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Transformation of Wooden Roof Pitches into Antiseismic Shear Resistance Diaphragms

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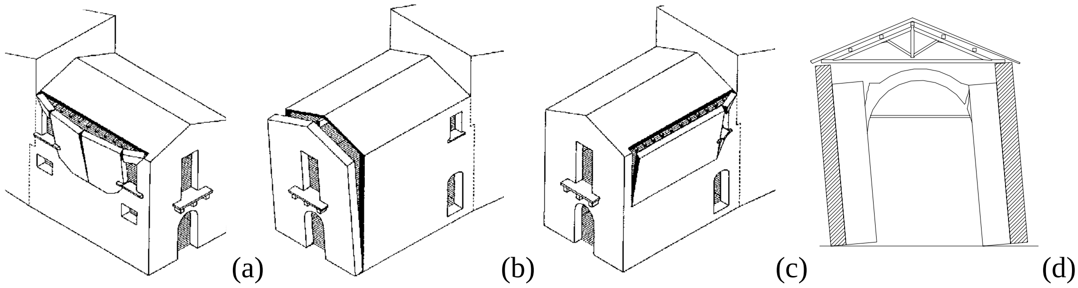
ABSTRACT: A wooden roof strengthening technique aimed transforming the roof pitches into antiseismic shear resisting diaphragms is presented in this paper. Shear diaphragms gather and transfer the seismic loads to the shear resisting walls. Diaphragms are built on top of the existing structures without significantly modifying the roof overall layout. The proposed strengthening technique is mainly reversible and minimally impairing of the building integrity, and can be easily applied for the construction of antiseismic wooden roofs in new buildings. A simplified design approach, based on the resistance criteria, is presented in this paper. The approach allows identifying the static role of each element and its proportioning. The diaphragm technique was recently applied for the antiseismic retrofit of some monumental buildings in Italy. A few case studies, as well as the basic design criteria for applying this technique are presented in this paper.

1 INTRODUCTION

Experience in the assessment and restoration of historical heritage has been carried out in Italy, especially in the last decades. The survey of buildings damaged or collapsed after recent earthquakes allowed to improve the understanding of the structural failure mechanisms and highlighted the inadequacy of some strengthening solutions, especially those implying significant self weight increase.

The structural response of historical buildings subjected to earthquake loads depends on many parameters, such as the overall plan layout, the distribution, typology, interconnection and texture of the masonry walls, the location and size of openings, the occurrence of arches and vaults, the typology of the roof, as well as the floor-to-wall and roof-to-wall connections. As a result, failure mechanisms are not straightforward and they are usually peculiar to each building (D'Ayala et al., 2002). Nevertheless, following recent seismic events, the most recursive collapses and damages observed, generally involved the perimeter wall overturning (Fig. 1a-c), and the rocking of transverse arch pillars, when existing (i.e. church structures, Fig. 1d).



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

Figure 1 : (a,b,c) Wall overturning (De Benedectis et al.,1993); (d) rocking of transverse arch pillars.

The global wall overturning and the pillar rocking occur when the floors, the roof and any possible peripheral ties provide insufficient confinement of the toppling seismic action. Furthermore, overturning actions can be increased by the horizontal thrust of the wooden roof (Fig. 1a, c).

In order to improve the seismic behaviour of masonry buildings, different earthquake-resistant features are traditionally proposed, namely: perimeter ties, or floor diaphragms. The perimeter horizontal steel ties can be inadequate in case of unfavourable length-to-thickness ratios of the masonry walls. In this scenario, in-plane shear resistant floor and roof diaphragms, transforming the building into a box structure, can be a viable solution to restrain the wall overturning (Cordini and Giuriani 1996; Giuriani and Marini 2002; Giuriani 2005). In case of particular structures, such as churches or at-sight wooden roofs, only roof diaphragms can be arranged. In these cases, it is worth noting that the use of steel trusses might induce problems in the compressed slender elements, as well as in the balance of the nodes along the ridge (Giuriani and Marini 2006).

In this paper, the box structure solution is addressed with concern to the formation of roof diaphragms. The roof pitches are transformed into folded plates, which gather and transfer the seismic action to the shear resisting walls.

The roof diaphragms can be designed to resist the roof seismic action and lateral thrust. In the case of long span walls, when the perimeter ties are ineffective, the roof has to be proportioned to resist also to the pertaining top wall seismic actions. Either way aims at reducing the wall toppling action and therefore allows enhancing the structure performance in case of a seismic event..

Finally, it is worth noting that the roof and floor diaphragm solution is suitable to secure the structure against the first and most significant collapse mode, involving the wall overturning. Conversely, the strengthening of roof and floors is ineffective to avoid further and secondary failure mechanisms, such as the wall in-plane shear sliding or the masonry out-of-plane collapse along the interstorey height and between the cross walls, which can be triggered by very strong earthquakes (Magenes et and Calvi 1997; Griffith et al. 2003).

The crowning walls between the roof and the top floor can be sometimes identified as another weak point of the structure. The natural arch transferring the masonry seismic action to the top floor and to the roof diaphragms might be unable to resist the out-of-plane load because of an insufficient vertical confinement. In this case, vertical ties or anchored bars can solve the problem. However, the study of these mechanisms and the structural strengthening technique to control them are beyond the scope of this paper.

2 PITCH DIAPHRAGM TECHNIQUE

The box structure is acknowledged by the modern recommendation (Eurocode 5 2005), however little relevance is given to the interaction between the different structure components and to the roof-to-masonry connection, which is of great importance for this technique to be statically efficient (Giuriani and Marini 2002).

In order to form the pitch diaphragms, the techniques which are commonly used and developed for the flexural strengthening of wooden floors, can be addressed. Minor adjustments allow improving the in-plane shear stiffness and strength of the wooden floor.

In case of roof diaphragms, careful attention should be paid to avoid any increase in the structural weight and thus any increase in the seismic action. To this end, roof diaphragms can be preferably obtained by superimposing plywood panels, connected to each other by means of nailed steel straps, see Fig. 2a (Giuriani et al. 2005). Each pitch web panel is nailed to the perimeter steel chords, and the whole diaphragm is connected to both roof rafters and masonry walls by means of steel studs and vertical anchored bars.

Alternatively, the pitch diaphragms can be arranged by superimposing new thick planks which are fastened together by means of horizontal studs embedded alongside neighbouring boards (Giuriani et al. 2005) (Fig. 2b). The efficiency of this technology was proved experimentally, but further refinement is needed for the technology to be applied in the construction practice. Note that this solution stems as an enhancement of the orthogonal wooden plank technique proposed by Benedetti et al. (1981).

Another “dry technique” was proposed by Giuriani and Plizzari (2000) by nailing very thin steel plates (2 mm) to existing wooden planks (Fig. 2c). In order to avoid buckling induced by in plane shear, the thin plate must be connected to the wooden floor by means of appropriate steel screws. This technique is expensive and suitable only for roofs with regular geometry.

The solution of the thin ordinary concrete slab (50 mm) can be easily applied to irregular shaped roofs and is fairly economical (Piazza and Turrini 1983; Gelfi and Ronca 1993, Giuriani and Frangipane 1993) (Fig. 2d). As a shortcoming, the concrete slab can cause problems to the masonry walls by significantly increasing the roof self weight and seismic action. For these reasons this solution, which is the most commonly used for the flexural strengthening, is often debated when applied for structure seismic upgrading.

The use of high performance concrete is acknowledged as a recent improvement of the concrete slab technique (Meda and Riva 2001). The slab thickness is reduced to 20 mm, thus halving the additional dead loads.

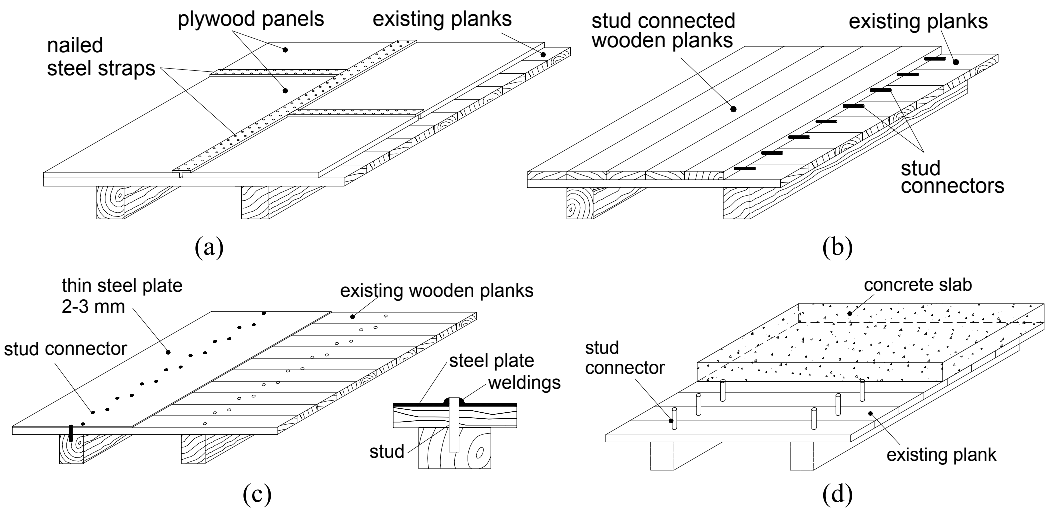


Figure 2 : Wooden floor in plane shear strengthening by means of (a) plywood panels; (b) stud connected wooden planks; (c) thin steel welded plates; (d) thin concrete slab.

3 DESIGN CRITERIA

The design of roof diaphragms should first satisfy the resistance criteria to allow the transferring of the seismic actions to the shear resisting walls. Nevertheless, a deformation control might be sometimes necessary in order to avoid large in-plane deflection of the roof diaphragm that could damage the walls and endanger the efficiency of any restoration work (Cordini and Giuriani 1996; Giuriani et al. 2005). To this end, sophisticated approaches, such as the Finite Element method or the classical folded plate theory, are available (Timoshenko, 1989). These approaches are also needed to model irregular and complex buildings, and whenever refined design is required.

In this paper, a simplified approach based on the resistance criteria is proposed. The approach easily allows identifying the static role of each roof diaphragm component, as well as its proportioning. The criteria discussed herein focus on the design of antiseismic diaphragms on gable roofs; minor adjustments are needed for this technique to be applied to different saddle roof typologies, including hipped roofs (Giuriani and Marini 2002).

The roof pitches are transformed into shear resisting diaphragms, which gather and transfer the seismic loads to the shear resisting walls.

The main structural elements composing the box-structure are: the pitch-panel (*1*), the eaves chords (c_{13}) and head gable chords (c_{12}), and the head gables (*2*) (Fig. 3).

The pitch diaphragms behave like chord and panel structures, in which the eaves chords and pitch panel withstand in-plane bending and shear, respectively (Bruhn 1973).

It is worth noting that the box structure must be designed by taking into account the whole structure behaviour, as well as by considering every single component and their mutual interaction. The pitch and the top floor diaphragm design cannot be decoupled from the structural verification of the head gable stiffness and strength, and the crowning wall capacity of transferring the seismic action.

The design of the main structural elements composing the box structure is based on the hypothesis that the lower floor (*4*), and the head gables (*2*) behave like in-plane rigid diaphragms (Fig. 3). The pitch rafter-to-wall as well as the rafter-to-rafter constraints along the roof ridge is modelled as hinged. The same conservative assumption is made for the perimeter wall footing.

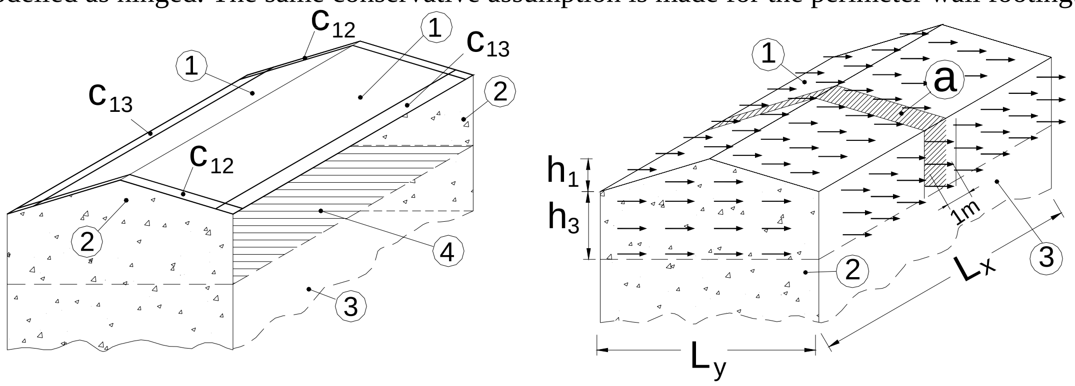


Figure 3 : (a) Structural elements for the wooden roof strengthening; (b) seismic action distribution.

In this scenario, the unit width frame (*A*) constrains are those shown in Fig. 4. In order to evaluate the internal forces in the box structure, a two-step approach is addressed, namely: (i) vertical and horizontal additional constraints are introduced along the roof ridge; (ii) provisional constraints are removed and their effect is backed out.

In the first step, frame (*A*), with additional vertical and horizontal constraints on the roof ridge, undergoing seismic actions p_1 and p_3, behaves like a frame, whose internal forces can be easily evaluated. Each element is subjected to both axial forces and bending moments. The wall elements resist axial forces and bending moments by developing a natural arch, whose resistance depends on the wall thickness, and on the vertical confinement provided by the vertical loads. Tensile uplifting vertical forces n_A (Fig. 4) must be balanced by the structural self weight. Tensile forces n_A are equal to:

$$n_A = (p_1 l_{12} h_1 + p_3 h_3 h_1) / L_y \tag{1}$$

In case tensile forces are larger than the self-weight, anchored bars, suitably embedded and distributed along the crowning masonries must secure the structure against uplift.

In the second step, the additional ridge constraint reactions (r_{1y} and r_{1z}) are cancelled with forces of equal intensity and opposite sign. It is worth noting that only the horizontal reaction needs to be cancelled, as the vertical force is null (f_{1y} = -r_{1y} ; r_{1z} =0; Frame B).

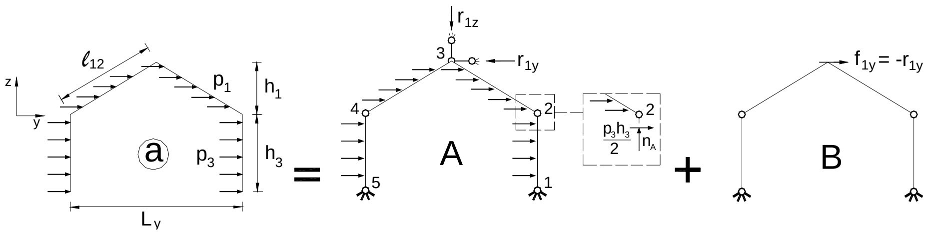


Figure 4 : Frame action.

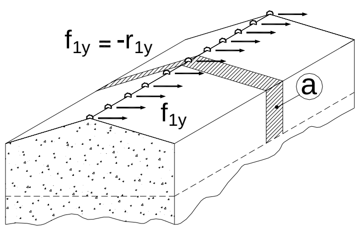


Figure 5 : Box-structure undergoing horizontal ridge loads.

Frame B is labile, thus unable to resist the horizontal force f_{1y}. For the structure to resist seismic actions, the structure box behaviour must be triggered (Fig. 5).

The horizontal loads applied to the roof folded diaphragm along the ridge axis are:

$$f_{1y} = 2p_1 l_{12} + p_3 h_3 \tag{2}$$

where l_{12} = pitch width, h_3 = upper floor interstorey height (Fig. 4).

The roof diaphragm behaves like a simply supported beam. The horizontal forces f_{1y} are applied to the folded plate shear centre (Fig. 6a), hence no torsion is introduced and therefore no vertical reactions, other than those generated in step 1 by the frame behaviour (Frame A in Fig. 4), are added in step 2 along the eaves lines.

The maximum bending moment at the folded plate mid span is equal to:

$$M = \frac{f_{1y} L_x^2}{8} \tag{3}$$

The bending moment is resisted by two axial forces F_{13} (Fig. 6b) acting on the eaves chords:

$$F_{13} = \frac{M}{L_y} = \frac{f_{1y} L_x{}^2}{8 L_y} \tag{4}$$

where L_y = distance between the eaves chord centre of mass.

The shear force is applied at the ridge level (Fig. 6c) and is transferred by the pitch diaphragms to the head gables. The maximum shear force is equal to:

$$T_1 = \frac{f_{1y} L_x}{2} \tag{5}$$

The shear flow along the diaphragm cross sections is constant and equal to (Fig. 6c):

$$q_1 = \frac{T_1}{2} \left(\frac{L_y}{2\cos\alpha} \cos\alpha \right)^{-1} = \frac{T_1}{L_y} \tag{6}$$

The head gable balance requires horizontal and vertical reactions R_z and R_y:

$$R_z = \frac{T_1 h_1}{L_y} \text{ and } R_y = T_1 \tag{7}$$

where h_1 = vertical distance between eaves and ridge axis.

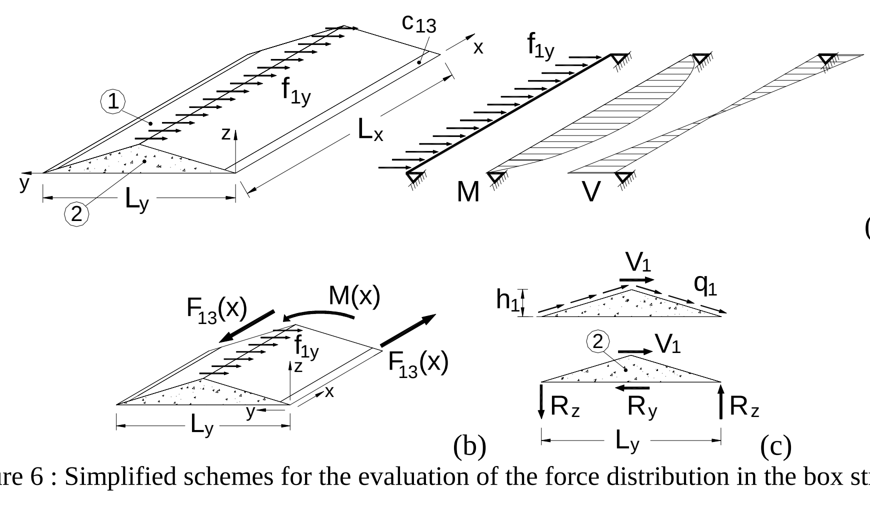


Figure 6 : Simplified schemes for the evaluation of the force distribution in the box structure.

Concerning the proportioning of the roof diaphragms, the eaves chords cross section is proportioned to resist the axial forces F_{13} (Eq. 4). The pitch panel thickness, the connections between pitch panels, as well as the connection of the panels to the head gable chords and to the head gables are proportioned to resist the maximum shear flow q_1 in Eq. (6) (Fig. 6c). Note that, as the shear flow decreases along the x-axis, the spacing of the connections between neighbouring plywood panels can be increased along the span. Finally, vertical anchored bars can be introduced to secure the roof against uplift when the uplift force R_z, in Eq. (7), and the tensile forces n_A in Eq. (1) are larger than the roof self weight.

4 ANTISEISMIC RETROFIT OF ANCIENT WOODEN ROOFS

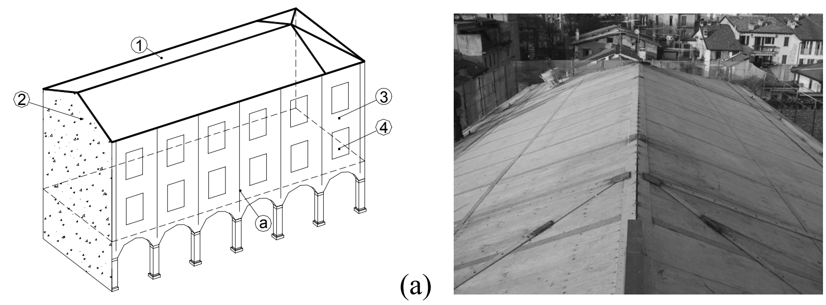
The proposed technique was recently adopted in the antiseismic design of some monumental buildings in Brescia, Italy. A few case studies are discussed in the following.

4.1 Palazzo Calini ai Fiumi

The western wing of the Palazzo Calini ai Fiumi, new headquarters of the Law Department of the University of Brescia, was recently restored with the antiseismic retrofit of the wooden roof (Giuriani, 1998) (Fig. 7). The two-layer building is composed of a porch at the ground floor, overlooking an internal courtyard, and a large conference room on the first floor (10×30 m, interstorey height of 7 m). The conference room outer walls lack a transverse reinforcing structure preventing the out-of-plane displacements. Furthermore, the masonry wall facing the internal courtyard is simply supported on the porch columns and is therefore not rigidly fixed to the ground. As for the roof, the inferior purlins are too slender to sustain seismic action pertaining to the wall.

The retrofit works aimed at strengthening the wooden roof and the main room floor 4 against seismic actions by placing resisting diaphragms on top of both structures (Fig. 7). Note that the traditional perimeter ties would have been inadequate given the large masonry span. The wall on the colonnade was strengthened with vertical steel bars *(a)* (Fig. 7) built in the wall between the openings. This way the masonry was provided with sufficient strength and stiffness to transfer the seismic actions to the resisting diaphragm of both the main room floor and the saddle roof.

The floor and the roof were designed to resist and transfer both their seismic actions and those pertaining to the lateral walls to the shear resisting head gables *2)*. The floor diaphragm was obtained by casting a thin concrete slab on top of the existing wooden floor, whereas the pitch diaphragms were made of plywood panels connected by thin steel plates nailed to the panels. Fig. 7 shows the principal elements composing the antiseismic structure, as well as the roof folded plate upon completion.

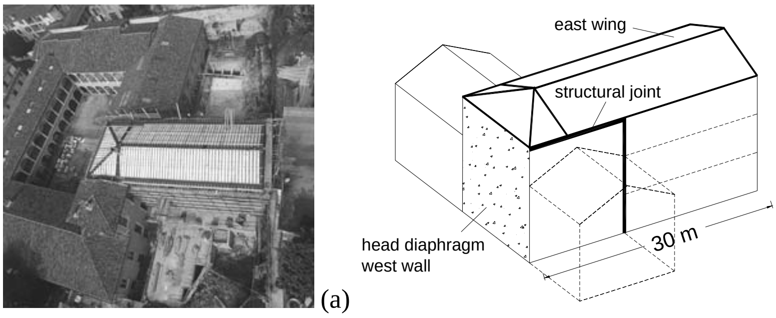


(a) (b)

Figure 7 : Palazzo Calini ai Fiumi: (a) simplified scheme of the structure; (b) antiseismic wooden roof.

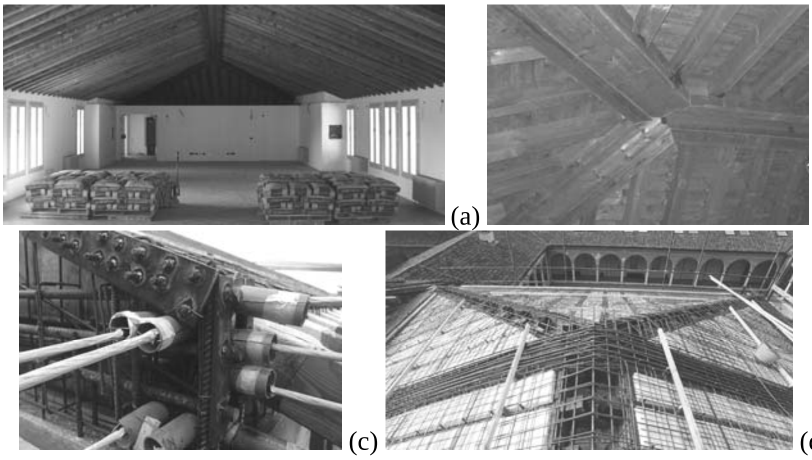
4.2 East wing of the Santa Chiara monastery in Brescia.

The east wing of the Santa Chiara monastery, headquarters of the Business Department of the University of Brescia (Fig. 8), was partly demolished in order to build the main room in the basement. A structural joint separates the east wing (9 m x 20 m in plant) from the adjoining building (Fig. 8b). The northern wall bridges the main room and carries 3 floors and the roof. The upper floor room, with a length of 30 m, occupies the surface of the east wing and extends into the adjoining building as shown in Fig. 9b. Given the peculiar shape of the structure and the presence of the structural joint, the roof was designed to transfer the seismic loads pertaining to the entire building to the lateral head walls. The structural joint crosses the entire building section with the exception of the roof. The roof is connected to the west head walls with special supports transferring the transverse seismic action while allowing every free longitudinal displacement of the roof.



(a) (b)

Figure 8 : Santa Chiara: (a) aerial view; (b) simplified scheme of the structure.



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

Figure 9 : Santa Chiara Building: (a,b) internal views of the roof structure; (c) detail of the ordinary rebars of the eaves chords and head of the prestressed rebars; (d) detail of the hipped end.

The wooden roof, made of a saddle segment and an adjoining hipped end, was strengthened against seismic loads by casting a thin concrete slab on top of the existing wooden plank.

To avoid any obstacle of the overall volume of the upper room, no intermediate wooden trusses were used. Accordingly, the vertical loads as well as the seismic actions pertain to the box structure. Fig. 9a, b shows an internal view of the roof.

Stud connections link the 50 mm thick concrete slab to the wooden purlins. The ridge and eaves chords are made of reinforced concrete (Fig. 9c). The eaves chords were partially prestressed by inserting strands next to ordinary rebars (Fig. 9d). In service conditions the partial prestress avoids crack formation, whereas the structure capacity against seismic loads was evaluated neglecting the strands and thus assuming that all the resistance be provided by the ordinary rebars. Prestressed chords were also built around the east head gable and the west hipped end. Fig. 9e shows in detail the steel reinforcement of the ridge chord, the lateral purlins, the diagonal hip rafters, and the pitch plates.

5 FINAL REMARKS

Masonry buildings, damaged or collapsed, during recent seismic events recursively exhibited the overturning of the perimeter walls. Damages can be even more pronounced in long-span buildings lacking cross walls, as the wall span-to-thickness ratio is unfavorable and little constraint is provided to the toppling walls. In these cases, existing or added peripheral ties are ineffective. In-plane shear resistant floor and roof diaphragms, transforming the building into a box-structure, can be a viable solution to restrain the wall overturning.

The diaphragm technique has to be mainly reversible and lightweight, as recommended by modern restoration principles. In order to meet these principles, the proposed technique adopts lightweight diaphragms, which are obtained by superimposing plywood panels to the existing wooden roof. Pitch panels are secured to each other and to the masonry walls by means of nail and stud connectors.

In this paper a simplified design approach, based on the resistance criteria, was introduced. The proposed simplified approach allows identifying the static role of each box-structure component, and yields its proportioning. The model takes into account the whole structure behaviour, with particular attention to the roof diaphragm-to-perimeter wall interaction.

Concerning the pitch diaphragms, the adopted simplified model highlighted the importance of the perimeter chords along the eaves and head gable axes, as well as the importance of the vertical anchorages embedded in the crowning masonries along the eaves.

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