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SEISMIC VULNERABILITY OF TIMBER ROOF STRUCTURES: AN ASSESSMENT PROCEDURE

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Abstract

The objective of synthetic procedures for assessing the seismic vulnerability of structures is to sort out by inspection and limited elaboration the characteristics and critical features that may induce an inadequate seismic response. A procedure for assessing the vulnerability of timber roof structures, which are present in the majority of traditional masonry buildings in many countries and highly condition the building response, has been the object of a long-term research project and has been developed as follows:

First, a visual inspection is performed, according to a predefined sequence of operations with the guidance of a form, or template, to be filled. Visual inspection is the basis of any in situ assessment of timber structures, aimed at providing information about their structural soundness. In this case, focus is on the characteristics that influence most strongly the response to seismic action. Information is collected following the template's tree-like structure, with branches following in increasing detail the different aspects to be considered. The exam goes from the description of the structural configuration down to individual timber members and to joints and construction details, covering geometry, the materials, and the state of conservation and including possible modifications of the original layout.

In a second step, this information is used for checking and grading selected structural characteristics that condition the seismic response and can be considered as vulnerability indicators. Four vulnerability classes, from A to D, have been defined for grading purposes, from the lowest vulnerability level, equivalent to a satisfactory situation, to the highest one, requiring thorough strengthening interventions.

Main indicators are,

The conceptual design: usually, roof structures were conceived for carrying vertical loads; their capability to respond to horizontal actions like earthquake-induced inertia forces is often less developed depending on the structural typology; correct sizing of elements is also considered here;

The carpentry joints, which must maintain connection in dynamic conditions;

The system of constraints between timber structure and supporting walls, which is critical because loss of support often triggers progressive failure of the whole building system;

The state of the structure: this point includes any condition specifically related to the current situation of the structure that may affect vulnerability, including the state of conservation and previous strengthening interventions, if any.

A particularly critical issue is the effectiveness of supports. Its evaluation has required developing specific models and synthetic parameters suitable for use in vulnerability assessment.

Finally, the implementation of the procedure template as a software tool is under way.

Keywords: timber structures; timber roofs; seismic vulnerability

1. Introduction

The observation of the large amount of damage in traditional masonry structures related to failure of the roof system has motivated the development of a long term research project devoted to timber roof structures under seismic action. The idea was to investigate vulnerability causes and to devise simple strategies of improvement that could transform the often negative interaction of timber structures and masonry walls into a favorable behavior, with the roof acting as a link between walls and enhancing their collaboration; besides the problem of wall-roof interaction, failure could occur in the timber structure alone, also with serious consequences.

The definition of criteria to assess vulnerability of timber roof structures and a practical procedure implementing it have been the main research objectives. A previous research project on the mechanical characterization of traditional carpentry joints and on techniques for their strengthening had pointed out the requirements for a satisfactory behaviour of joints in the presence of seismic action and constituted a basis for the current project when dealing with this issue [1, 2].

Initially, a procedure was outlined by sorting out the main structural characteristics that would produce a low or high vulnerability level and by defining corresponding grading criteria. In a first version, [3], the focus was particularly on the quality of connections and on the structural typology [4].

Further work, following also some damage observations after the earthquake of L'Aquila, Italy, 2009, concerned the current state of the structure as an indicator of vulnerability. The effectiveness, or at times the negative impact, of modifications of the original structural concept was considered at this point. Some of these interventions were performed for seismic strengthening, following criteria that are now obsolete [5].

In this last part of the work, the restraint condition of the roof truss and the consequent interaction with the wall supporting it have been studied. A simple mechanical model and the analysis of a large number of real structures that included cases of damage occurred in the Emilia, Italy, 2012 earthquake, have indicated a parameter that could be used to assess vulnerability.

Additionally, passage from the original paper template for data collection to a computer-based form that should be more practical during surveys has started and is under way.

2. Damage patterns in roof structures

Damage data of timber roof structures after earthquakes are limited, in spite of the frequency of damage occurrence. When damage involves the walls in a global collapse, or when the falling roof engages the lower floors in a progressive collapse, causes and results appear evident; when damage is limited to the roof or to some of its components, it is seldom analyzed and reported in detail, in part for the difficulty of reaching the structure and examining it from close.

The main damage mechanisms, however, have been identified, and may be summarized as follows:

- collapse of the roof falling off the support in a rigid body mode, with possible involvement of other structures below, like floors or vaults;
- collapse of the roof and of the supporting wall, by interaction of the two elements; this collapse is frequent in churches and involves especially the roof trusses and nave walls;
- damage, and possibly collapse, of the timber structure due to overstressing of members;
- damage, and possibly collapse, due to inadequate joints, typically with disconnection of joints.

Failure of timber elements is usually of brittle type; often the consequences extend to the whole structure; loss of connectivity in a joint is equally dangerous, implying a sudden capacity loss.

3. Seismic vulnerability assessment

The objective of synthetic procedures for assessing the seismic vulnerability of structures is to sort out by inspection and limited elaboration the characteristics and critical features that may induce an inadequate seismic response.

Vulnerability is generally expressed with respect to a reference structure that is considered capable of a satisfactory seismic response. Considering the damage patterns above, a low-vulnerability roof structure should have,

- no unrestrained thrusts;
- a suitable conceptual design, capable to sustain also horizontal forces and offering comparable capability in different directions;
- effective connections to the supporting walls;
- suitable carpentry connections; some metal closure should be present to avoid disassembly;
- element cross-sections sufficient for the stress increase caused by seismic action, and
- good maintenance conditions.

The assessment procedure defined with reference to these points consists of two parts:

1. A visual inspection performed with a predefined sequence of operations, implemented in a form, or template, to be filled with the collected data and information. For timber structures, visual inspection is always the basis of any in situ assessment, aimed at providing information about their structural soundness. In this case, focus is on those structural features and material properties that influence most strongly the response to seismic action. Inspection is performed following the template's tree-like structure, with branches following in increasing detail the different aspects to be considered. The procedure starts from the description of the structural configuration down to individual timber members and to joints and construction details, covering geometry, the materials, and the state of conservation and recording modifications of the original layout.

2. The collected data and information are used for checking and grading structural characteristics that affect the seismic response and can be considered as vulnerability indicators.

At the end of this process, the set of class values assigned to the indicators shows the severity of the situation, pointing out the criticalities. A final judgement for the whole structure may be expressed from the results of partial indicators. This last step may also be formalized with combination rules. Such result is to be considered as a support for the final decision to be taken by the professional examining the structure.

Vulnerability grading is expressed with linguistic variables: four classes, from A to D, have been defined. The lowest vulnerability level, equivalent to a satisfactory situation, corresponds to grade A; the highest one, likely to require thorough strengthening interventions, is class D. Intermediate levels are B and C. The main indicators are,

- The conceptual design: roof structures were mostly conceived to bear vertical loads; their capability to sustain horizontal actions like inertia forces in some structural typologies is often low; correct sizing of elements is also considered here;
- The carpentry joints, which must maintain connection in dynamic conditions;
- The system of constraints between timber structure and supporting walls, which is critical because loss of support often triggers progressive failure of the whole building system;
- The state of the structure: this point comprises any condition specifically related to the current situation of the structure that may affect vulnerability, including the state of conservation; previous strengthening interventions, when applicable, are classified here.

Criteria for classification for each indicator have been formulated; corresponding grades are synthesized in reference tables [6]. These are, again, a guide for judgement. The analysis of case studies and numerous applications have been performed, supplying useful indications for refining definitions and grades and improving the assessment process. Calibration with new cases is still continued.

The procedure is particularly tailored for roof structures that are most common in Italy, composed of parallel trusses linked with different kinds of elements (simple purlins, diagonal bracings, secondary trusses…), limited roof inclination, and trusses that are mainly of simple king-post type, often with struts, for regular span sizes and more complex configurations for longer spans.

In order to classify the global vulnerability of a roof structure, the possible consequent damage may be considered. For instance, if high vulnerability in a partial indicator may result in a risk of global collapse also of the building (e.g. roof failure causing progressive collapse of underlying structures) the global index would be very high, D; high vulnerability, C, will correspond to the risk of partial collapse of the roof and local damage to the building structure, a B grade will correspond to the risk of damage to the truss system, with limited consequences for the building. This concept has been applied in various cases and seemed valid for a synthetic judgement.

4. The development of indicators

The most demanding task in setting up a vulnerability procedure is in defining meaningful indicators, detailing the corresponding critical conditions, representing them with parameters that are easily recognizable or inferable from survey data, associating ranges of variation and classes. For the various indicators, reference tables were prepared, together with some guidance from example cases. A summary of this work follows.

4.1 The structural typology

This indicator is particularly important, because roof structures were usually conceived to support vertical loads, with little or no consideration of horizontal actions. Structural typologies valid for vertical loads may differ in their response to seismic action. The dynamic and seismic analysis of a very large number of example cases, with parametric studies of the main components, has given the possibility to define ranges of response associated to the geometric configuration and other design parameters.

The level of interconnection between trusses gives a first classification criterion summarized in Table 1, part A. For trusses that are well interconnected, the association among the span length, the cross section size of the main members, especially chord and rafters, and the typology of the truss has been analyzed. It has to be mentioned that different truss types are best suited for specific span ranges, as recognized in the timber construction tradition. Results obtained have been the basis for proposing a classification of design parameter combinations, as reported in Table 1, part B, where the shorter spans correspond to a king post truss scheme and the larger to a queen post one, more suitable for the truss size. In some surveys, cases with clearly underdimensioned cross-sections or structures presenting conceptual errors in terms of statics are occasionally met. These cases are not listed in the table and are classified as D [4].

It is interesting to note that the lowest vulnerability levels resulted for structures where sound principles of traditional constructional practice had been rigorously applied.

Table 1. Classification of conceptual design

<table>
<tr><th colspan="2">A) Effect of interconnection</th><th colspan="6">B) Effect of dimensions and type</th></tr>
<tr><th colspan="2">Structural scheme</th><th rowspan="2">Section
[cm × cm]</th><th colspan="5">Span [m]</th></tr>
<tr><td>Trusses in orthogonal directions</td><td>A</td><td>6</td><td>9</td><td>12</td><td>18</td><td>24</td></tr>
<tr><td>Parallel trusses with transversal bracing</td><td>A – B</td><td>15 × 15</td><td>A</td><td>B</td><td>C</td><td></td><td></td></tr>
<tr><td>Parallel trusses with at least 2 purlins per pent</td><td>A – B</td><td>20 × 20</td><td></td><td>A</td><td>B</td><td>C</td><td>C</td></tr>
<tr><td>Parallel trusses with 1 purlin per pent</td><td>B – C</td><td>25 × 25</td><td></td><td></td><td>A</td><td>B</td><td>B</td></tr>
<tr><td>Couple roof (no truss)</td><td>C – D</td><td>30 × 30</td><td></td><td></td><td></td><td>A</td><td>A</td></tr>
</table>

4.2 Carpentry joints

The experience acquired and the results from the previous research project on carpentry joints have supplied a basis for assessment. In seismic conditions a joint must not disassemble, continuing to operate and transmit forces by direct wood contact; it must not fail in a brittle mode, and should provide dissipation to the best possible extent. Metal connectors preventing joint opening in extreme conditions must be present and efficient; yet, some deformation and rotational capability of the joined members must remain, in order to accomodate wood deformation and avoid brittle behavior. To this end, reinforcement fully enclosing the joint, like metal cuffs, must be avoided, but also the positioning of a small amount of connectors must not result in excessive stiffness, fig. 1. Table 2 summarizes the indications for a step joint (birdsmouth joint) with different types of reinforcement. Reference is made particularly to step joints, which have been largely investigated, although other types are also treated. The criteria valid for these joints may guide decisions for other configurations not specifically covered.

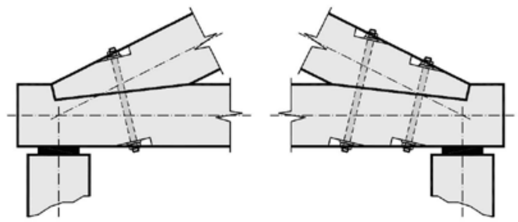

Figure 1. Rafter-to-chord joint with different reinforcement arrangement, the insertion of connectors aligned transversally (on the left) gave better results; the layout on the right limits the rotational capability.

Table 2. Classification of carpentry joints: the rafter-chord node

Reinforcement type		class
Unreinforced, no provisions for disconnection		D
Reinforced, with		
	1 bolt	B
	≥ 2 bolts, small diameter,	
	transversal	A
	longitudinal	C
	Stirrups	C
	Binding strip	
	fixed	B
	adjustable	A
	Steel cuff	D

4.3 The state of the structure

Many different issues are to be examined in order to judge the current state of the structure in the perspective of vulnerability assessment, as shown in table 3. Among them is material decay, for instance due to biotic attack, that should be detected at visual inspection time. Particularly in humid environments fungi may reduce wood element capacity and deteriorate the condition at supports.

Within this indicator, also the interventions that have modified the original structure are considered. These may have remediated initial construction errors, occasionally present in structures that were built based on carpenters' experience and tradition rather than engineered design, or on the contrary may have been performed for a variety of non structural reasons, often with negative effects on safety. In fig. 3, at right, cutting the truss chord and blocking the stump at the floor with a probably ineffective metal hoop is highly questionable.

Table 3 - State of the structure

Item		**Class range**
maintenance		
	roof cover	C-D
	date of last general maintenance	B-D
decay		
	reduction of element sections	B-D
	decay of joints	B-D
Previous interventions		
	modification of elements	B-D
	Increased loads	C-D

Interventions may have been specifically intended for seismic strengthening. Some seismic strengthening operations performed in the last part of the 20th century according to design codes and guidelines of the time were often increasing excessively masses and stiffness. In fig. 3, at left, a heavy brick and concrete slab was laid over the roof truss to increase link between facing walls, which created problems during the L'Aquila, 2009, earthquake. Other cases of strengthening, performed with low impact interventions, may be found. The former, during recent earthquakes, have often generated damage; the latter have shown positive effects. Interventions are, therefore, quite difficult to evaluate in terms of vulnerability assessment. Detailed study of specific cases has been performed, as a guidance for classification decisions [5].

Figure 3. Results of interventions; a massive intervention over a roof truss (left) [5]; the chord of a truss was cut for unknown reasons jeopardizing safety (right)

4.4 Conditions of supports

Making reference to the damage patterns described at section 2, the importance of the support conditions on the outcome in an earthquake stands out clearly. A first classification was proposed, with indications for grading that were basically referred to the effectiveness of restraint at the truss chord end. This covers the case of fall of the truss as rigid body, but not as well that of damage resulting from the interaction of the truss with the wall. In order to give indications for assessing vulnerability with respect to this situation, further study has been necessary. The results are described in the following section.

5. The truss and wall interaction

A critical issue that needed investigation was the effect of the type of support on the seismic response of the timber roof trusses. Often roof collapse may be ascribed to a loss of support due to many causes. The limited extension of the support area, the decay of timber, which is not rare when the chord end is restricted in a humid environment with insufficient ventilation, and other adverse conditions of the chord extremes may foster collapse of a truss as a rigid body. Yet, the case is more complex when part of the supporting wall is involved in the mechanism. The collapse mechanism that develops with the interaction of the roof structure with the wall has been examined in order to determine the design conditions for which collapse may occur. The aim has been at expressing a synthetic parameter based on visual inspection and simple measuring that may be associated to the vulnerability of the system.

Because failures of this type have been very frequent in nave walls of church buildings during recent earthquakes in Italy, fig. 4, the study has been performed on this kind of structures. Results may then be extended to other truss-wall situations in similar geometric conditions.

The approach adopted has been to study the limit equilibrium of the wall portion (the upper part of the nave or the full wall depth according to geometry and possible restraints from other building parts) and to obtain the lateral load multiplier [7]. 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. [8, 9] and is now part of the analyses commonly carried out for the verification of masonry buildings. In spite of simplifications in the assumptions, the method gives a measure of the lateral force level by which the system reaches its limit, as a function of the parameters associated to geometry and to the acting loads. Results are particularly useful for comparing situations and for identifying the most important contributions in the system.

The model of the truss-and-wall system, shown in fig. 5, may be extended to a central portion of the wall, of length b in the figure, not significantly affected by the restraint action due to the end transversal walls (façade and transept in the case of churches).

Figure 4. Partial or total collapse of roof structures in churches during the 2012 earthquake in Emilia, Italy [7].

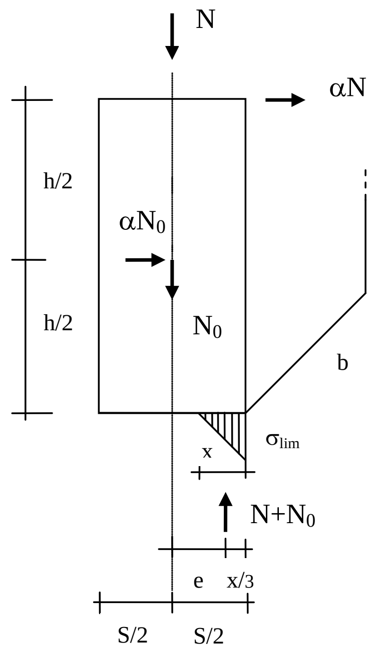

Figure 5. Limit equilibrium conditions of a wall section under a roof applying load N, [7].

In the situation of the figure, the first order limit equilibrium supplies the horizontal load multiplier as from eq. (1), where symbols are as in fig. 5.

$$\alpha = \frac{\frac{s}{2} - \frac{2}{3}\frac{N+N_0}{b \cdot f_m}}{h}\left(\frac{N+N_0}{N+{N_0}/{2}}\right) \qquad (1)$$

Considering the very small value of the subtractive term in (1) in realistic cases [6], for practical purposes the formula may be simplified as

$$\alpha = \frac{{s}/{2}}{h} \times c \qquad (2)$$

with parameter *c* summarizing the effect of weights from the truss and from the masonry element

$$c = \frac{N+N_0}{N+{N_0}/{2}} \qquad (3)$$

Expression (2) promises to be very useful for fast, synthetic estimations, as long as a simple way of expressing coefficient *c* may be found. Loads depend on wall geometry and material, type of roof cover over the truss, span length, etc. Coefficient *c*, consequently, ranges between 1 and 2 depending on whether the weight of the roof or that of the masonry dominates. Roof loads are, however, basically of two types: regular loads amount to about 2 kN/m^2, while heavy roofs like those that were strengthened as in fig. 3 weight about 5 kN/m^2. With this reference, a series of cases has been examined. Results from the lighter, and more common, roofs, yielded a coefficient *c* with an average value of 1.83 and very low variability, as in fig. 6; heavier roofs were clustered around *c* = 1.33 with equally low variability. Therefore, as a first approximation and for the case of church roofs, eq. (2) requiring knowledge of only a couple of geometric dimensions and of the load type, offers the possibility of a simple, first estimation of the load multiplier value.

Associating the estimated load multiplier to a vulnerabililty condition of the roof support that could be adopted in the vulnerability procedure above, and checking the validity of results, with possible modifications for buildings other than churches is the next step, for which work is in progress. Possible variations in the position of loads and other effects have also been examined.

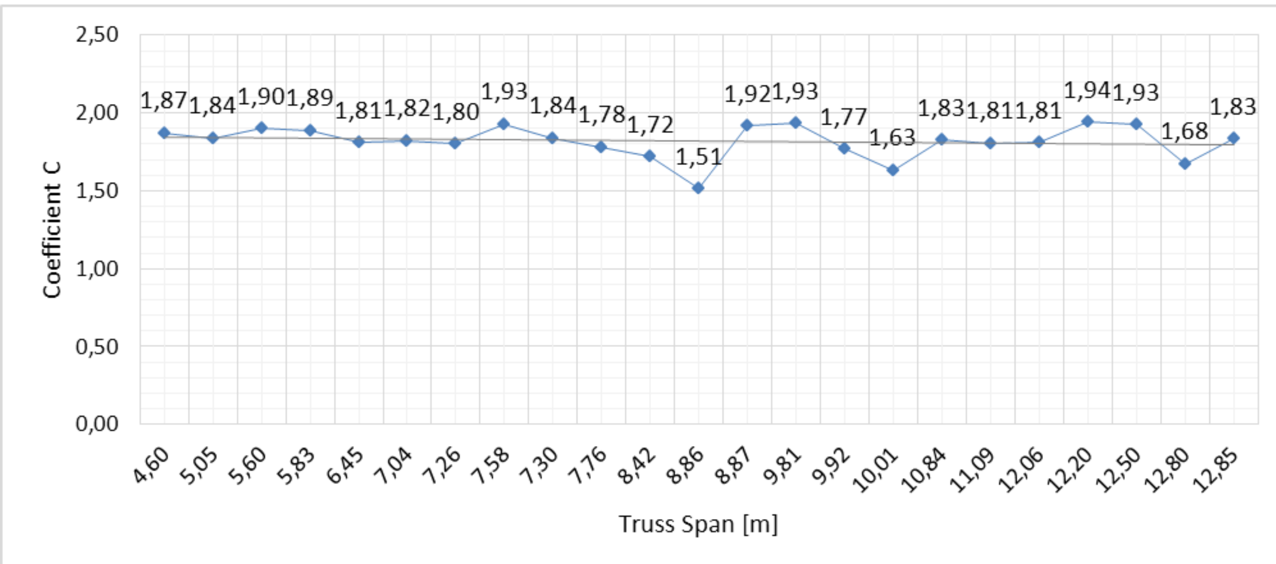

Figure 6. Coefficient *c* values obtained from a series of case studies.

6. Data collection modalities

The form developed to collect data and guide the vulnerability assessment operations was originally on paper for practical reasons related to the survey conditions. Such paper forms have been used so far in the case studies and applications, with some inconvenience in the inspection of large structures requiring to manage bulky form packs in inhospitable environments.

Given the fast development of highly portable digital tools, the procedure is now being implemented in a software system for portable computers, which also offers the advantage of data digitalization on site and the possibility of a more efficient management of branching. It is worth noting that new possibilities offered by mobile technology have encouraged various projects for computer-based assessment surveys. Making reference to timber structures only, the European Union COST project FP(Forest products) 1101 has proposed and defined a computer-based template for the assessment of the conditions of timber structures, implementing a first version [10]. A template for damage assessment of timber structures of modern type was proposed in [11]. The Mondis project [12], not only for timber, aims at implementing a tool for on-site monitoring of monument damage using mobile devices.

Given the expected field of application, the implemented questionnaire, described above, is currently in Italian. Figure 7 reports part of the original paper form, fig. 8 show screen images of the computer-based version, related to the same points.

1. BUILDING NUMBER				
2. LOCATION				
ADDRESS				
SEISMIC ZONE	PGA			
3. TIME of CONSTRUCTION				
4. TYPE OF OCCUPANCY	4.1 Residential			
	4.2 Industrial			
	4.3 Commercial			
	4.4 Agricultural			
	4.5 Public Buildings			
	4.6 Other			
5. DIMENSIONS and GEOMETRY	5.1 Symmetric		y	n
	5.2 n° of axes of symmetry			
	5.3 Plan regularity		y	n
6. ELEVATION	6.1 N° of stories above ground			
	6.1.1 timber stories:			
	6.2.1 Regularity in elevation		y	n
	6.2.2 Cantilevers and setbacks:			%
7. MATERIAL	7.1 Main structure	7.1.1 masonry		
		7.1.2 r.c. frames		
		7.1.3 steel		
		7.1.4 timber		
		7.1.5 other:		
	7.2 Infills	7.2.1 masonry		
		7.2.2 wood boards		
		7.2.3 gips carton		
		7.2.4 other:		
8. OPENINGS	8.1 Vertically aligned		y	n
	8.2 opening ratio:			%
9. STRUCTURAL ELEMENT DIMENSIONS	9.1 bearing wall thickness (or frame elements dimensions)			
10. ROOF TO WALL CONNECTION	10.1 Constraint type	10.1.1 Simple support		
		10.1.3 Hinge		
		10.1.4 Semi -rigid		
		10.1.5 Built-in		
		10.1.6 Other:		
	10.2 Wall plate	DESCRIPTION:		
	10.3 Ring beam	DESCRIPTION:		

Figure 7. Original paper template

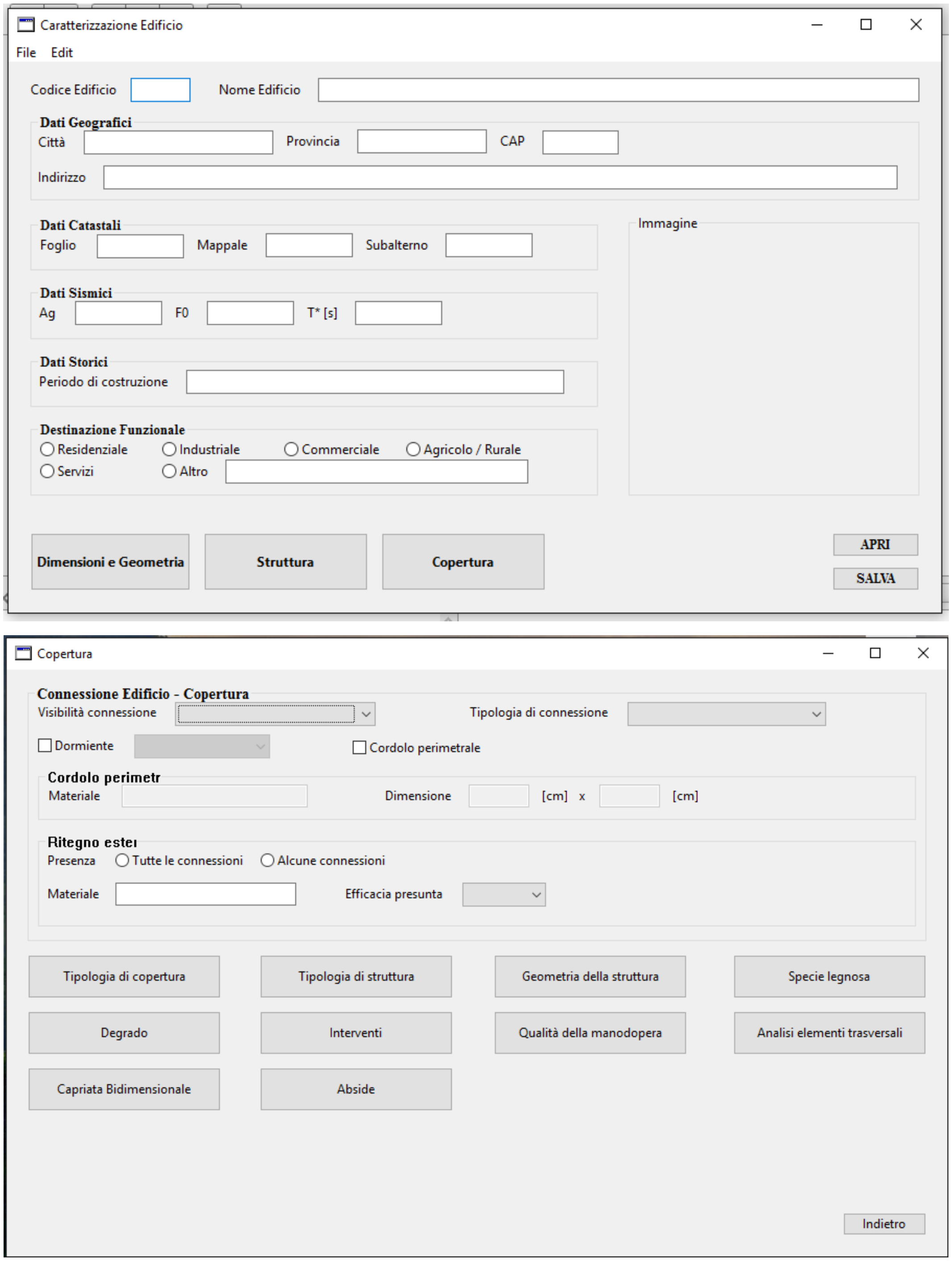

Figure 8. Computer-based template

7. Conclusions

A research program has been devoted to define and implement a procedure for assessing the seismic vulnerability of timber roof structures. A series of vulnerability indicators representing the main factors that affect the seismic response have been identified and are considered in the assessment as vulnerability indicators. Among these, the quality of the support supplied from the walls to the roof trusses is of particular importance.

The wall-truss interaction has been examined in order to express a synthetic parameter based on visual inspection and simple measures that may be associated to the vulnerability of the system.

Application to several real cases has given the possibility of improvements of the whole procedure and has given satisfactory results.

The procedure is being implemented in a computer-based system with the purpose to exploit the current possibilities offered by highly portable instruments in order to improve applicability, also in the difficult environmental conditions of roof structure surveys.

8. Acknowledgements

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