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SEISMIC INTERACTION OF TIMBER ROOFS AND SUPPORTING WALLS

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ABSTRACT

Damage or collapse of traditional timber roof structures often occurs during earthquakes, with possible extension to the underlying masonry structure. Several cases of damage involving roof structures have been observed during recent earthquakes in Italy, particularly at L'Aquila in 2009 and in the Emilia region in 2012, with particularly devastating effects on churches, where the particular configuration and large dimensions of the elements have probably worsened the effects. The collapsed roof structures were conceived as a series of parallel, interconnected trusses. Being part of historical buildings, most of these timber structures were important art works, of interest for their manufacturing and often presenting decorative works. Collapse was generally due to loss of support, which could derive from lack of local restraint, with the truss ends falling off the wall, or more often from deterioration or collapse of the supporting wall. In the work proposed here, the limit equilibrium of the truss-wall system is analysed, pointing out the main conditioning parameters of the two components. The objective is to develop synthetic rules and criteria permitting to sort out the most vulnerable cases by simple inspection and basic measuring, in order to prevent damage to buildings as well as the loss of the timber roofs, often an important part of the cultural heritage.

Keywords: *timber roof structures, timber trusses, seismic response*

1. INTRODUCTION

Within the scope of preservation of traditional timber roof structures, a particularly delicate issue is their behavior in seismic conditions. Many cases of extended damage or collapse recur in earthquakes, often involving also the underlying masonry structures and possibly evolving into total building destruction. As a consequence, during the seismic strengthening of masonry buildings the timber roof structures have been often replaced, recurring to different types and materials; at present roof structures often undergo massive interventions that are in most cases unsatisfactory from both points of view of safety increase and of preservation principles.

Yet, cases reporting the positive effect of a well-organized and well-connected roof on the global response of the building have also been observed. The point is to distinguish the conditions bringing to the two different outcomes, in order to eliminate criticalities that the timber structures may present and to enhance their positive contribution.

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To this purpose, a research program has been developed with the aim at linking the different characteristics of timber roofs to their seismic response and expressing criteria for evaluating their seismic vulnerability. On the basis of direct observations and numerical modelling, some structural characteristics, recognized as particularly influent on the response, have been selected as indicators of seismic vulnerability [1]. For each of these indicators it is necessary to define a classification criterion, associating specific values of geometric and structural parameters to a measure of their effect, e.g. [2].
The condition of the support of the truss at the end wall is a major vulnerability indicator: collapse of a roof during an earthquake develops from different causes, among which is the loss of support at the truss-masonry interface. This could derive from lack of local restraint, with the unrestricted truss sliding and falling off the wall, or otherwise from deterioration or collapse of the supporting wall. This last case is triggered by the interaction of the wall and the truss, which loads it with its inertia force generated by the horizontal seismic acceleration.
The outcome of the interaction may be negative for some ranges of the truss and wall characteristics, with the wall slenderness and the truss span length among them.
Yet, where appropriate construction and design parameters supply adequate support and a very efficient connection exists between trusses, like in the case of a light timber pent deck, the timber roof may be very effective in connecting the walls and enhancing their collaboration in the seismic response of the whole building. In this work, the limit equilibrium of the truss-wall system is analyzed, pointing out the main conditioning parameters of the two components.
The truss-and-wall collapse mechanism occurs in common residential buildings and in heritage buildings as well, but it seems to present especially devastating effects in churches, where the particular configuration of the walls and the large dimensions of the roof elements create particularly favourable conditions to its development. This study is, therefore, developed making reference to the case of churches, for which
The mechanism is a prevailing cause of damage and collapse;
– The influence of the different construction parameters, as well as their range of variability may be assessed on the basis of an extended set of cases including roofs collapsed in the Emilia earthquake;
– A church roof structure has usually a well-defined, clean layout which makes it particularly suitable to recognize and point out effects related to construction characteristics; roof structures in other buildings often present irregularities, due to modifications of the original conceptual design or to needs specific to the context, that may interfere with a systematic investigation of the main characteristics and their effects;
– The state of protected heritage has been declared in most cases for these buildings; activities like survey campaigns carried out in view of their seismic improvement in a perspective of preservation need specific tools and procedures for vulnerability assessment also for the timber structures.

Results may then be extended to other building types presenting similar features in the relationship between wall and timber structure.
The final objective is to develop synthetic rules and criteria permitting to sort out the most vulnerable cases by simple inspection and basic measuring, in order to prevent damage to buildings as well as the loss of the timber roofs, an important part of the cultural heritage.

2. OBSERVED DAMAGE TO TIMBER ROOFS

Churches are particularly prone to earthquake damage, because of their structural layout, like large spans and surfaces, tall walls, large windows, and their usually brittle construction materials. In the earthquakes occurred in Italy in the last fourty years, starting with the Friuli sequence of 1976, a fairly systematic work of damage data collection and analysis has been carried out on these buildings.
In the months immediately following the L'Aquila earthquake, out of 973 churches surveyed in order to assess the damage level, check safety, and permit or restrict occupancy, only one third, i.e. 324, were judged accessible to the public; the others had suffered damage of different level, but in any case to an extent sufficient to prevent access [3].
Making reference to the two most recent Italian seismic events, the earthquakes of L'Aquila, 2009, and of the Emilia and Po valley region, 2012, the damage mechanism that seems to have occurred most frequently in churches is the detachment and out-of-plane rotation of the façade or of its tympanum, followed in frequency by damage and collapse of the roof structure, e.g. [4], [5].

In most instances, the roof collapsed over the nave area and drew along also the nave wall. Figure 1 shows two cases occurred in the Emilia earthquake: the cathedral of Mirandola and the Church of San Michele Secchia. In the former, the timber trusses on the central nave fell over the barrel vault underneath and drew it down, sparing the nave walls. Failure has concerned apparently the restraint condition at the interface. In the latter, the involvement of the nave wall is evident, with collapse reaching the base of the clerestory.
In some cases, however, the truss-and-wall system has survived and clearly contributed to maintain the cohesion of the entire structural system. Figure 2 presents two cases: on the left side, a church of Mirandola, hit by the Emilia earthquake, where the trusses and the clerestory, with large windows, were preserved, as may be seen through the collapse of the barrel vault underneath; the right-hand-side picture is from the L'Aquila earthquake and shows a single nave where the roof structure survived and seems to have been active in maintaining connection between facing nave walls for an extended part of the nave length. The beneficial effect is partial here, but damage at the end wall and vicinity is probably due to local conditions.
In three-nave churches, the roof structures over the central nave are usually more susceptible to damage than the lower external naves, which being part of the main body of the building are more restrained to lateral displacement.
A few cases of total collapse of roof structures covering other areas, like the transept and the apse, were also reported, but they have been apparently less frequent. Roof structures in these cases may present different layouts that, together with less deformable walls, may result in reduced damage. In many cases, damage consisted in the sliding of the timber elements, tie-beams or rafters according to the roof configuration, on the support surface. The amount of displacement did not evolve into a collapse. The collapse mechanism would be different with respect to the truss and nave wall case studied here. Its characterization is currently in progress.

Fig. 1 Cathedral of Mirandola, Emilia, (left) and Church of S. Martino Secchia, Emilia (right)

Fig. 2 Mirandola, Emilia, Chiesa del Gesù (left); damage from the L'Aquila earthquake (right)

3. TRUSS AND NAVE WALL COLLAPSE MECHANISM

The evaluation of maximum capacity based on limit equilibrium is a well-established method for the local analysis of structures under lateral loads, as in the case of seismic action, e.g. [6], [7]. In spite of simplifications in the assumptions, the method supplies an assessment of the lateral force level necessary for the system to reach its limit, as a function of the parameters associated to geometry and to the various acting loads. Results are particularly meaningful for comparing situations and for identifying the most important contributions in the system.

In the analysis of the truss-and-nave wall relationship, the model may be extended to a central portion of the longitudinal walls, of length b, not significantly affected by the restraint action due to the end transversal walls (façade and transept). Windows may be present in the nave walls; in such a case, the wall length to be analyzed is included between the axes of two subsequent windows, while b is the length of the wall portion delimited by windows [8]. Figure 3 shows a three nave scheme and Fig. 4 reports the geometry of the wall portion involved in the mechanism. Note that the example in Fig. 3 corresponds to the specific layout of the church of San Biagio Amiterno, damaged in the L’Aquila earthquake in a different modality, but is representative of churches with three naves in general.

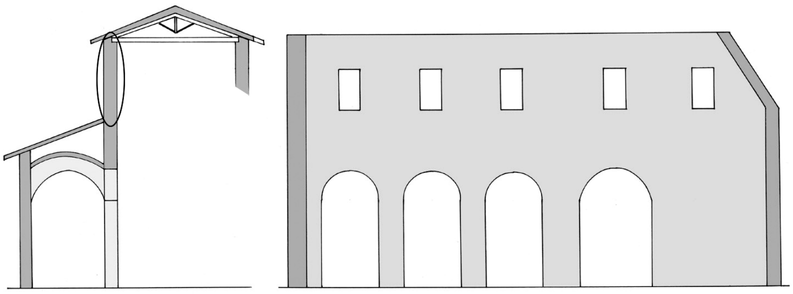

Fig. 3 Transversal section of a church (left): the central nave top wall where the out-of-plane rotation mechanism may develop is encircled in order to put it in evidence; longitudinal cross-section showing the central nave wall with openings (right)

In the limit situation, according to linear kinematic analysis, the horizontal load multiplier α_0 reaches its maximum value; correspondingly, the vertical load eccentricity is also maximum, while the stress state at the wall base implies the maximum migration of the neutral axis and the maximum value for the resistance, f_M.

The position of the centre of gravity, at mid-height of the wall section, will migrate of a usually modest amount in the presence of windows. For the analysis of the collapse mechanism, involving the wall loaded by the roof in the upper part of the nave, the limit equilibrium of a rigid body under the effect of the set of forces shown in the figure is considered,

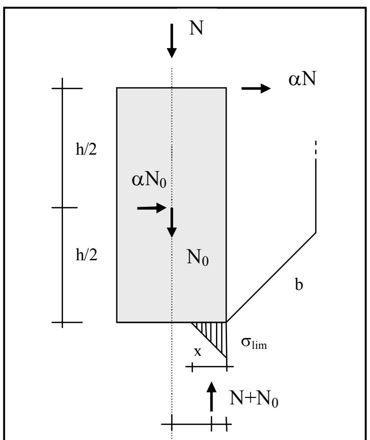

Fig. 4 Forces acting on the wall supporting the truss

Where:

- α is the horizontal load multiplier, corresponding to the horizontal structural acceleration to be reached in order to activate the mechanism;
- b, h, s describe the wall geometry (length, height, thickness);
- N_0 is the wall weight;
- N is the vertical load coming from the roof;
- f_M is the masonry compression strength;
- e is the eccentricity of the total axial load, $N+N_0$;
- x is the neutral axis depth.

The equilibrium conditions with respect to rotation and translation can be expressed, respectively, as:

$$\frac{x}{3} = \frac{s}{2} - e \tag{1}$$

$$N + N_o = b \cdot x \cdot \frac{f_M}{2} \tag{2}$$

considering that:

$$e = \frac{M}{N + N_0} = \frac{\alpha N \cdot h + \alpha N_0 \cdot h / 2}{N + N_0} \tag{3}$$

The values of the load multiplier, α, and the depth of the neutral axis, x, are unknown; by substituting equation (3) into (1), expressing x from equations (1) and (2) and equating the two expressions, α can be derived as:

$$\alpha = \frac{s/2 - \frac{2}{3}\frac{N + N_0}{b \cdot f_M}}{h} \left(\frac{N + N_0}{N + N_0 / 2} \right) \tag{4}$$

Once the value of the lateral load multiplier is obtained from a mechanism, the subsequent steps would include converting the multiplier into a spectral acceleration of an equivalent single degree of freedom system, in order to compare the result to the demand, that is, to the local design spectrum acceleration, with the purpose of safety checking. Considering that this kind of mechanism in the case of multiple nave churches occurs usually at the upper level of the building, the amplification of motion from ground level to the mechanism location should also be considered for safety checking. The purpose here is, however, that of recognizing the role and impact of different forces and contributions to the evolution of damage to the timber structure, which does not require this second step.

4. THE ROLE TRUSS AND WALL PARAMETERS

The expression of the lateral load multiplier in equation (4) suggests to investigate the role of the major parameters involved.

A first remark concerns the role of vertical loads in the equilibrium, that is, the weight of the wall and the load coming from the roof through the truss support. These vertical loads have a stabilizing effect, but considering their masses, the inertial effects will act destabilizing the system. In this perspective, the question arises on what is the impact of the roof load, namely what is the influence of the span length, to which the load is associated, and what would be the effect of an increase of roof weight due, for instance, to massive strengthening interventions.

In equation (4) the load corresponding to the wall weight, N_0, and the truss and roof weight impinging on the wall, N, appear in the expression enclosed in parentheses. This is a non-dimensional coefficient, indicated as *c* in the following, ranging between 2 if the weight of the roof is insignificant, and 1, if it is the wall weight to be negligible. Both these extreme values are unrealistic, but in any case the range of possible values is rather small. Considering the two weights comparatively, a very heavy roof would reduce this coefficient *c* to a value close to 1, which could mean cutting down to half the maximum capacity of the system.

It must be noted for completeness that the term with minus sign in the expression of α also depends on the two loads. This term, however, is rather small and with low influence, being governed numerically by the masonry compression strength, f_M.

In order to obtain some realistic measure of all these effects, a series of churches were surveyed in order to collect the basic data necessary for the analysis of the mechanism. The data sample contained churches of rather different type and size with truss span lengths ranging from 4.6 m to almost 13 m, chosen from different regions in order to satisfy statistical independence. Some of these data referred to churches actually damaged with this mechanism in the 2012 earthquake. For all these churches, the value of coefficient *c* as a function of the truss span is plotted in Fig. 5. The coefficient is almost constant, its value being in the upper range of the interval, with an average value slightly above 1.8, that is, close to the upper limit of 2. The small variability obtained from the real cases indicates a limited influence of the span dimension, and of the corresponding load, on the result.

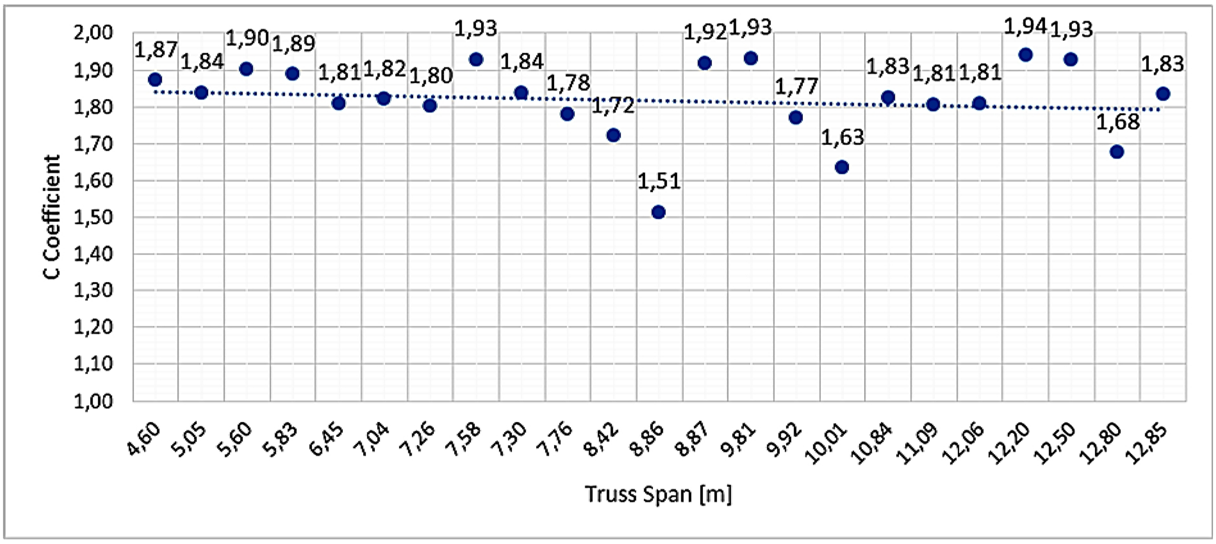

Fig. 5 Coefficient *c* versus truss span length

A correlated problem worth investigating is the influence of the span length on the load multiplier. An answer may be found in the diagrams of Figs. 6 and 7, corresponding to churches with a single nave and with three naves, respectively.

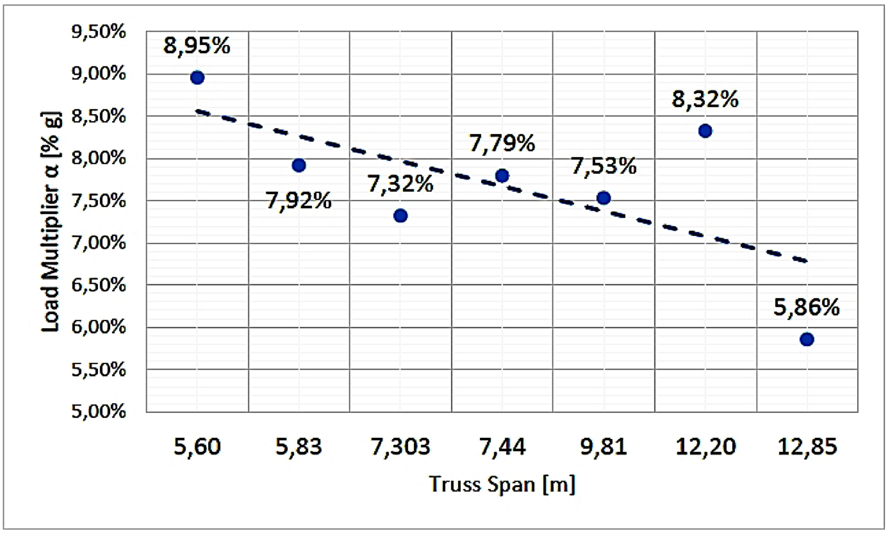

Fig. 6 Horizontal load multiplier values for single nave churches

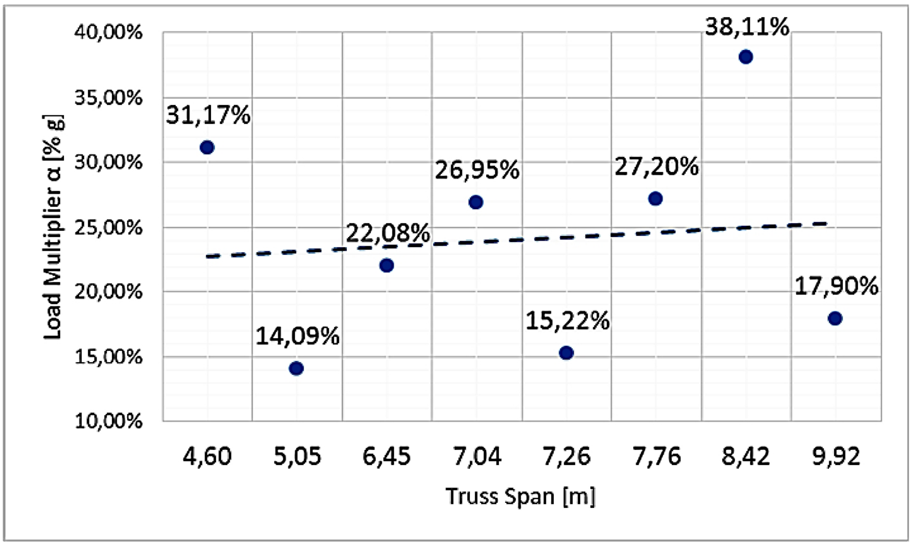

Fig. 7. Horizontal load multiplier values for churches with three naves

For single nave churches and with a range of span lengths between approximately 5 to 13 m, there is a mild dependence of the multiplier from the span, decreasing for greater lengths. Values of α are rather low, ranging between approximately 6 and 9 percent.

Considering the central nave top of churches with three naves, results are more spread, but values appear to be more favorable. The span length tends to be shorter. In any case, even looking at the minimum values obtained, the value of the multiplier is more favorable than for the previous set: the minimum here is about 14%, while the mean value in the single nave cases was about 7.7 percent. This comparative result is shown in Fig. 8.

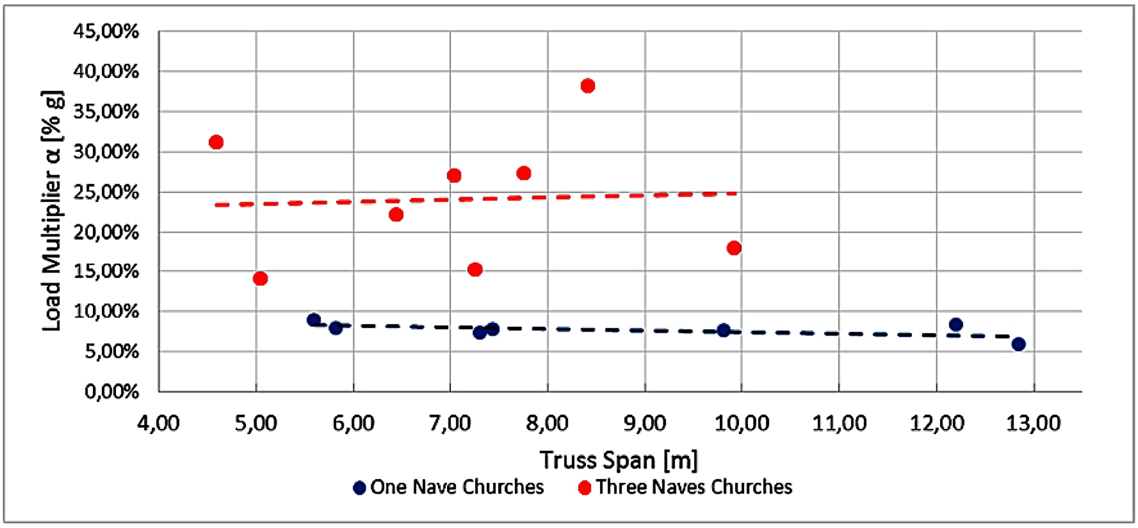

Fig 8. Comparison of horizontal load multiplier values for single nave and multiple nave churches

Some further consideration emerges considering once more equation (4). For an "order of magnitude" evaluation, considering the coefficient *c* close to 2, as from previous results, and neglecting the comparatively small negative term, the expression for the multiplier would reduce to

$$\alpha \cong \frac{s}{2h} \cdot 2 \cong \frac{s}{h} \tag{5}$$

This expression puts into evidence the importance of the wall slenderness, which can be measured by the aspect ratio *h/s*. This very simple reduced formula would be compatible with use for synthetic evaluation of vulnerability. In any case, it accounts for the finding that for single nave cases, where the wall height is usually greater than for the top wall of the central nave, load multiplier values were lower. It must be noted, however, that in terms of acceleration for the central nave case, where the mechanism develops in the upper part of the building, the amplification of motion due to elevation will be an additional factor affecting negatively the behaviour of the system.

4. CONCLUSIONS

In order to protect traditional timber roof structures from earthquake damage, the collapse mechanism that occurs most frequently, involving the roof truss and the wall underneath, has been studied. The case of church roofs, where it often develops, has been investigated with reference to the cases of single nave and multiple nave churches. Considerations on the role of the truss span and on the supporting wall geometry have been developed on the basis of data from a sample of these buildings, including some churches actually damaged in this way during recent earthquakes. In the real situations considered, the slenderness of the supporting wall, defined as the ratio of depth to thickness, has a dominant role in determining the maximum lateral load that the system can bear. This quantity may be easily estimated also in a rapid survey and could, thus, be used in vulnerability inspections.

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REFERENCES

[1] Parisi M.A., Chesi C., Tardini C. (2010) Seismic vulnerability indicators for timber roof structures. In: Proc. *9th US National and 10th Canadian Conference on Earthquake Engineering*, Toronto, Proceedings Volume 1: 157

[2] Parisi M.A., Chesi C., Tardini C. (2013) Seismic vulnerability of timber roof structures: classification criteria. *Advanced Materials Research* 778: 1088-1095

[3] Parisi M.A., Piazza M., Chesi C. (2011) Seismic response of traditional timber elements and roof structures: learning from the L'Aquila earthquake. In: Proc. of *The First International Conference on Structural Health of Timber Structures*, SHATIS; Vol. 1: 064, Lisbon

[4] Cattaneo S. (2014) *Vulnerabilità sismica delle chiese in muratura: il meccanismo di collasso per riposta trasversale dell'aula*. Master Thesis, Politecnico di Milano

[5] Falconi C., (2014) *Studio del comportamento sismico delle chiese: analisi di interazione tra copertura e muratura nel meccanismo d'aula*. Master Thesis, Politecnico di Milano

[6] Lagomarsino S., Podestà S. (2004) Seismic vulnerability of ancient churches: part 1. Damage assessment and emergency planning. *Earthquake Spectra* 20(2): 377-394

[7] Lagomarsino S. Podestà S. (2004) Seismic vulnerability of ancient churches: part 2. statistical analysis of surveyed data and methods for risk analysis. *Earthquake Spectra* 20(2): 395-412

[8] Parisi M.A., Chesi C. (2014) Seismic vulnerability of traditional buildings: the effect of roof-masonry interaction In: Proc. *Tenth U.S. National Conference on Earthquake Engineering*, July 21-25, Anchorage, Alaska