

Experiences from the Northern Italy 2004 earthquake: vulnerability assessment and strengthening of historic churches

Ezio Giuriani

University of Brescia, Italy

Alessandra Marini

University of Brescia, Italy

ABSTRACT: This work is about the lesson learnt from the analysis of the damages caused to some churches by the strong earthquake, which struck Northern Italy in the year 2004. Focus is paid to some mechanisms which are not well known in the literature: the excessive and differential rocking of neighbouring transverse arches, the differential deflection of single leaf vault rings, the tie over-tension induced by the transverse arch rocking. In the paper, the structural solutions appositely studied to repair and to reduce the structure seismic vulnerability of these buildings are presented. Among these solutions: internal perimeter ties, lightweight wooden roof box structure, lightweight spandrel ribs are illustrated.

1 INTRODUCTION

In the year 2004, a strong earthquake struck the Benaco region in Northern Italy. The earthquake caused severe damage to both historical and modern constructions. Within the historical heritage about 250 churches were listed among the most severely impaired buildings.

This work is about the lesson learnt from the analysis of the damages caused by the 2004 earthquake to some churches, and discusses the structural solutions appositely studied to repair and to reduce the structure seismic vulnerability of these buildings.

During the damage survey campaign, some recurrent and classical collapse mechanisms were identified: the overturning of the main façade and the compound walls, the out of plane deflection of the perimeter walls between the transverse arches and along the interstorey height, the wall in-plane shear failure. These mechanisms are well discussed in the literature (De Benedectis et al. 1993, Giuffrè 1993, Magenes and Calvi 1997, Lagomarsino 1998, D'Ayala and Speranza 2002, Griffith et al. 2003, Lagomarsino et al. 2004).

In some churches, where the lateral wall overturning was inhibited either by massive buttresses alongside the nave walls, or by thick abutments, excessive, unconstrained rocking of transverse arch pillars was frequently observed (Doglioni et al. 1994, Lagomarsino et al. 1999, Giuriani et al. 2007 and 2008).

Against these well known failure mechanisms perimeter ties placed along the masonry walls are the

classical solution. Alternatively, steel truss works are often built on top of the roof pitches. As a drawback however, in the case of long spanned building perimeter ties might be ineffective; whereas with the adoption of steel truss work high stresses might be concentrated in rather small zones and the effectiveness of the strengthening solution should be carefully evaluated.

In some churches vulnerability was shown to be associated to less frequent mechanisms, which are not so well known and discussed in the literature.

In San Pietro Church (Roè Volciano, Brescia), one of the most severely damaged churches, two single leaf barrel vaults collapsed during the earthquake because of the differential rocking of the diaphragm arches, and because of the differential deflection of the thin vault rings. In other churches the over-tensioning of the existing steel ties caused either their unthreading from the masonry walls, from exceeding the anchorage resistance, the tie yielding (San Lorenzo Church in Clibbio) or failure (San Antonio Church in Roè Volciano).

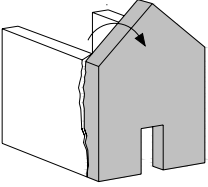

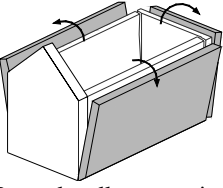

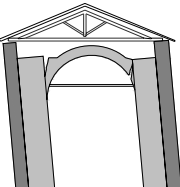


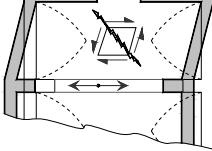

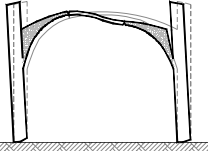




Against these mechanisms, different solutions were proposed: internal perimeter ties hidden behind the nave mouldings, lightweight plywood roof box structures, and lightweight clay mortar spandrel ribs.

These strengthening techniques, as well as the above mentioned collapse mechanisms are discussed in the paper.

2 SEISMIC VULNERABILITY OF CHURCHES

The vulnerability of the churches is generally associated to the overturning of the main façade and the perimeter walls, as well as to the excessive unconstrained rocking of the transverse diaphragm arches.

Table 1. Seismic vulnerability assessment of some churches hit by the 2004 earthquake with respect to some failure mechanisms.

 <p>Façade overturning</p>	$C^{(*)} = 2.5\%$ (A) $C = 1.0\%$ (B) $C = 3.5\%$ (C) $C = 5.0\%$ (D) $C = 5.0\%$ (E) $C = 5.2\%$ (F) $C = 3.3\%$ (G) $C = 7.8\%$ (H) $C = 10.7\%$ (I)	 <p>(A) Madonna della Rocca, Sabbio Chiese, Brescia.</p>
 <p>Lateral wall overturning</p>	$C = 5.0\%$ (A) $C = 4.0\%$ (B) $C = -$ (C) $C = -$ (D) $C = -$ (E) $C = 6.0\%$ (F) $C = -$ (G) $C = -$ (H) $C = 12\%$ (I)	 <p>(B) San Lorenzo Church, Clibbio, Brescia.</p>
 <p>Diaphragm arch excessive rocking</p>	$C = 3.5\%$ (A) $C = 7.0\%$ (B) $C = 6.3\%$ (C) $C = <0$ (D) $C = 0$ (E) $C = -$ (F) $C = <0$ (G) $C = 7.4\%$ (H) $C = 48\%$ (I)	 <p>(C) San Pietro Church (Roè Volciano, Brescia)</p>  <p>(D) Sant'Antonio, (Roè Volciano, Brescia)</p>
 <p>Diaphragm arch differential rocking</p>	<ul style="list-style-type: none"> • remarkable crack pattern surveyed in A, B, D, E, G, H • Collapse surveyed in C 	 <p>(E) San Rocco, (Roè Volciano, Brescia)</p>
 <p>Single leaf vault differential bending</p>	<ul style="list-style-type: none"> • remarkable crack pattern surveyed in A, B, D, E, G, H • Collapse surveyed in C 	 <p>(F) Santissima Trinità, (Roè Volciano, Brescia)</p>
 <p>(H) SS. Faustino e Giovita (Montemaderno, Brescia)</p>	 <p>(I) SS. Pietro e Paolo (Preseglie, Brescia)</p>	 <p>(G) Santa Maria Assunta (Bione, Brescia)</p>
$(*) F_S = C W_{TOT}$ where: F_S is the seismic force triggering the mechanism; C is the load collapse multiplier (also regarded as vulnerability index); W_{TOT} is the structure total weight.		

In the churches surveyed following the 2004 earthquake, other mechanisms, such as the excessive and differential rocking of the neighbouring diaphragm arches and the differential deflection of the thin vaults, were acknowledged as further causes of the severe damages observed.

The seismic vulnerability of the churches was assessed by reference to the limit analysis approach.

Table 1 shows the vulnerability coefficients estimated for some significant case studies.

2.1 Overturning of the main façade

Overturning of the main façade was often observed during the survey campaign (Figure 1, Table 1). This collapse mechanism was caused by the seismic action of the wall and by the possible lateral seismic thrust of the roof beams. Provided that wooden roofs are usually unconstrained against lateral movements, the latter contribution to the toppling action was quite relevant in most of the analyzed churches.

The onset of the overturning mechanism is underlined by the development of cracks extending at the wall edges and progressively reducing their width from the top to the footing of the building. The cracks can be vertical or angled, depending of the degree of anchorage to the perimeter walls. However, it is worth noting that, the measured crack angle was always very small, thus the stabilizing contribution of the lateral walls was almost negligible in most cases.

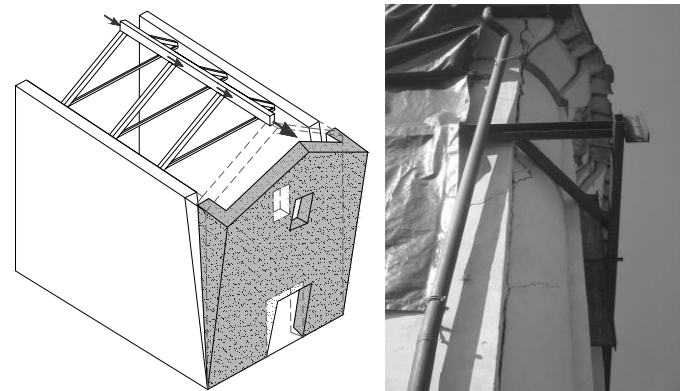


Figure 1. Overturning of the façade induced by the roof seismic thrust.

2.2 Excessive rocking of the diaphragm and triumphal arches and existing tie over-tension

Excessive, unconstrained rocking of the diaphragm and triumphal arches was recognised as another source of vulnerability of many churches in the Benaco area (Table 1).

In rest condition, when only vertical loads are applied to the structure, the arch lateral thrust can be either largely or even entirely resisted by the buttress action developed by the abutments, depending

on their shape and massiveness (Figure 2a, Giuriani and Gubana 1993 and 1995). As a result the possible ties are required to confine the arch thrust portion, which exceeds the buttress action. The proportion of the confining contribution mainly depends on the structure geometry and tie pretension. Conversely, in the case of rocking, the resisting mechanism significantly modifies (Figure 2b, Giuriani et al. 2007 and 2008). The crack pattern is characterized by two large horizontal cracks developing at the abutment bases and two cracks opening at the diaphragm arch springing (Figures 2b, 3 and 4). The location of the cracks depends on both the structure geometry and the local resistance.

plus the thickness of the abutment (d) (Giuriani et al. 2007, 2008). For typical case studies the ratio d/L^* was observed to range between $0,15 \div 0,2$, thus when shifting from rest condition to rocking the arch thrust could be $30 \div 40\%$ larger than the thrust at rest condition.

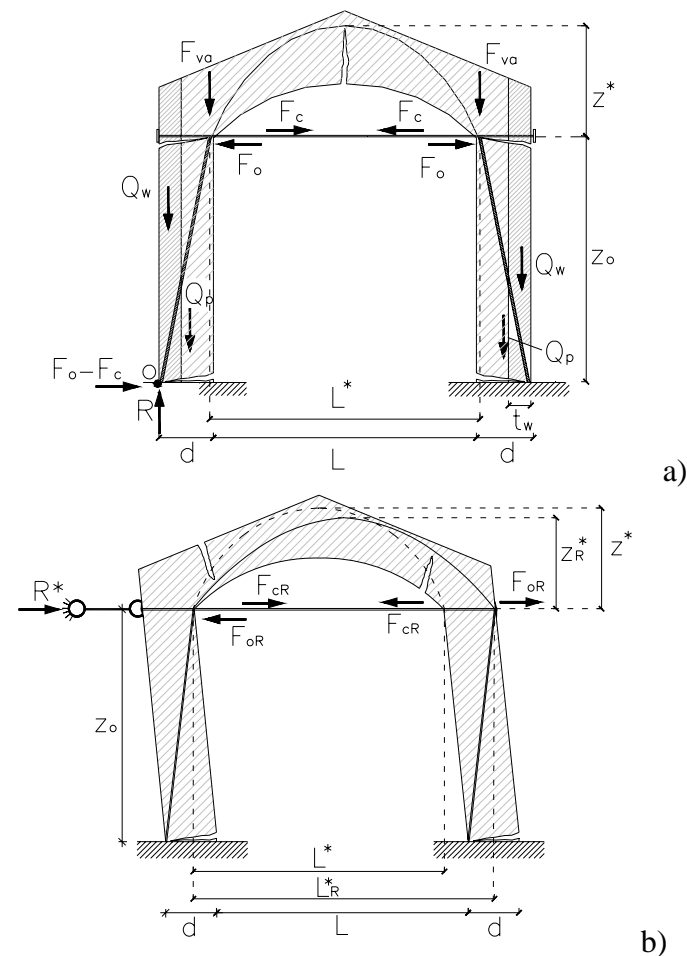


Figure 2. Transverse arch resisting mechanisms under a) vertical loads (rest condition) and b) horizontal loads (case of full rocking, Giuriani et al. 2008).

The abutment buttress action dramatically diminishes as the crack penetration progresses. In the case of full rocking, when the crack pattern is fully developed, no abutment buttress action can be accounted for, because the resisting ideal struts become parallel to each other. As a result, the arch thrust has to be entirely confined by the existing ties.

Furthermore, following the cracking triggered by the rocking motion, the span of ideal arch significantly increases and so does the arch lateral thrust (Figure 2b). The span of the ideal arch in rocking condition can be conservatively assumed to be equal to the span of the ideal arch in rest condition (L^*)

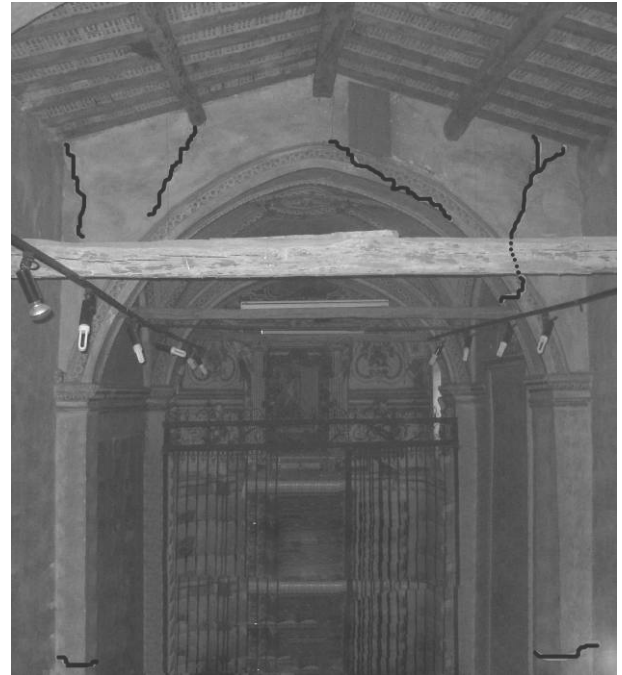


Figure 3. Madonna della Rocca Church, Sabbio Chiese, Italy. Transverse arch crack pattern induced by the seismic action.



Figure 4. Detail of the horizontal crack pattern at the transverse arch abutment base induced by the seismic action.

In the case of full rocking, the decrease of the abutment buttress action and the increase in the ideal arch span cause the existing tie over-tension. When the tie resistance is barely sufficient to withstand the traction force in rest condition, no extra resources are available in case of earthquake, and the collapse of the tie can be expected.

Given this foreword, the failure of the existing ties in San Antonio Church (Roè Volciano, Figure 5) was interpreted as a consequence of the onset of the transverse arch full rocking mechanism. In some

other churches, where weak existing ties were also surveyed, the over-tension caused either the tie unthreading from the masonry walls from exceeding the anchorage resistance (Madonna della Rocca church, Sabbio Chiese), or the tie yielding (San Lorenzo Church in Clibbio).



Figure 5. Detail of the welded tie rod in San Antonio Church (Manerba, Brescia, Italy). The tie rod collapsed during the 2004 earthquake.

As for the vulnerability assessment, the evaluation of the safety factor with respect to the onset and development of the mechanism can be obtained by reference to the traditional approach of the principle of virtual work (Abruzzese and Lanni 1999). The same approach can be also addressed to define the structure capacity curve (Housner 1963, Curti et al. 2006), to study the abutment out-of-phase rocking (Como et al. 1991), and to evaluate the tie tension (Lagomarsino et al. 2004).

A simplified method was proposed in Giuriani et al. (2007 and 2008) for the evaluation of the collapse multiplier and the tie tension in the case of either over resistant or weak ties. The seismic collapse multiplier was evaluated by introducing an ideal additional horizontal constraint at the tie level, and by enforcing that its reaction force R^* was nil (Figure 2b).

2.3 Differential rocking of the diaphragm arches and differential deflection of the thin groin vault.

The different stiffness of the transverse arches along the nave can be regarded as another source of seismic vulnerability of the churches (Table 1). The maximum top displacement experienced by the transverse arches during the rocking motion depends on the geometry and stiffness of the structure, therefore differential displacements can be expected between adjacent transverse arches (Figure 6). Noticeable differential displacement can also occur in the

case of out-of-phase rocking of neighboring diaphragm arches.

The largest differential displacements are usually experienced by the first and last bay transverse arches. As a matter of fact, the façade rarely undergo rocking, unlike the nearby transverse arch; and the triumphal arch is typically much stiffer than the neighboring transverse arch. This way, the vaults covering the first and last bays are subjected to shear distortion. When differential rocking is severe, shear distortions can be remarkable and diagonal cracks may form in the vault ring (Figure 6). The damage is more severe for thinner vaults, like the single leaf vaults. In the case of very strong earthquake differential rocking may even cause the collapse of the vault.

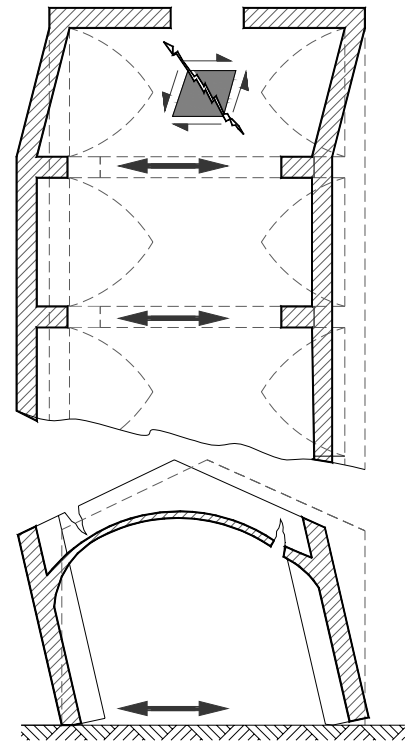


Figure 6. Crack pattern induced by the differential transverse arch rocking. Differential rocking, which is maximum in the first and last bay of the nave, is the result of the different stiffness of the transverse arches.

When subjected to earthquake loads, the nave vaults also experience differential deflection of the vault groins (Figure 7). Traditional masonry groins are connected to the perimeter walls at the impost, whereas the vault rings are built adjacently to the perimeter walls. This result in the structural discontinuities shown in Figure 7.

When subjected to the seismic actions F_0 illustrated in Figure 7, groins *a* and *b* rotates counter-clockwise around points A and B. the vault groin *a* pushes against the perimeter wall, whereas groin *b* detaches from the perimeter wall and the discontinuity width increases for increasing seismic actions. As a result, the central portion of the vault is subjected to concentrated differential deflection. The mechan-

ism is amplified by the rocking of the diaphragm arches.

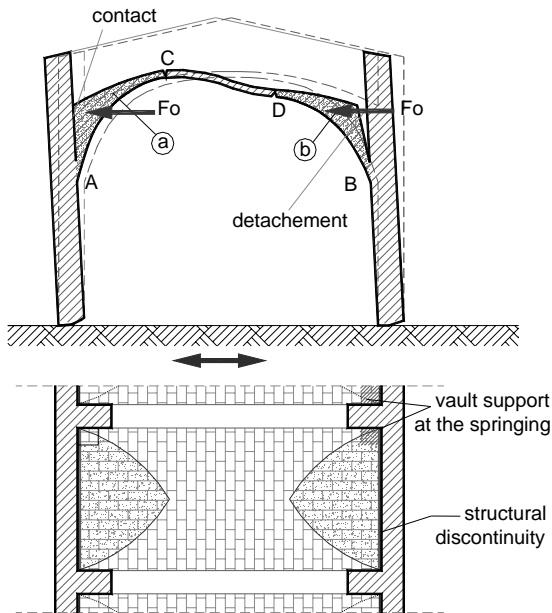


Figure 7. Differential flexural mechanism of the groins of the single leaf vaults.

In the case of very strong seismic action, cracks induced by the differential deflection may extend through the vault ring thickness causing its collapse.

Severe shear distortions induced by differential rocking and differential deflection of the vault were recognized as the major causes of total and partial collapse of the single leaf barrel vaults of the first and second bay of the main nave of San Pietro Church in Roè Volciano Brescia (Figure 8).

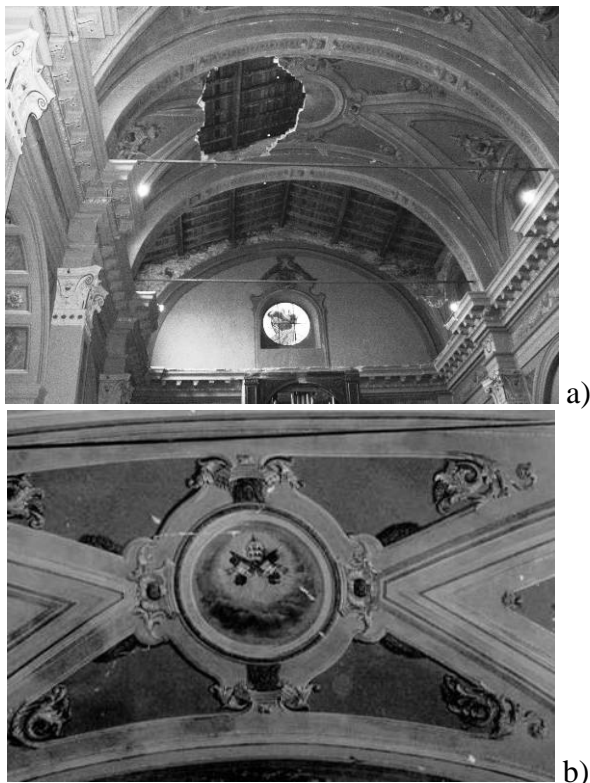


Figure 8. S. Pietro Church in Roè Vociano, Brescia, Italy: a) total and partial collapse of barrel vaults of the first and second bay of the main nave, and b) crack pattern induced by

the differential rocking and deflection of the single leaf groin vaults.

3 STRENGTHENING TECHNIQUES

In the following, a few anti-seismic strengthening techniques are illustrated. The presented techniques were adopted for the seismic upgrading of some churches after the 2004 earthquake.

3.1 Internal perimeter ties

The adoption of perimeter tie system is a traditional technique, which was effectively adopted to secure the structure against wall overturning. Historic constructions often present perimeter ties embedded within the wall thickness.

When the tie confining effect is insufficient the tie system must be strengthened or replaced. This operation is usually done by placing a new external tie system, provided that the embedment of the ties within the wall width would require expensive and difficult drilling works (Figure 9a).

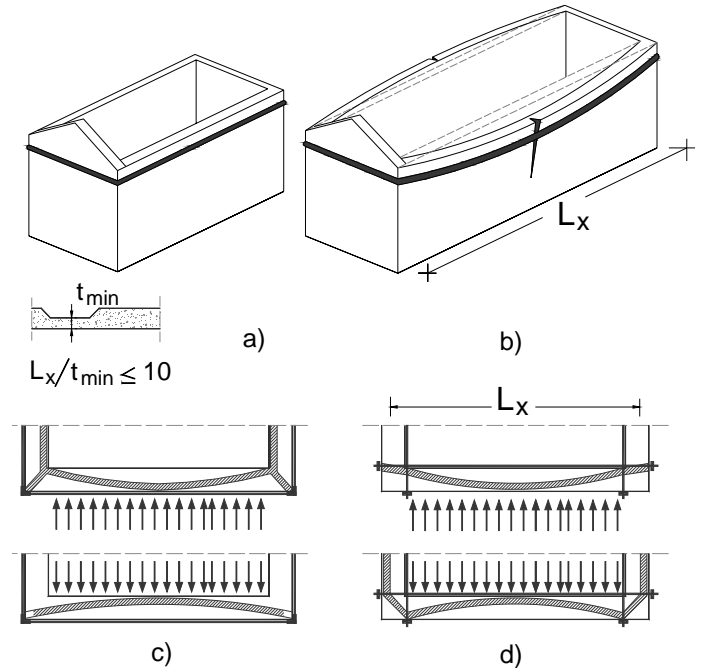


Figure 9. Introduction of internal or external perimeter ties.

The static scheme of the external perimeter tie system is illustrated in Figure 9c. For the solution to be effective, the perimeter ties must confine the resisting ideal horizontal arch, developing within the masonry wall width, at the springing. This solution was applied in San Pietro and Paolo Church, Preseglie, Italy (Figure 10).

The same confining action can be effectively obtained by adopting an internal tie system, hidden on top of the nave mouldings (Figure 9d). This solution has the obvious advantage of being hidden from the sight, but it may also serve in case of irregular or preciously decorated external walls. The confinement of the resisting natural arch can still be obtained, provided that the inclined compressed strut

forms in the masonry intersections (Figure 9d). This solution was applied in San Pietro Church, Roè Volciano, Italy (Figure 11).



Figure 10. San Lorenzo Church (Clibbio, Brescia, Italy): external perimeter ties adopted as safety emergency provisional solution following the 2004 earthquake.



Figure 11. Detail of the internal perimeter ties hidden over the nave moldings in San Pietro Church, Roè Volciano, Italy.

Perimeter horizontal steel ties are inadequate in long-span buildings lacking strong transverse arches, as the wall span-to-thickness ratio is unfavourable and little constraint is provided to the toppling masonry walls (Figure 9b). In this case, regardless of the positioning of the perimeter ties, the resisting arch is excessively low-raised and its resistance is negligible. In case of long spanned churches, roof box structure, which are described in the following, were alternatively adopted.

3.2 Roof box structures

As shown in the previous section, most of the analysed churches were found to be vulnerable with respect to overturning of the perimeter walls, as well as to excessive and to differential rocking of the diaphragm arch pillars (Table 1). In-plane shear resistant roof diaphragms, transforming the building into a box-structure, were adopted in these churches as a viable solution to avoid or limit these mechanisms (Giuriani and Marini 2006 and 2008).

Roof box structures behave like floor diaphragms, forming a sort of lit to the underlying masonries, thus preventing the perimeter wall overturning (Figure 12). The roof pitches are transformed into folded plates, which gather and transfer the horizontal seismic actions of the lateral walls, the diaphragm-arches and the roof to the shear resisting walls.

The roof box structure introduces an elastic constraint along the roof ridge, whose stiffness depends on the horizontal flexural deformability of the roof box structure (y_e , Figure 13b). The box structure can be proportioned in order to confine the maximum mid span displacement (y_e) and the masonry wall drift (β), as well as to limit the shear distortion near the supports (y_e' , Figure 13b). This way the box structure can be an effective solution to the excessive and differential rocking of the diaphragm arches.

The main structural elements composing the roof box structure are: the pitch-panel (1), the eaves chords (c_{13}), the head gables (2) (Figure14).

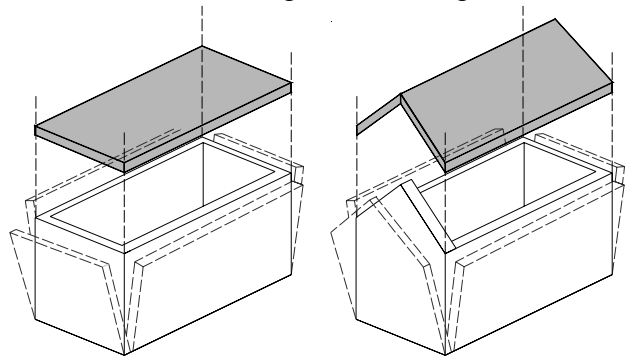


Figure 12. Floor and roof box structure preventing perimeter walls overturning.

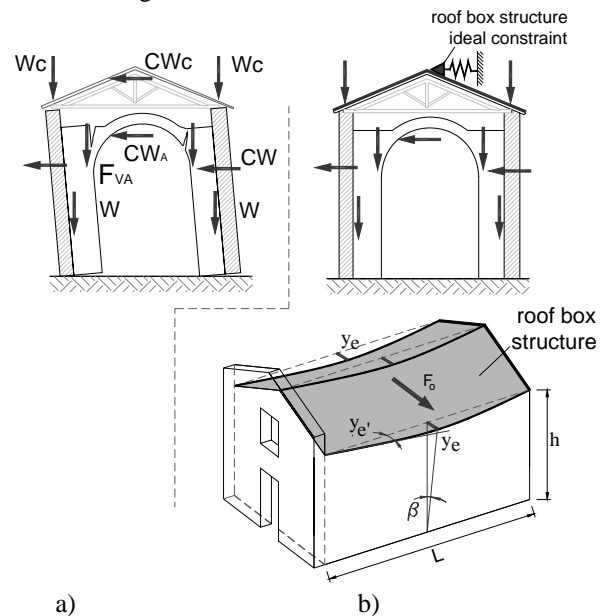


Figure 13. Structure behavior a) prior to and b) following the introduction of the roof box structure preventing excessive and differential transverse arch rocking.

In order to design the roof box structure of the analyzed churches, the same techniques usually adopted for the flexural strengthening of wooden floors, could have been addressed (Giuriani 2004).

Among all possible techniques, however, lighter solutions were preferred in order to prevent the increase of the dead loads, and thus of the seismic action.

The wooden lightweight box structure proposed in Giuriani and Marini (2008) were adopted. With the proposed technique roof diaphragms were obtained by placing overlaying plywood panels on the existing wooden roof pitches (Figure 14). Plywood panels were connected to each other by means of nailed steel flanges. The whole pitch diaphragms were nailed to the perimeter steel eaves chords, and to both roof rafters and masonry walls by means of steel studs and vertical anchored bars.

The adopted lightweight wooden box structure technique is mainly reversible and minimally impairing of the building integrity, and thus respectful of the modern restoration principles.

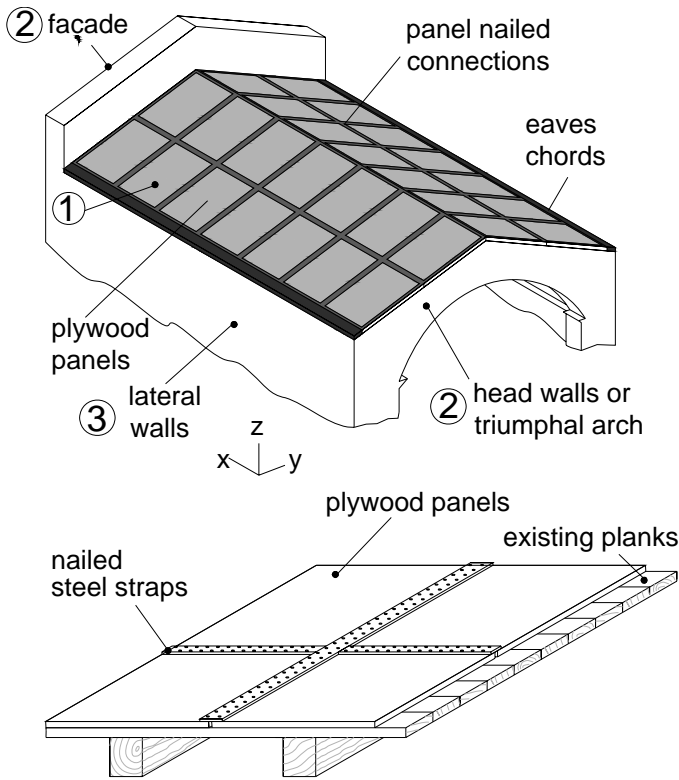


Figure 14. Wooden roof box structure by means of overlaying nailed plywood panels. Principal structural component and detail of the plywood panel assembly.

As for the structure analysis and proportioning, reference to Giuriani and Marini (2008) was made.

By reference to the load distribution shown in Figure 15, in order to analyze the roof box structure behavior, a two step approach was adopted. In the first step vertical and horizontal idealized additional constraints, having r_{1z} and r_{1y} reaction respectively, were introduced along the roof ridge line (Figure 16). In the second step these provisional constraints were removed and their reaction forces (r_{1z} and r_{1y}) were backed out.

In the first step, the unit-width stripe (a) with additional vertical and horizontal constraints, behaves

like a frame undergoing the seismic actions (case A-Figure 16). In this phase, significant uplifting vertical force per unit length n_A may arise along the top lateral wall. These possible tensile forces must be balanced by the wall self-weight, through anchored bars suitably embedded and distributed along the crowning masonries.

In the second step, the additional ridge constraint reactions were backed out with forces of equal intensity and opposite sign (f_{1y} , f_{1z} in Figure 16). Provided that Frame A would be a free mechanism with respect to the horizontal force f_{1y} , for the balance to be restored, the roof box-structure must sustain these forces (case B, Figure 16).

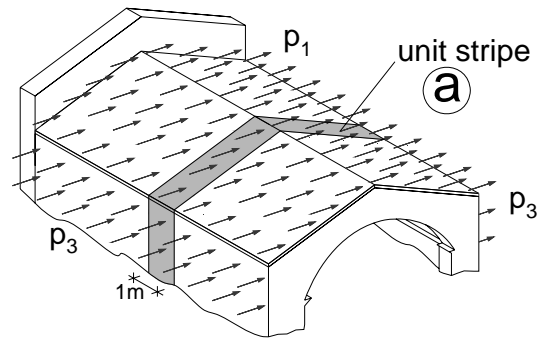


Figure 15. Seismic action distribution

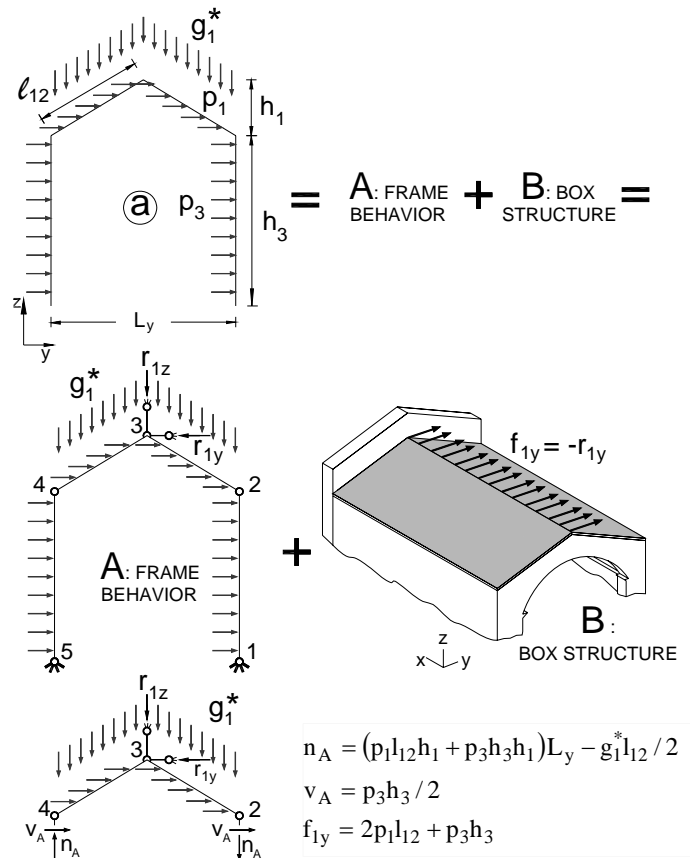


Figure 16. Structural behavior. a) Frame action and b) box-structure undergoing horizontal ridge loads.

According to Giuriani and Marini (2008), the roof box structure behavior was interpreted by reference to the classical analysis of the ribbed panels, which

is based on the assumption that the in-plane bending moment and the shear force be decoupled and resisted by the chords and panels, respectively (Bruhn 1973). Considering a gable roof, the ribbed panels box structure is assumed to behave like a simply supported beam undergoing distributed forces f_{1y} , in which the eaves chords withstand the global horizontal bending moment, and the pitch panels resist to the shear forces induced by the horizontal load (Figure 17).

The structure proportioning was based on the resistance criteria, by enforcing the elastic behavior throughout the design seismic event. It is worth noting, however, that ductility was guaranteed at the ultimate limit state by the ductile behavior of the nailed connections (Giuriani and Marini 2008), which are the weakest link of the structure.

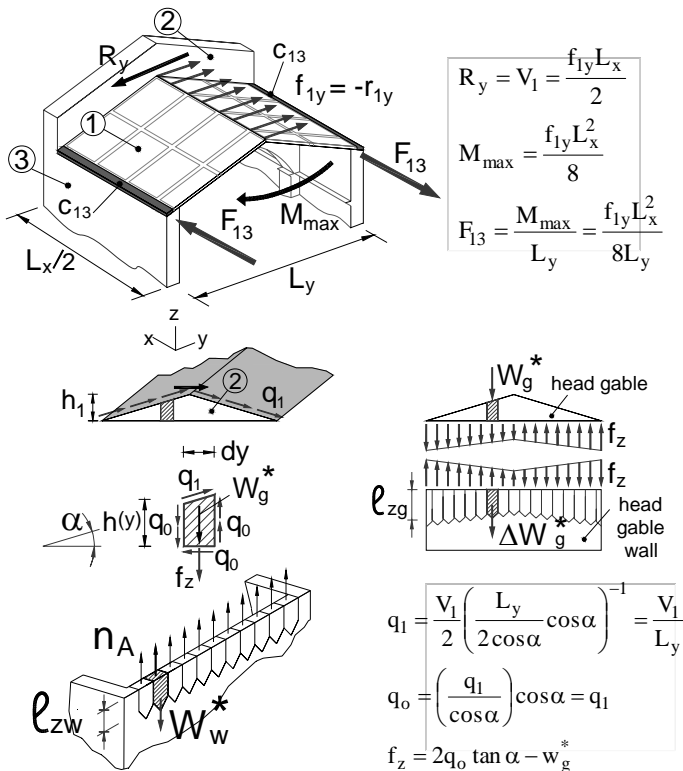


Figure 17. Simplified schemes for the evaluation of the force distribution in the box structure.

The expressions of the most significant internal forces are shown in Figures 16 and 17. The eaves chord cross section was proportioned to resist the eaves chord maximum axial forces (F_{13}); whereas the pitch panel thickness, the panel mutual connection, and the diaphragm connections to both the head gables and the eaves chords were proportioned to resist the maximum shear flow q_1 .

Horizontal and vertical steel stud connections were adopted to allow shear transferring to the shear resisting head gable and longitudinal walls, respectively (Figure 18). For the proportioning of the stud connection, mindful attention was paid to the reduced shear resistance of the masonry wall crown caused by the lack of vertical confining load. To increase the stud connection resistance, strengthening

of the masonry wall crown was sometime necessary. To this end, injections of clay mortar along the perimeter wall crown served the purpose in most cases. Alternatively, the whole masonry crown was replaced, or a thin 20÷40 mm slab of high performance clay mortar stiffened by means of plaster meshes was cast on top of the masonry walls. Experimental results proving the effectiveness of these techniques are available in the literature (Gattesco and Del Piccolo 1998; Giuriani 2004; Tengattini et al. 2006).

Distributed vertical deep anchor rebars, embedded along the head gable crowning masonries, were needed in some restoration works to confine the uplift traction forces per unit length n_A and f_z induced by the horizontal loadings (Figure 16). The tensile forces must be balanced by the self-weight of the “lifted masonry”. The distributed anchor bar embedment length was carefully proportioned for the weight of the “lifted masonry” to be sufficiently larger than the traction forces.

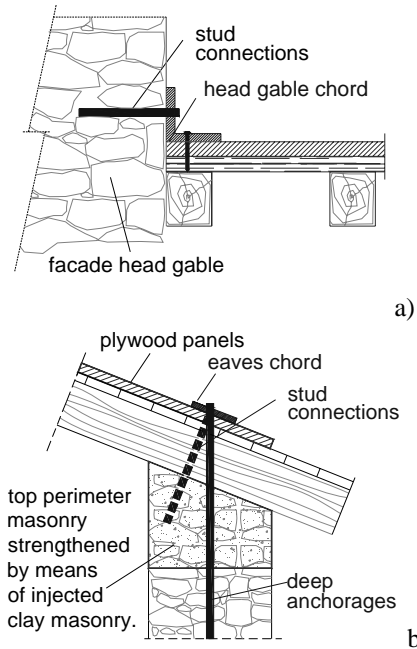


Figure 18. Detail of the connection a) to the facade head gable and b) to the longitudinal walls.

Besides the structure bearing capacity, the appraisal of the roof structure deformability was sometime necessary. Unlike perimeter walls, which can usually endure large out-of-plane displacements with little damage, excessive in-plane shear deformability of the masonry vaults and unconstrained drift of the diaphragm arch pillars were shown to be responsible of serious damages and even partial collapses in some churches. To this end, restrictions were enforced that the roof box structure lateral deflection y_e was smaller than the maximum allowable drift of the diaphragm arch pillars; and the slope of the box structure transverse deflection at the head gable y_e was smaller than the maximum shear deformation γ^* allowed by the vaults.

For the evaluation of the horizontal deformability of the plywood box structure, the analytical model based on the principle of virtual works proposed in Giuriani and Marini (2008) was adopted. In the model, in order to account for the nailed connection shear slip, which largely affect the global deformability, reduced equivalent young and shear elastic moduli are entered.

The lightweight wooden box structure applied for the anti-seismic retrofit of the San Pietro church is shown in Figure 19.

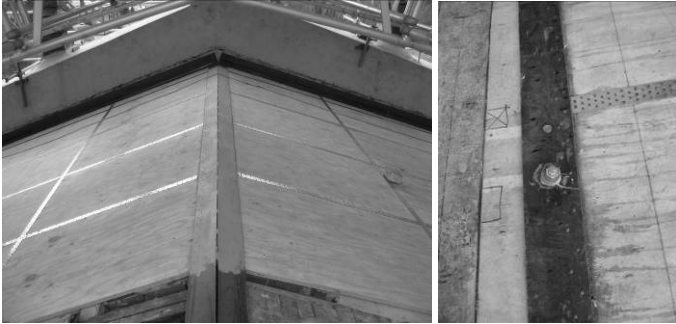


Figure 19. General view of the anti-seismic plywood roof box structure in San Pietro Church (Roè Volciano, Brescia, Italy) and detail of the eaves chord connected to the underlying masonry by means of steel studs and deep anchorages.

3.3 Light spandrel ribs

In San Pietro Church (Roè Volciano, Brescia), covered by a single leaf masonry vault, the strengthening of the structure against the differential deflection of the groin vault was necessary (Table 1).

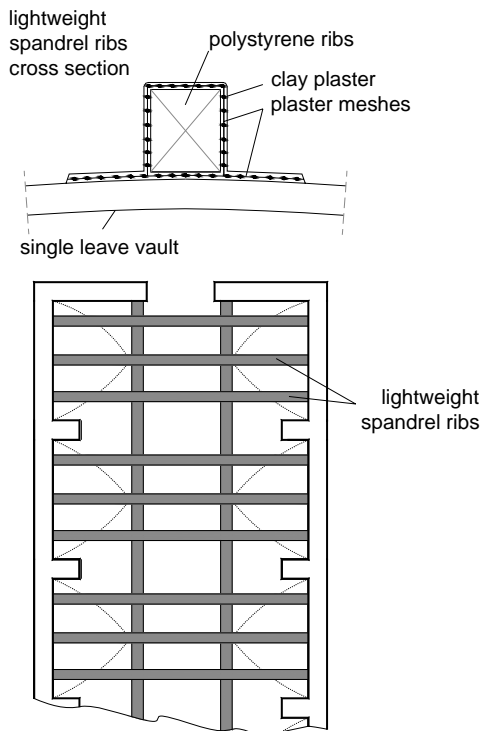


Figure 20. San Pietro Church (Roè Volciano, Brescia): distribution of the lightweight spandrel ribs along the extrados of the nave vault and detail of the spandrel rib cross section.

In masonry vaults, which are incapable to resist tensile stresses, equilibrium is guaranteed by the ideal arch developing within the vault thickness. The ideal arch corresponds to the anti-funicular of the set of applied loads. After cracking, the ideal arch must cross the solid part of each cracked sections, both at the vault key and at the springing. Single leaf vaults are usually very thin, thus the ideal resisting arch has little possibility to shift and modify within the vault thickness to adapt to different unsymmetrical load distributions. As a result, the structure is often quite vulnerable with respect to seismic load distributions, which require pronounced deviation of the ideal arch.

In order to enhance the structure resistance, spandrel masonry walls are traditionally proposed. The structure resistance is increased by increasing the thickness of the vaults, thus allowing the ideal resisting arch to adjust within the spandrel wall thickness.

In the case of single leaf thin vaults, such as in San Pietro church, attention was paid to avoid dead load increase, which in turn could result in additional seismic actions. Accordingly, lightweight spandrel ribs were proposed as a new vault strengthening technique.

Spandrel ribs were designed to resist both compressive axial forces and bending moment. This way, the flexural stiffness of the structure was greatly enhanced.

The lightweight spandrel ribs tubular cross section is shown in Figure 20. The resisting cross section is made of clay mortar reinforced with plaster meshes. The inner lightweight core is made of polystyrene elements.

In San Pietro Church, spandrel ribs were placed along the vault extrados as shown in Figure 20. The construction phases are briefly illustrated in Figure 21.

The same technique was applied to strengthen the single leaf masonry dome covering the presbitery (Figure 22).

The efficiency of the proposed technique was also verified experimentally (Figure 23). The structure was subjected to cyclic, unsymmetrical load conditions. Good results were obtained both in terms of stiffness and strength (Giuriani et al. 2007).

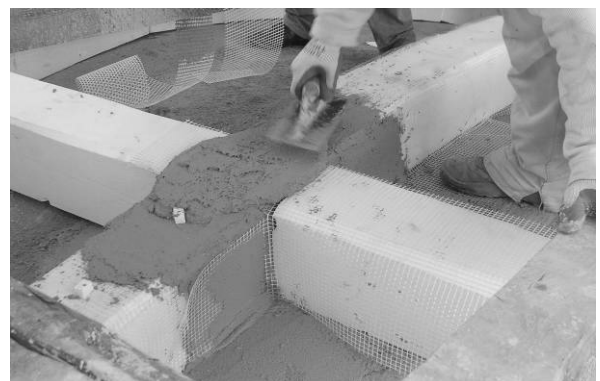


Figure 21. Light spandrel ribs construction phases: a) a layer of plaster mesh is placed overlaying the vault extrados; b) the mesh is covered with a layer of clay mortar; c) polystyrene ribs are placed along the vault ring and covered by plaster meshes; d) a thin finishing layer of clay mortar is spread over the structure.



Figure 22. San Pietro Church (Roè Volciano, Brescia): light spandrel walls strengthening the single leaf masonry dome covering the presbitero.

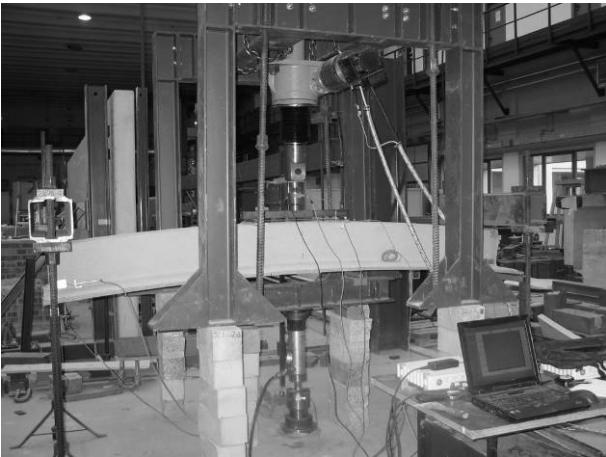


Figure 23. Experimental test on light spandrel walls (Pisa Laboratory, University of Brescia, Italy).

4 CONCLUDING REMARKS

The analysis of the damages caused by the 2004 earthquake to some churches, as well as the structural solutions proposed to reduce their structure vulnerability were discussed in the paper.

The most significant results of the study can be summarized as follows:

- In the case of full rocking, the decrease of the abutment buttress action and the increase in the ideal arch span can cause over-tension of the existing ties. In the case of very weak ties, collapse of the tie can be expected. In this case, the adoption of a roof box structure, limiting the rocking of the transverse arches, together with the strengthening of the existing tie system might significantly reduce the structure vulnerability.

- Differential rocking of transverse arch pillars induces shear distortion within the vault ring, especially in the first and last bay of the nave. Depending on the magnitude of the differential rocking, large diagonal cracks may extend over the vault leading up to its collapse. In this scenario, the structure vulnerability can be reduced by adopting a roof box structure and by limiting the shear distortion near the supports.
- In the case of a seismic event, pronounced differential deflection can be expected along the central part of thin vaults. Depending on the severity of the earthquake, differential deflection can cause the collapse of single leaf vaults. Against this failure mechanism, lightweight spandrel ribs were proposed. This solution allows to significantly enhance the stiffness and strength of the vaults.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge Claudia Porteri and Lorenzo Giuriani for their fundamental contribution to the vulnerability assessment of the churches and to the design work.

This work was developed within the research project DPC-ReLuis 2005-2008, Research line n. 1: "Vulnerability assessment and anti-seismic strengthening of masonry buildings". ReLuis and MURST financial contributions are gratefully acknowledged.

REFERENCES

- Abruzzese, D., Lanni, G. (1999). Some developments on the lateral strength of historical reinforced vaulted buildings. In *Structural Studies, Repairs and Maintenance of historical buildings, VI International Conference, STREMA*, Brebbia and Jager Ed., Dresda.
- Bruhn E.F. (1973). *Analysis and design of flight vehicle structures*. Jacobs Publishing Inc.
- Como, M., Lanni, G., Sacco, E. (1991). Sul calcolo delle catene di rinforzo negli edifici in muratura soggetti ad azione sismica. VII convegno nazionale *L'ingegneria sismica in Italia, ANIDIS*, Palermo.
- De Benedectis, R., De Felice, G., Giuffrè, A., (1993). Restauro antisismico di un edificio. In: *Giuffrè A., Sicurezza e conservazione dei centri storici: Il caso Ortigia*, pag. 189-205. Editrice Laterza, Bari.
- Curti, E., Lemme, A., Podestà, S., Risemini, S. (2006). Criteri di verifica per la progettazione di interventi di miglioramento sismico di edifici monumentali. *Ingegneria Sismica*, Anno XXIII, n. 1 gennaio-aprile.
- D'Ayala, D. and Speranza, E. (2002). An integrated procedure for the assessment of seismic vulnerability of historic buildings. In *Proceedings of the 12th European Conference on Earthquake Engineering*, London, Elsevier Science, paper n. 561 (CD-ROM).
- Doglion, F., Moretti, A., Petrini, V. (1994). *Le chiese e il terremoto*, Trieste, Edizioni Lint.
- Gattesco N., Del Piccolo G. (1998). Shear transfer between concrete members and stone masonry wall through driver dowels. *European Earthquake Engineering*, Vol.1.

- Giuffrè, A. (1993). *Sicurezza e Conservazione dei centri storici: il caso Ortigia*, Bari, Ed. Laterza.
- Giuriani, E., Gubana, A. (1993). Recupero e consolidamento di volte in muratura. In *Atti dei Colloqui Internazionali Castelli e Città fortificate, Storia Recupero, Valorizzazione*. Palmanova, 24-25 settembre.
- Giuriani, E., Gubana, A., (1995). Extradados ties for structural restoration of Vault. In *Proceedings of the Fourth International Conference on Structural Studies of Historical Buildings Strema 95*, Architectural Studies, Materials and Analysis, Vol. 1, Ed. C. A. Brebbia, Wessex Institute of Technology, UK.
- Giuriani E. (2004). L'organizzazione degli impalcati per gli edifici storici. *L'Edilizia*. Speciale Legno strutturale, N. 134. (In Italian).
- Giuriani E., Marini A., Porteri C., Preti M. (2007) "Dondolio degli archi diaframma e vulnerabilità sismica delle chiese". *Technical Report n.7*, Università degli Studi di Brescia.
- Giuriani E., Marini A., (2008). Wooden roof box structure for the anti-seismic strengthening of historic buildings. Accepted for publication in the *International Journal of Architectural Heritage*.
- Giuriani E., Marini A., Porteri C., Preti M. (2008) "Vulnerability of churches associated to transverse arch rocking". *Submitted to the International Journal of Architectural Heritage for possible publication*.
- Griffith, M.C., Magenes, G., Melis, G., Picchi, L. (2003). Evaluation of out-of-plane stability of unreinforced masonry walls subjected to seismic excitation. *Journal of Earthquake Engineering*, Vol. 7, Special Issue 1, pp. 141-169.
- Housner, G. W. (1963). The Behaviour Of inverted pendulum structures during earthquakes. *Bulletin of the seismological Society of America*, 53 (2)
- Magenes, G., Calvi, G.M. (1997). In-plane seismic response of brick masonry walls. *Earthquake engineering and Structural Dynamics*, Vol.26, pp.1091-1112.
- Marini A. and Giuriani E. 2006. Transformation of wooden roof pitches into antiseismic shear resistance diaphragms. V *International Conference on Structural Analysis of Historical Constructions*, November 6-8. New Delhi. MacMillan Ed. ISBN. 10: 1403-93156-9. Pp. 445-452.
- Lagomarsino (1998). A new methodology for the post earthquake investigation of ancient churches. In *Proceedings of 11th European Conference on Earthquake Engineering*, Balkema, Rotterdam, ISBN 90 5410 982 3
- Lagomarsino, S., Brun, S., Giovinazzi, S., Idri, C., Penna, A., Podestà, S., Resemini, S., Rossi, B. (1999). Modelli di calcolo per il miglioramento sismico delle chiese. In *Proceedings of 8th Italian Conference on Earthquake Engineering*.
- Lagomarsino, S., Podestà, S., Risemini, S., Curti, E., Parodi, S. (2004). Mechanical models for the seismic vulnerability assessment of churches. In *Structural analysis of historical construction* (C. Modena, P. B. Lourenço, P. Roca Eds.), *Proc. of IV Int. Seminar SAHC*, Padova, Italy, A.A. Balkema, London (UK), Vol 2, pp. 1091-1101.
- Tengattini C.G., Marini A., Giuriani E. (2006). Connessioni a taglio nelle murature. *Tecnical Report 3a.1-UR11-1 RELUIS* - Progetto di ricerca N.1 - Vulnerability assessment and anti-seismic strengthening of masonry buildings.