ties were sound as well, and according to the static calculations performed, had adequate dimensions for their structural purpose. Yet, the existing connections with metal pins (Figure 4b) were not effective, because the holes in which they were located had become too large over time: thus, the ties could not fully absorb the horizontal thrusts induced by the struts, which seemed to be partly taken by the masonry buttresses.

The presence of these additional horizontal thrusts could be particularly critical in the event of an earthquake, also considering the absence of effective joints among all structural elements of the roof. Besides, connections between the roof itself and the walls or buttresses were also absent, thus the shear forces induced by an earthquake could not be transferred properly. Furthermore, although the analysed church is a compact building with thick walls, the aforementioned limited budget for the retrofitting interventions did not allow for detailed investigations on the actual quality or homogeneity of the stone masonry, as well as of the mutual connections among walls, buttresses and façade.

Hence, for the seismic assessment of the church, considering that the wooden roof structure could not act as a diaphragm absorbing the seismic actions and redistributing them to the masonry walls, it was conservatively assumed that in the event of an earthquake the building would not be able to develop a proper box-like behaviour and would exhibit local (out-of-plane) collapses of masonry structural elements.

3. Seismic retrofitting and conservation of the wooden roof structure

The applied retrofitting methods were designed in the framework of the *seismic improvement* intervention (pursuant to § 8.4.2 of the Italian Building Code, NTC 2018). According to the standard, a quantity ζ_E is defined, representing the ratio between the seismic action that can be withstood by the existing structure, and the seismic action that would be considered in the design of a new building in the same site. In terms of peak ground acceleration (PGA), this quantity can be defined as $PGA_{capacity}/PGA_{demand}$. After having evaluated $\zeta_{E,1}$ for the as-built state and $\zeta_{E,2}$ for the building after retrofitting, the requirement of seismic improvement is met when the designed strengthening solutions ensure that $\zeta_{E,2} - \zeta_{E,1} \geq 0.1$ (NTC 2018), thus $PGA_{capacity,2} \geq 0.1 \times PGA_{demand} + PGA_{capacity,1}$. For the location of Ceto, PGA_{demand} was determined considering soil type A, topographic amplification factor of 1.2 (accounting for the sloping ground), and importance class III, given the historical relevance of the building. This corresponded to $PGA_{demand} = 0.08$ g for the no-collapse limit state (EN 1998:2004).

In order to meet the requirement of seismic improvement, retrofitting interventions were applied to the structural element responsible for most vulnerabilities, i.e. the wooden roof, seizing also the opportunity to combine both strengthening and architectural conservation. Taking into consideration the improvement of seismic shear transfer and redistribution of the existing roof, the main retrofitting intervention consisted of transforming it in a diaphragm. To this end, an overlay of 30-mm-thick C24 structural plywood panels fastened to the existing sheathing with 4 × 60 mm Anker nails at 80 mm spacing, was realized (Figure 5). The plywood panels overlay was designed considering each roof pitch as a shear wall. Since the panels have a standard width of $b_i = 1200$ mm, 15 nails were present on each panel's short side, following the adopted spacing $s = 80$ mm. The long side of the roof above the barrel vault featured 16 rows of panels (see Figure 5), thus the total strength was derived as the design shear strength $F_{v,Rd}$ of a single nail, multiplied by the number of nails in the panel's width (equal to b_i/s) and by the number n of rows of panels. For this case, $F_{v,Rd} = 1.15$ kN, $b_i/s = 15$, and $n = 16$, thus the total strength of a roof pitch is 1.15·15·16 = 276 kN. During a seismic event with the design PGA of 0.08 g, this value would enable the roof to effectively transfer the shear loads of 200 kN induced by the seismic weights of the gables.

The applied, reversible solution enables the development of the box behaviour of the construction, but without significantly changing the stiffness of the entire building (Mirra and Ravenshorst 2021). Besides, this type of diaphragm can also potentially act as a dissipative element, absorbing part of the energy imparted by the earthquake by means of the yielding of the fasteners connecting planks and plywood panels (Mirra, Ravenshorst, and van de Kuilen 2021a). The influence of this energy dissipation capacity on the present case study is quantified through the time-history analyses conducted in the second numerical model (Section 4).

On top of the plywood panels overlay, as an extra safety measure, an additional light bracing system consisting of 5 × 80 mm S275 steel plates was designed, to promptly prevent local overturning mechanisms of the gable and the other walls, by directly transferring the pertaining shear forces to the buttresses (Figure 5). Besides, to enable an effective and distributed transfer of shear and tensile stresses, an adequate connection system between the new diaphragm and the wall had to be designed. Because the existing joists at the supports of the roof structure rested on the masonry for only 75% of their thickness (Section 2.2, Figures 6–7), the realization of anchors directly crossing them would not have been fully effective in preventing overturning mechanisms. Therefore, along the perimeter of the church, in correspondence to the eaves channels, a steel curb was realized in the position of the centre of mass of the wall (Figures 6–7). To allow the transfer of the shear and tensile forces between the plywood panels overlay and the curb, a connection with steel plates was created, in combination with a cable bracing system (Figure 6). The steel curb was then connected to the walls and the buttresses with M20 anchors (80–120 mm depth). The global seismic retrofitting intervention was particularly appreciated by the Superintendence for Architectural Heritage, since it allowed to avoid more invasive solutions and preserve at most the original wooden roof structure.

Considering now the interventions more linked to the architectural conservation of the roof, mostly timber-based solutions were once more adopted.

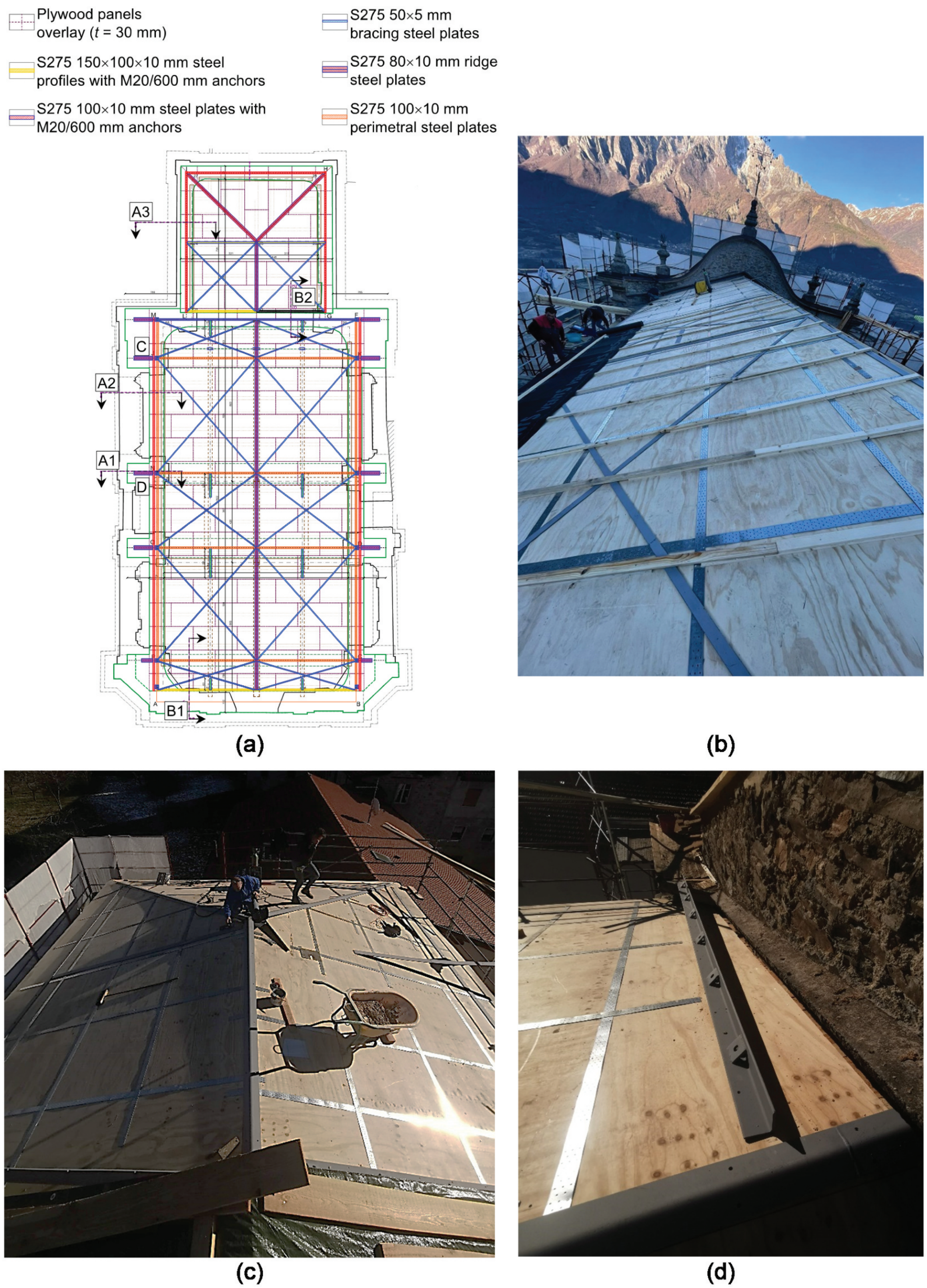

Figure 5. (a) Overview of the seismic retrofitting interventions; plywood panels overlay above the nave (b) and the apse (c); steel profile for the connection to masonry (d).

The few joists and planks damaged by water infiltrations were integrated with newly supplied C24 joists, laid alongside the existing ones, and featuring their same geometry and wood species. Besides, the longitudinal rafters, which appeared to be very undersized, were provided with an adequate flexural reinforcement, by placing additional C24 wooden beams clamped to the main ones by means of steel plates and threaded bars (Figure 8). These newly supplied beams had again the same geometrical

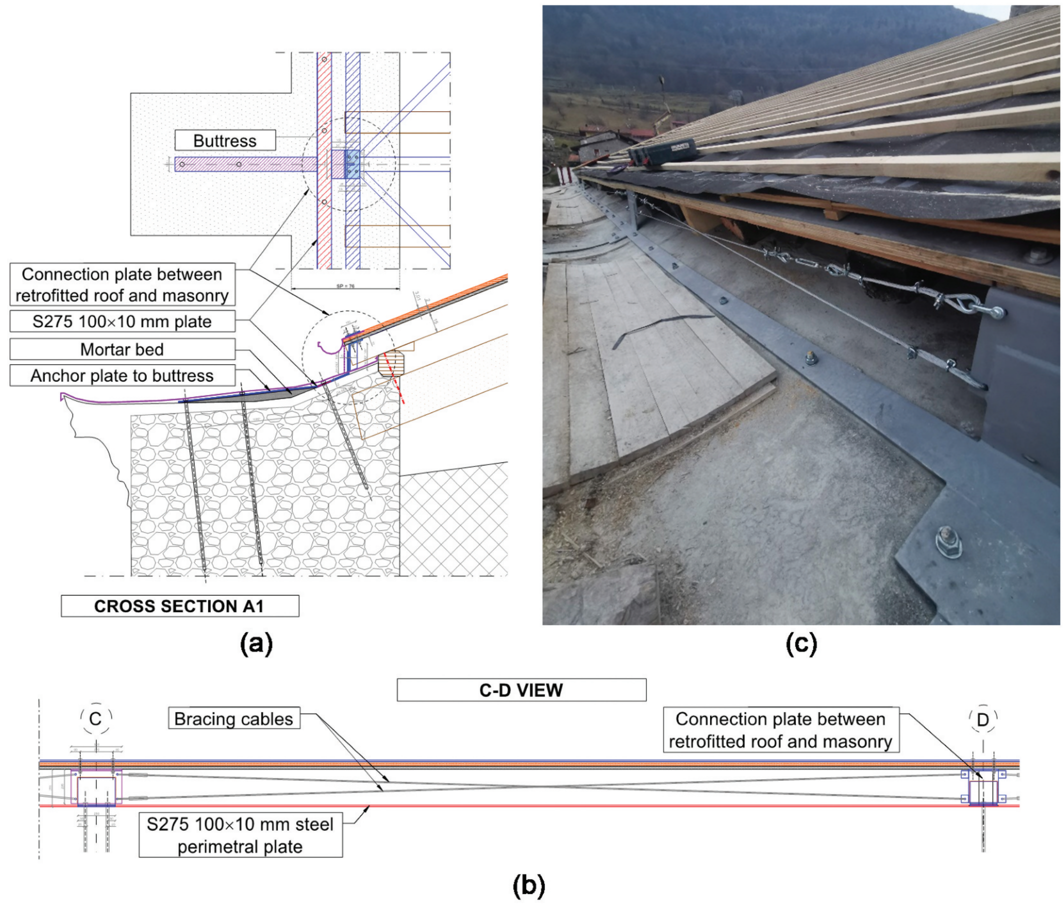

Figure 6. Connection between retrofitted roof diaphragm and masonry walls, with reference to the plan of figure 5: (a) cross section of the joint system in correspondence of the buttress; (b) system of bracing cable between buttresses; (c) realization of the intervention on site.

characteristics and wood species as the existing ones. Such interventions are reversible and compatible with the as-built roof and could be conveniently realized without removing the existing structural elements.

Additionally, screwed joints were created between all secondary joists and the main loadbearing beams, to avoid the possible loss of support due to the vertical seismic component. Likewise, connections between main beams and wooden struts, as well as between the struts themselves and the timber ties, were realized (Figure 8b). In this way, besides improving the structural in- and out-of-plane response of the whole roof, it was possible to create an adequate contrast to the aforementioned thrusts induced by the struts, thus beneficially removing this out-of-plane action on the masonry walls.

The static and seismic retrofitting measures, as well as the whole restoration work on the church, were concluded in the second half of 2022: the building after these interventions is shown in Figure 9.

4. Numerical models

4.1. First numerical model: seismic assessment of the church before and after retrofitting

The first numerical model of the case-study church (Figure 10a) was created by the engineering firm responsible for the interventions in the commercial software Aedes.PCM (AEDES 2022), aimed at professional engineers. In this software, the global seismic response of the masonry structure is assessed adopting an equivalent-frame modelling strategy (Quagliarini, Maracchini, and Clementi 2017), whereas (local) out-of-plane collapses are evaluated with limit analysis. For all walls, the properties for stone masonry suggested by the Italian Building Code (NTC 2018), in the most conservative case of limited knowledge and no available investigations on materials and structural components, were adopted, and are reported in Table 1. Given the overall good state of timber structural members, a C24 strength class (EN 338:2016) was assumed for them in this model, thus the same of the new structural members,

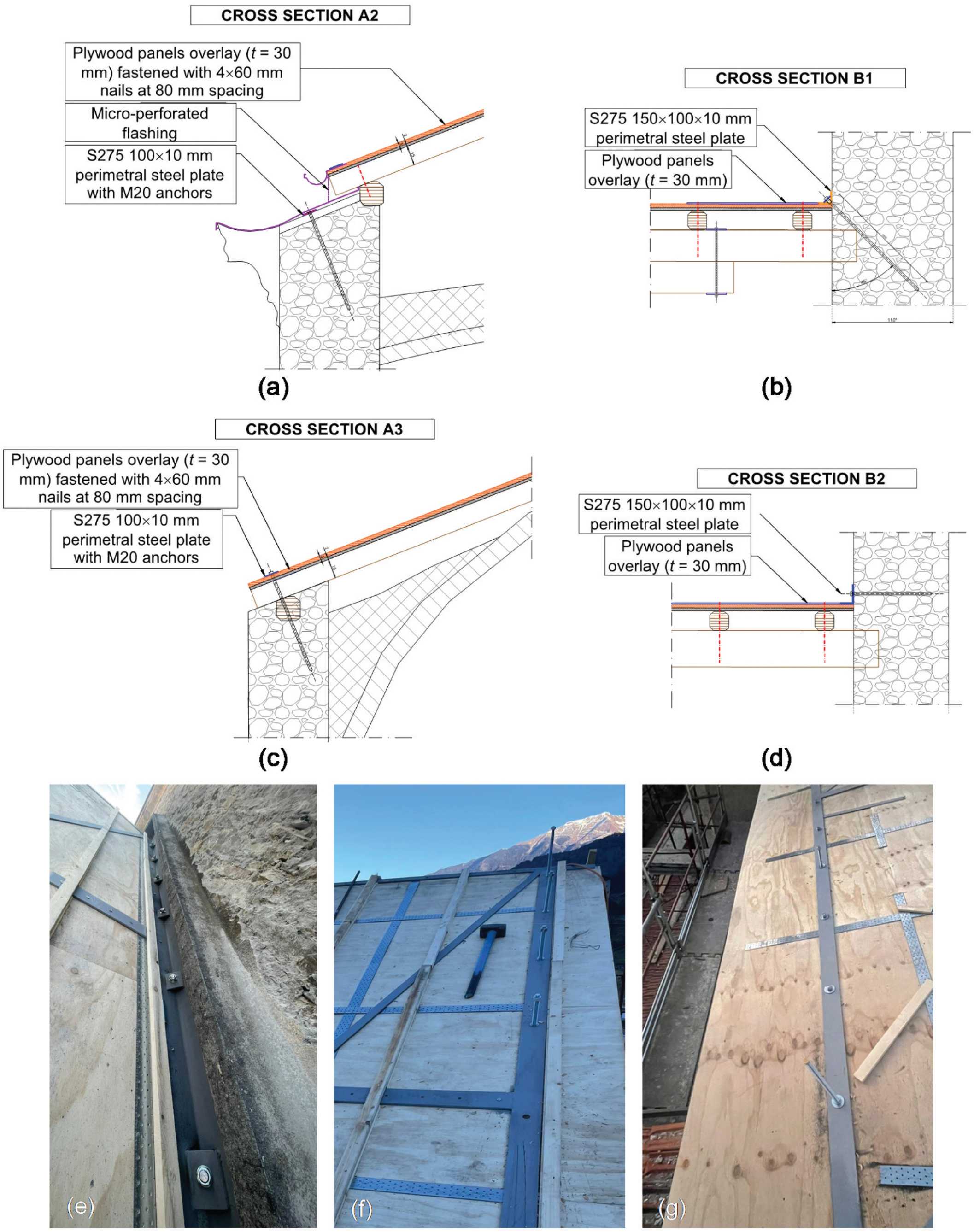

Figure 7. Connection between retrofitted roof diaphragm and masonry walls, with reference to the plan of figure 5: (a) cross section of connection system along the nave walls; (b) cross section of connection system in the main façade; (c) cross section of connection system along the apse walls; (d) cross section of connection system in the wall separating nave and apse; (e-g) examples of realized joints.

including plywood panels. It should be noted that more variability in timber material properties of the existing components could be expected in the case of historical buildings, but due to time constraints more in-depth analyses could not be conducted. In any case, since in the as-built state local collapse mechanisms of the church were evaluated, and in the post-retrofitting assessment the roof was considered as rigid, it was expected that the material properties of masonry walls, rather than timber elements, had the largest influence on the results. Hence, the most conservative values from the Italian Building Code were selected, coherently with the lack of detailed knowledge of the actual material properties. The seismic action for the location of Ceto was prescribed, with an on-site design PGA of 0.08 g, as previously mentioned in Section 3.

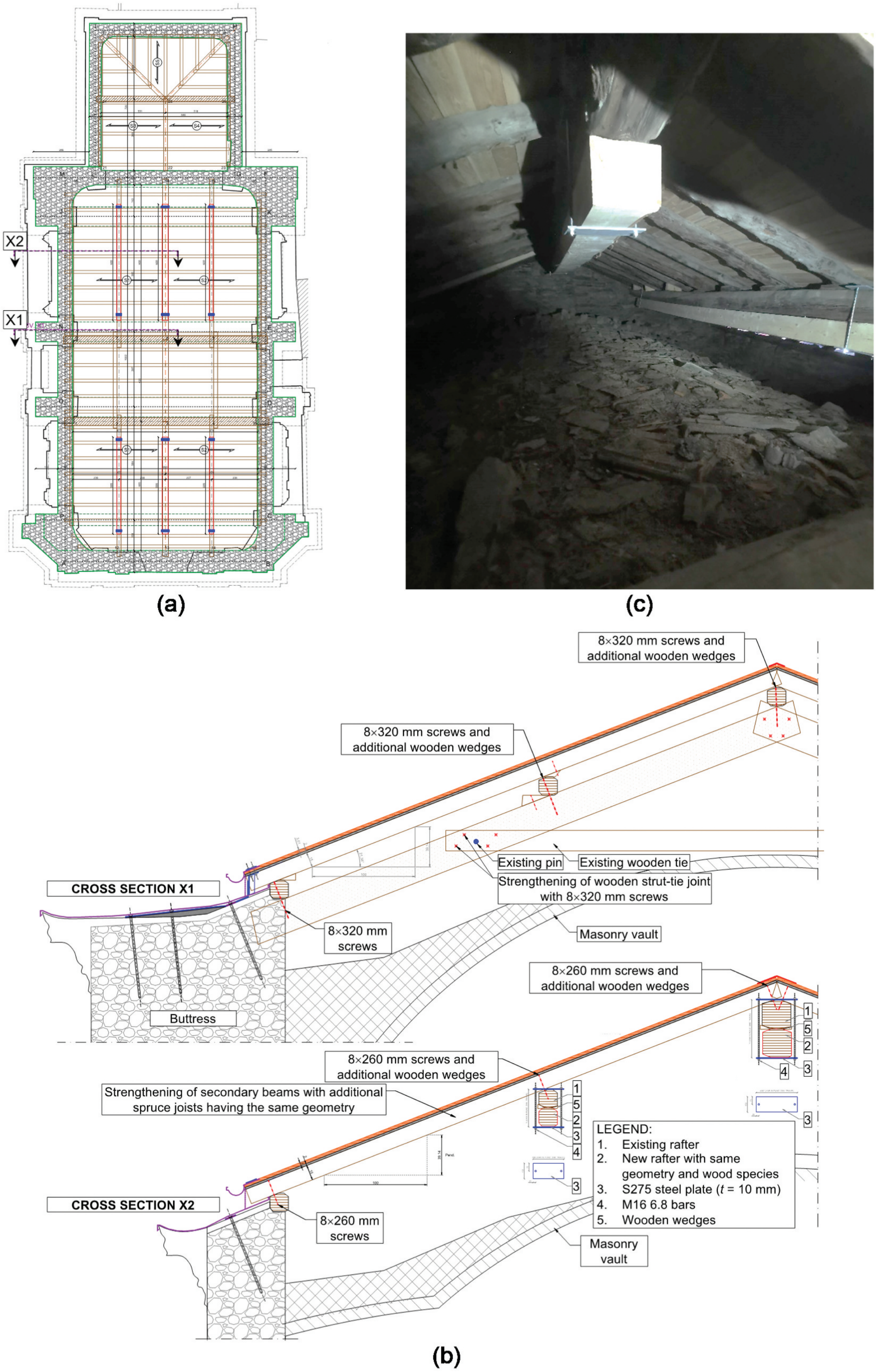

Figure 8. Plan (a) and cross sections (b) of the flexural reinforcement interventions on the roof; (c) realization of the interventions on site.

Figure 9. Views of the church of Ceto after the seismic retrofitting and restoration work.

After having specified these properties, the as-built configuration of the church was firstly analysed. Since, prior to the retrofitting, the building did not feature any effective connections between the roof and the masonry wall, the development of an effective box behaviour was conservatively considered to be unlikely (Section 2.3). Thus, the vulnerability of the construction was evaluated by analysing local collapse mechanisms, in agreement with the Italian guidelines for seismic assessment of churches (Ministero per i Beni e le Attività Culturali 2011), previously mentioned in the Introduction. From these investigations, the aforementioned ratio $\zeta_{E,1}$ was determined.

For the retrofitted configuration, with a roof well connected to the walls and able to act as a stiff diaphragm, local mechanisms can be prevented, also given the compactness of the church. Thus, after a modal analysis, a series of 16 pushover analyses were conducted on the whole building in both plan directions, and from the most vulnerable combination of seismic actions, the aforementioned ratio $\zeta_{E,2}$ was derived. For these pushover analyses, the control node was taken as the centre of mass of the building, on top of the vault, and the roof was modelled as a rigid diaphragm. In order to conservatively prevent the possible overestimation of the displacement capacity of the building, each analysis was stopped as

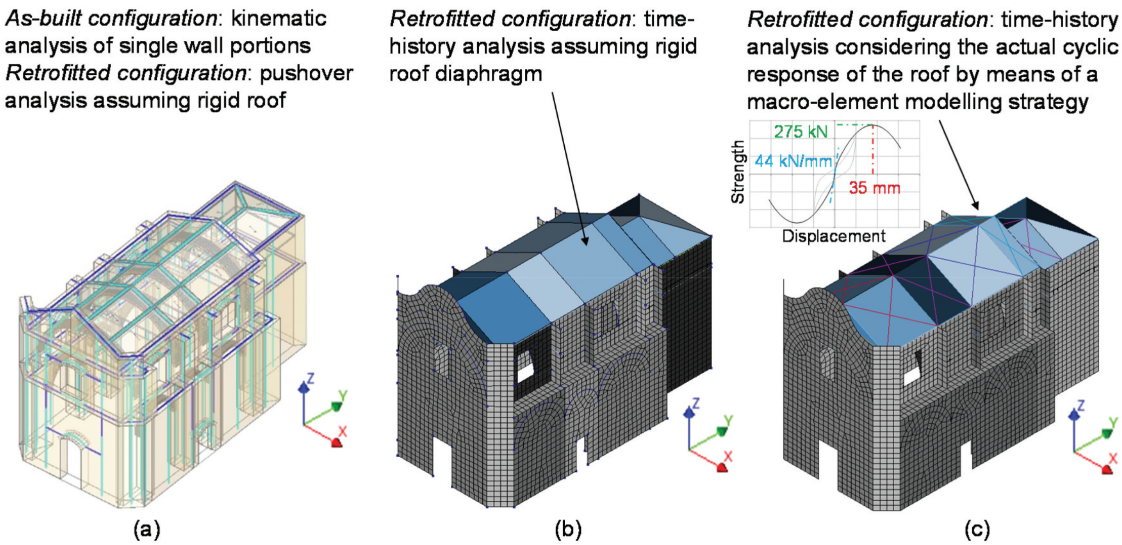

Figure 10. First equivalent-frame numerical model created in PCM.Aedes software (a); Second numerical model created in DIANA FEA software, featuring a retrofitted timber roof assumed as rigid (b) or considering its actual energy dissipation (c).

Table 1. Material properties of stone masonry adopted in the numerical models.

Property	Units	Value	Input to numerical models
Young modulus	MPa	870	Both models
Shear modulus	MPa	290	Both models
Compressive strength	MPa	1.0	Both models
Tensile/shear strength	MPa	0.018	Both models
Density	Kg/m^3	2000	Both models
Fracture energy in tension	N/mm	0.001	Second model
Fracture energy in compression	N/mm	1	Second model
Friction coefficient	-	0.6	Second model

soon as the first masonry pier experienced a drop in in-plane capacity larger than 20% of its strength. During an earthquake, because of the beneficial redistribution effect provided by the roof, it is expected the building could globally retrieve a larger displacement capacity; in these analyses, the objective was to quantify as conservatively as possible the improvement in seismic capacity after retrofitting the church. Instead, the possibility of redistributing seismic forces and providing additional energy dissipation was investigated with the second numerical model, presented in the next section.

4.2. Second numerical model: evaluation of the additional dissipative contribution of the retrofitted roof

The second model (Figure 10b-c) was created in DIANA FEA (Ferreira 2020), where masonry can be simulated as a continuum through shell elements, and nonlinearities and cracks can be accounted for by means of the *Engineering Masonry Model* (Schreppers et al. 2017). The material properties reported in Table 1 were adopted for the whole stone masonry structure, whereas C24 strength class (EN 338:2016) was once more assumed for the timber structural members, to replicate the same conditions of the first numerical model. Two configurations of this second model were realized:

(1) A first case (Figure 10b), in which, similarly to the model in Aedes.PCM, the roof was considered as rigid, thus without accounting for the potential energy dissipation of the plywood panels overlay. In this way, the previously adopted modelling strategy of Section 4.1 and its results could be validated;
(2) A second case, in which the full nonlinear cyclic response of the roof was taken into account (Figure 10c) by means of a macro-element modelling approach (Mirra et al. 2021d). Each macro-element was composed of a quadrilateral of rigid truss elements, and two diagonal springs featuring the constitutive law of the roof: based on the layout of plywood panels, both backbone curves and pinching cycles of the retrofitted roof starting from the single fasteners were constructed, following the analytical model presented in Mirra, Ravenshorst, and van de Kuilen (2021a). The nonlinear behaviour of the floor was simulated by means of the diagonal springs of the macro-elements, in agreement with the modelling strategy implemented in Mirra et al. (2021d), Mirra and Ravenshorst (2021). The nonlinear springs referred to a designed retrofitted roof

strength of 275 kN per pitch (see Section 3), reached at 35 mm in-plane deflection, and an initial stiffness of 44 kN/mm (Figure 10c). Thus, for this case, the roof was regarded as a deformable element and its contribution in terms of hysteretic energy dissipation could be quantified.

For both configurations, after a modal analysis, non-linear incremental dynamic analyses were performed, adopting seven accelerograms, compatible with the response spectrum of Ceto. Among these scaled input signals, three highly destructive events were selected to conservatively assess the seismic capacity of the church, and, namely, the accelerograms referring to the earthquakes of Friuli 1976 (Finetti, Russi, and Slejko 1979), Montenegro 1979 (Benetatos and Kiratzi 2006), and Irpinia 1980 (Bello et al. 2021). The signals were applied contemporarily in both plan directions X and Y of the church, considering 100% of the seismic action for one, and 30% of it for the other, and vice versa (EN 1998:2004).

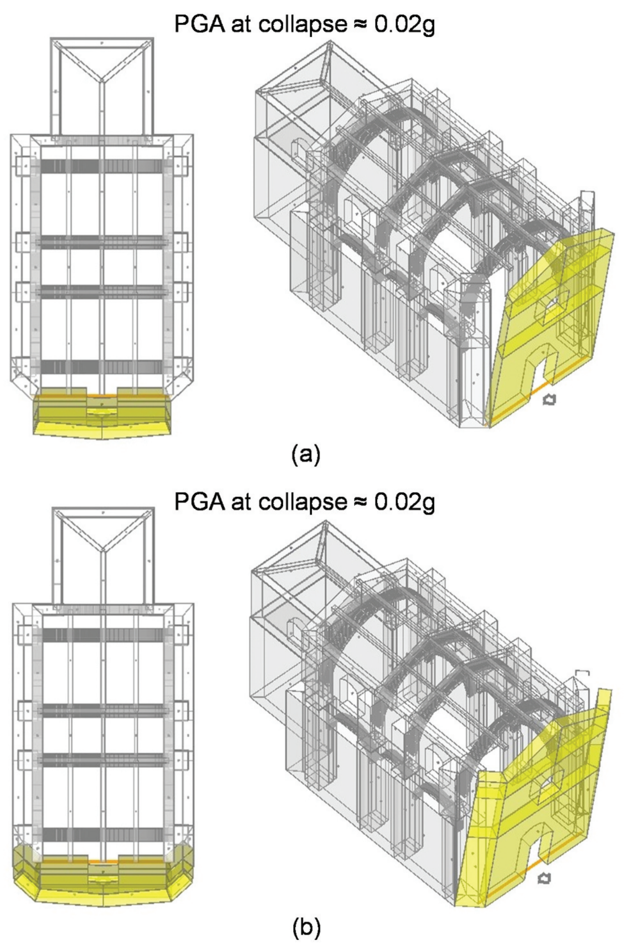

Figure 11. Kinematic analysis of the overturning mechanisms of the front façade in case of partial (a) or total (b) collapse of the masonry wall.

5. Results

5.1. Results from the first numerical model

Because of the low PGA intensity of the site, as well as the regularity and compactness of the church, the as-built configuration showed sufficient capacity to withstand the expected seismic action for several of the most common local collapse mechanisms (e.g. overturning of gables or of nave walls). However, the front façade showed a strong vulnerability (Figure 11), linked to $\zeta_{E,1} = 0.23$ only (PGA at collapse of≈0.02 g). This value can be considered as very conservative, since it was not possible to conduct detailed investigations on the connections between the façade and the rest of the structure. Nevertheless, without a roof acting as diaphragm and proper joints, able to redistribute seismic actions among other structural components, similar failure mechanisms could still take place in the event of a moderate earthquake.

With regard to the retrofitted configuration, the modal analysis showed a regular dynamic response for both plan directions X and Y, with periods of 0.47 and 0.35 s, respectively. This is coherent with the regularity and symmetry of the church, as well as the proper distribution and arrangement of all masonry piers and buttresses. The pushover analyses identified direction X as the most vulnerable, as could be expected based on the presence of more slender piers. The governing curve for this direction is shown in Figure 12a and corresponded to a ratio $\zeta_{E,2} = 0.61$ (PGA at collapse of≈0.05 g). However, as will be shown in the next section, this value represents a lower bound for the seismic performance of the church after retrofitting, because of the conservative assumptions behind the conducted analyses.

Nevertheless, the prescribed requirement of seismic improvement according to the Italian Building Code (NTC 2018) was by far met, since $\zeta_{E,2} - \zeta_{E,1} = 0.38 >> 0.1$. Besides, for buildings with public functions (e.g. schools) and importance class III or IV, an additional requirement of $\zeta_{E,2} \geq 0.6$ is specified, and even if this is not directly applicable to the case-study church, the retrofitted configuration would comply also to this prescription. Thus, the applied interventions allowed to greatly improve the structural behaviour of the building, from both static and seismic perspective.

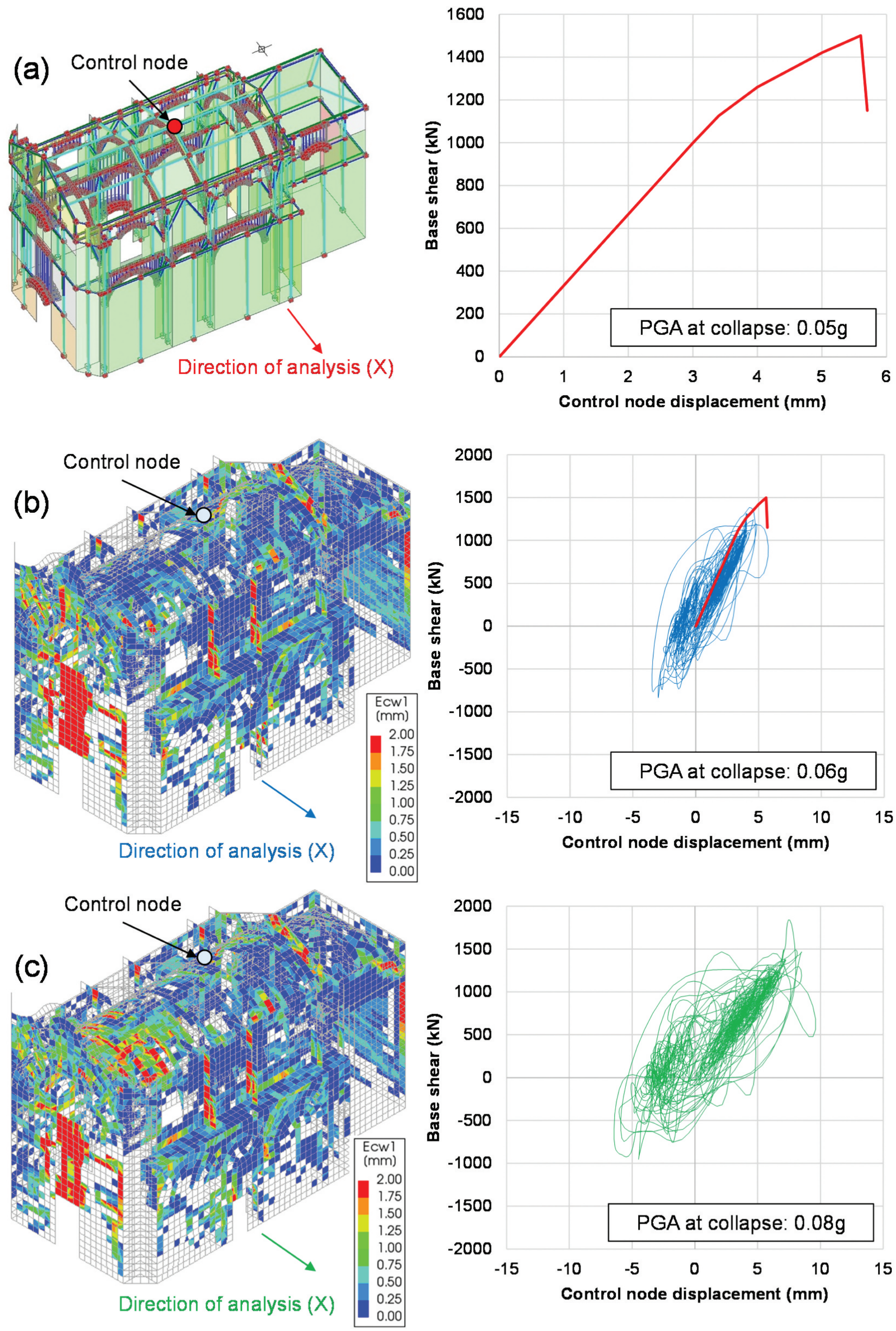

Figure 12. (a) Governing pushover curve from the first model in PCM.Aedes; (b) overview of crack pattern and base shear-displacement graph of the model in DIANA FEA with rigid roof, including comparison with pushover curve; (c) overview of crack pattern and base shear-displacement graph of the model in DIANA FEA with dissipative roof.

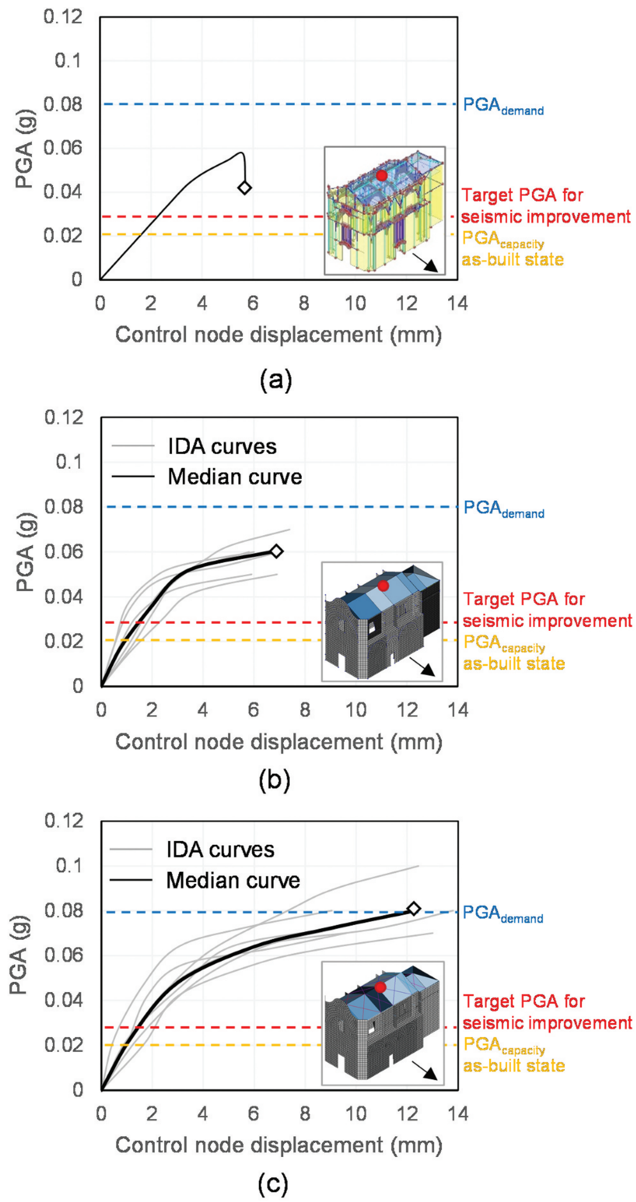

Figure 13. Results from incremental dynamic analyses (IDA) for the most vulnerable direction (X). As a reference, the governing pushover curve from the first model in PCM.Aedes scaled in terms of PGA is shown (a), together with the IDA curves of the configurations with rigid (b) and dissipative (c) roof modelled in DIANA FEA.

5.2. Results from second numerical model

5.2.1. First configuration

The modal analysis conducted on the first configuration of the model (featuring a rigid roof) showed also in this case a regular dynamic response in both plane directions X and Y, with periods of 0.47 and 0.29 s, respectively. These values are in line with those obtained from the first numerical model, although the Y direction showed a slightly stiffer response in this case, probably because of the more detailed modelling of squat walls with respect to an equivalent-frame approach.

With regard to the outcomes of time-history analyses, these also confirmed the response evaluated with pushover analyses in the first numerical model, with an overall slightly larger displacement capacity (Figures 12b and 13). However, the average PGA at collapse obtained for this case along the governing X direction was approximately 0.06 g (±11.8%), corresponding to $\zeta_{E,2} = 0.75$. This result is coherent with the conservative assumptions at the basis of the conducted pushover analyses, which thus led to a value on the safer side. The modelling strategy adopted for this first configuration with rigid roof therefore validates the outcomes from the previous equivalent frame model, as also the incremental dynamic analyses curves appear to be comparable to the pushover (Figure 13a,b).

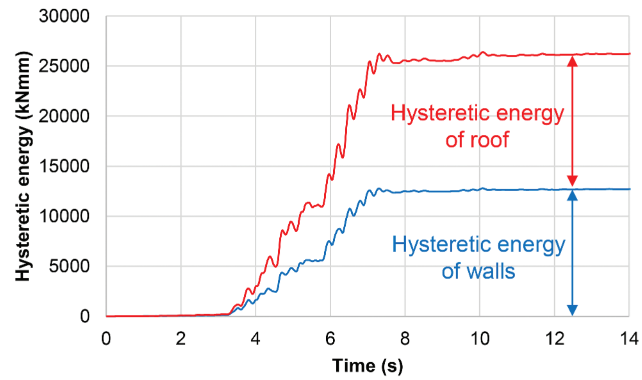

Figure 14. Accumulation of hysteretic energy in the case-study building over time under the 1976 Friuli earthquake; the activation of energy dissipation in the roof is noticeable.

5.2.2. Second configuration

The modal analysis executed on the second configuration of the model led to slightly larger values of the periods, since the strengthened roof was not considered anymore as rigid. In this case, the obtained periods were 0.50 and 0.30 s along the X and Y directions, respectively. Besides, from the results of time-history analyses, accounting for the actual cyclic, dissipative response of the retrofitted roof, an average PGA at collapse of approximately 0.08 g (±12.5%) along the governing X direction, was obtained. In other words, it appears that the case-study church could be able to survive the expected design seismic action for Ceto. This highlights the great contribution of wood-based seismic retrofitting techniques able to increase the displacement capacity and activate energy dissipation in the roof, whose contribution is evident in this case (Figures 12c, 13 and 14).

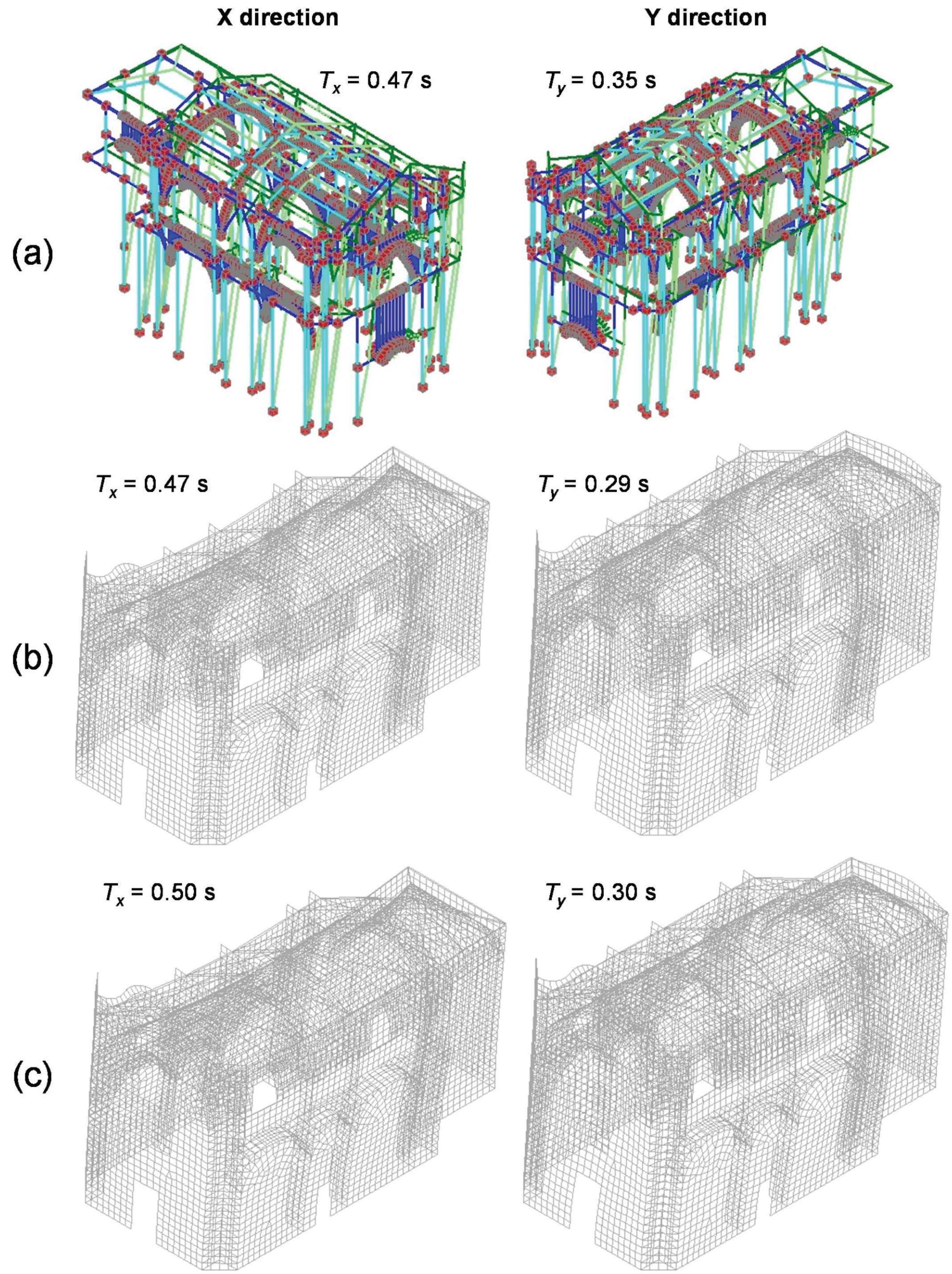

Figure 15. Overview of the mode shapes obtained from the modal analysis on the retrofitted configuration in the first (a) and second numerical model, considering rigid (b) or dissipative roof (c).

5.3. Discussion

As noticeable from the previous sections, the two numerical models provided similar results in terms of both dynamic behaviour and seismic response. Figure 15 provides a visual comparison of the two main mode shapes along X and Y directions for all analysed configurations. Thus, for this case-study building, featuring a compact, regular and mainly symmetric structure, also a simplified modelling strategy based on the equivalent frame approach appears to be appropriate and conservative.

It is also interesting to compare the models in terms of damage to the masonry elements (see again Figure 12). The most loaded elements during the pushover analysis in the governing X directions were located in portions of the façade, and in the bottom part of the apse. This is also confirmed when examining the crack pattern from the second model, highlighting a large amount of damage in these areas. When considering the configuration featuring the actual cyclic response of the roof (Figure 12c), slightly more cracks are present on the vault as well, because the roof was not modelled as rigid in this case. Yet, even with a higher PGA, under the same signal the damage on the in-plane piers is comparable, if not even lower, to the configuration with rigid roof (Figure 11Figure 12Figure 12b). This is an additional confirmation of the beneficial, dissipative role of the retrofitted diaphragm, whose contribution is also highlighted in Figure 14.

As a last consideration, it can be noticed that, in the second numerical model, the ratio between the average PGA at collapse of the first configuration (rigid roof) and second one (actual cyclic response of the roof) is approximately $0.06/0.08 = 0.75$. Given the similarity in terms of fundamental periods between the two configurations, this difference can mainly be attributed to the additional damping contribution of the floor. If the roof was modelled as rigid, in, e.g., a pushover analysis, this energy dissipation could be taken into account by means of an overdamped response spectrum, considering the reduction factor $\eta = [10/(5 + \xi)]^{1/2}$, where ξ is the equivalent damping ratio of the structural system (EN 1998:2004). In this case, with a factor η corresponding to the ratio between average PGAs at collapse of 0.75, the dissipative role of the roof would lead to an equivalent damping ratio of 12–13%. This value appears to confirm, also for this monumental building, previous studies on the dissipative properties of timber diaphragms retrofitted with an overlay of plywood panels (Mirra and Ravenshorst 2021; Mirra, Ravenshorst, and van de Kuilen 2021c).

5.4. Benefits of the applied timber-based techniques for the case-study building

The timber-based retrofitting solutions applied to this case-study church are reversible interventions, and appear to be compatible with the existing structural members, which could be effectively strengthened and protected. Thus, the adopted retrofitting methods enabled the conservation of the existing building, which could now retrieve all seismic strength resources already present in the construction, without adding other more invasive earthquake-resistant vertical elements. In other words, as the interventions allow the roof to act as a diaphragm and prevent local (out-of-plane) collapses of masonry walls, the building is now able to develop a box behaviour against seismic actions. Furthermore, the additional plywood panels overlay fastened to the existing sheathing constitutes a reversible, not invasive intervention, which does not excessively increase mass and stiffness of the floors, and potentially enables additional ductility and energy dissipation, as proved in the conducted numerical analyses.

An important aspect to consider is also linked to the benefits of the applied retrofitting methods not only in terms of improvement of seismic performance, but also from a more practical point of view. The designed solutions were particularly appreciated by the Curia of Brescia and the Superintendence for Architectural Heritage, as the historical and architectural values of the church was preserved, without removing or heavily altering the existing structural members. The whole original roof structure from 18th century could be kept in place, and all wood-based designed solutions allowed for compatibility and reversibility, improving at the same time its static and seismic behaviour. This low impact on the construction, linked to a large improvement in its structural properties, is surely a first point of strength of wood-based techniques in this case study.

Besides, from the perspective of professional engineers, these interventions can be efficiently designed and are particularly affordable. The main strengthening solution applied to the roof, i.e. the overlay of plywood panels, besides being not invasive, was also very cost-effective, and could be realized within the time constraint and limited budget available for seismic retrofitting. This result was possible not only due to lower material costs compared to other solutions (e.g. from 10% to 30% less than the use of steel straps or light steel bracings fastened to the existing sheathing), but especially because of the relatively fast and manageable realization of the intervention: the whole plywood panels overlay was fastened to the existing roof by a local building enterprise composed of only three employees within a single working day. Besides reducing the time needed for the retrofitting execution, this also allowed to keep the existing roof structure minimally uncovered and exposed, as the overlay of plywood panels can be realized rapidly and in separate roof sectors as well, without the need of extensive provisional works.

6. Summary and conclusions

In this work, the application of wood-based seismic strengthening techniques on the roof of a stone masonry church has been discussed. The original wooden structure of the roof, dated back to 18th century, did not show specific or urgent issues from the structural static point of view, with the exception of light decay on a small number of joists, and the presence of undersized load-bearing timber structural elements at the roof level. These issues were solved by means of additional timber elements, having the same geometry and wood species as the original ones, and aiming at the full conservation of the roof structure.

However, several vulnerabilities to seismic loading were detected, since the wooden roof did not feature effective connections among its structural members, or to the surrounding masonry walls. It has been proved that, in the as-built state, local out-of-plane collapses of masonry walls, particularly of the front façade, are very likely for already very low seismic actions. Hence, the roof was strengthened with a plywood panels overlay and well connected to the walls, in order to prevent local failure mechanisms and ensure the development of a proper box behaviour. The benefits of wood-based solutions have been highlighted from both the seismic and the practical perspective, focusing on their reversibility, lightness, as well as cost- and execution effectiveness. In particular, after retrofitting, a great increase in seismic performance of the church can be obtained: the outcomes from the conducted numerical analyses show that the building could potentially be able to withstand the expected seismic action on the site, because of the improved box behaviour and additional energy dissipation provided by the strengthened roof.

The results obtained within the analysis of this case study can contribute to further highlight the benefits of timber-based retrofitting techniques, and to support the research framework promoting their use for the preservation of architectural heritage in seismic-prone countries.

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