

## Seismic vulnerability and preservation of timber roof structures

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**ABSTRACT:** Seismic vulnerability of buildings is a basic component of risk that may be reduced, when necessary, by retrofitting and strengthening interventions. A procedure for assessing the vulnerability of timber roof structures has been studied and is outlined here. The procedure is organized in two levels. A first level collects data on the various elements of the structure. This information is to point out aspects of seismic interest, giving as well a detailed picture of the general state of the structure. A second step focuses on the items that mostly control seismic vulnerability. The main factors examined are the presence of unrestrained thrusts, the connection with the supporting walls, the capability of responding in both principal directions, and the quality of carpentry joints and connections.

### 1 INTRODUCTION

In Italy, strengthening of an existing building toward seismic action is enforced whenever it undergoes major renovation or extensive and extraordinary maintenance. This regulation aims at eventually upgrading the significant fraction of the aging building stock originally constructed with little or no consideration of local seismicity. In these buildings, timber typically supports roofs and floors. In important historic and monumental buildings roof structures may be considerably complex.

During a long period that peaked in the nineteen-eighties, timber floors and roof structures were often wholly substituted with new materials and construction techniques. In some cases, mass and stiffness were excessively increased, resulting in serious damage and even collapse that expanded from the new roof structure to the whole building when new earthquakes occurred, as was the case of the Umbria-Marche earthquake of 1997. Poor performance in these cases suggested the need to develop less invasive interventions by maintaining the original structural concept, yet eliminating known deficiencies.

Furthermore, a stronger consciousness of the importance of the cultural heritage imposes the preference towards conservation whenever possible. This principle, unquestioned for a monument or building with recognized artistic value, extends now to buildings that may be of interest, either by themselves or as part of a larger urban settlement. Such buildings

indeed represent architectural and structural styles and construction traditions of their own time.

In this general context, it seems important to develop a methodology for assessing the seismic vulnerability of timber roof structures. The methodology should be capable of classifying a structure in a scale of recognized values and giving a term of comparison within the domain of similar typologies. The result should also highlight the most significant deficiencies. On the basis of a vulnerability model, a first evaluation of the effect of typical strengthening interventions becomes possible.

### 2 SIGNIFICANCE OF SEISMIC VULNERABILITY ASSESSMENT

Vulnerability of buildings is a basic component of risk in seismic areas. It is a modifiable component: vulnerability may be reduced, when necessary, by retrofitting and strengthening interventions. For this reason, in the last two decades research has devoted a big effort to defining criteria for assessing the seismic vulnerability of different structural typologies.

Studies focused first on common residential buildings, particularly on traditional masonry, with the aim at devising strategies to reduce the risk at the territorial level. Semi-empirical methods that would permit the vulnerability assessment of a large number of buildings in a relatively fast manner based on the visual exam of different elements and characteristics

of the building and on a simplified evaluation of lateral capacity were developed.

A method developed and widely adopted in Italy (Benedetti et al., 1988; Petrini, 1999) yields a numerical index for each building component and a general vulnerability index for the building as a whole. This is expressed as a linear combination of the component indices. By this approach it is possible to estimate the effect of possible interventions in reducing the vulnerability level (Benedetti et al., 1988, Chesi et al., 2006).

Several field surveys performed by this or by similar methods and covering whole towns and regions have yielded an expressive picture of the quality of old construction across Italy.

Subsequently, research focused on protecting the monumental heritage, a very critical issue in many seismic areas.

For monuments, a vulnerability assessment by inspection is particularly justified by the difficulty of formulating expressive numerical models. Such models would require first a detailed field survey to identify the actual structural scheme. The objective of a heuristic vulnerability approach is to identify an expected behaviour directly from this survey, which would give the possibility to include in the evaluation aspects that are not directly related to the structure, yet have a significant impact on the seismic response. The assessment should yield a comparative measure of vulnerability, but should especially point out possible deficiencies.

Vulnerability studies of this kind were developed first on historical church buildings (Doglioni et al, 1994; Lagomarsino 2006) and have constituted the basis for addressing the vulnerability of other monuments and special structures.

The seismic behaviour and vulnerability of traditional timber structures has not been fully treated so far. In particular, timber roof structures have not been considered per se, even though procedures for assessing the seismic vulnerability of buildings generally consider their contribution.

Acknowledging the need of their conservation as previously discussed, a procedure for assessing the vulnerability of timber roof structures has been developed by the authors and is outlined in the following.

### 3 TOWARD A VULNERABILITY MEASURE OF TIMBER ROOF STRUCTURES

To efficiently define a procedure for vulnerability assessment it is necessary to;

- define the objectives in the context specific to the typology examined, and
- identify a reference situation, called “paradigmatic case”, that may be considered as the zero point of the vulnerability scale.

The paradigmatic case will not correspond to full safety, but will present all the positive characteristics that would result in the target seismic behaviour. In the vulnerability scale for masonry buildings, for instance, the paradigmatic case will respond like a new building designed according to current seismic norms. The paradigm for structural behaviour of masonry buildings is the realization of a box-like structure, with all its structural walls capable of collaborating to the response.

Timber structures exist in a variety of configurations; furthermore, the execution of joining details seems to be very influential on the quality of the seismic response. From observation and analysis, a low-vulnerability timber roof structure should have, in order of importance:

- no unrestrained thrusts;
- an effective connection with the supporting walls;
- a comparable response capability in both principal directions;
- effectively reinforced carpentry connections to avoid disassembling.

Consequently, these seem to be the main points that a procedure for evaluating the seismic vulnerability should check.

As to the issue of satisfying context-related needs, the case of timber roof structures is akin to that of vulnerability measures related to churches (Lagomarsino, 2006) and monuments in general: general statistical information on the situation of these structures on a territory and comparison of the structure currently under examination with statistical values may be interesting, but it is not the primary goal. The interest is, rather, centered on gathering significant information on the state of the roof under consideration.

In this perspective, the capability of evaluating vulnerability in comparison with a reference case is the most important issue, as long as the possibility of tracing back with sufficient detail the specific elements that cause vulnerability is offered by the method. In order to satisfy this need, a two-level procedure is proposed herein.

A first step collects data on the various elements of the structure. This information is directed to point out aspects that may affect the seismic response giving, as well, a detailed picture of the general state of the structure.

A second step focuses on the items that mostly contribute to the seismic vulnerability, according to the points listed above; data collected in the first step are used for this evaluation.

Once the vulnerability level is assessed, the detailed information collected on the structure may be a basis for evaluating possible strengthening strategies as well as for analyzing costs and benefits of various interventions.

Table 1. First level of information.

1. Building general info
2. Roof material info
3. Roof-to-wall connection
4. Roof description
5. Supporting structure
6. Structural elements
7. Carpentry joints

The two levels proposed are described in the following section.

The procedure is implemented presently by compiling paper forms. A users' manual is currently being prepared.

#### 4 DEFINING A PROCEDURE: A TWO-LEVEL APPROACH

The survey of the roof structure that is the basis for the seismic vulnerability assessment requires visual inspection and some measuring of the basic dimensions of the structure and elements. Collecting these data constitutes the first step of the procedure. From this information the actual seismic vulnerability assessment follows as a second step.

#### 4.1 First level: describing the state of the structure

Data describing the roof structure characteristics and the state of conservation are collected by visual inspection. The collection is performed by completing a form that guides the analysis by listing different items, as in Table 1. Each item is further developed in a tree-like structure, with branches detailing different aspects to be considered. Guidance for compilation is offered in the form by multiple-choice answers and images, and by corresponding indications in the users' manual.

The first point identifies the building that contains the timber roof (type, location, period of construction) and describes its general geometry (dimensions, general layout) and material (brick, stone masonry, etc.).

The wood species present in the structure are then specified. Visual grading provides the expected strength according to current standards (e.g. UNI-11119 2004, CEN 2006). The presence or effect of decay agents, like mushrooms, mildew, rot, insects, and excessive humidity levels, are indicated at this point.

An insufficient roof-to-wall connection is the primary source of damage and collapse of roof structures during earthquakes. The type and state of the connections are examined at the third point. The form branches into:

- type of constraint (simple support, hinge, built-in end, semi-rigid moment transmitting connection);

Table 2. Roof description.

1. Type
  - a. flat
  - b. pent
  - c. couple
  - d. gable: single pitch  
double pitch ...
  - e. hipped ...
  - f. ...
2. Unrestrained thrusts – Y (localization info follows)  
– N
3. Covering system (choice as from manual list)

- supporting element (wall plate, ring beam. . . ) and further description of each type.

From point 4 onward, the procedure classifies the roof typology and subsequently details its structure, elements and joints. Point 4 itself collects information on the roof type and on other general characteristics (Table 2). The typologies listed in the form were those most frequently found by the authors.

The selection of the type is guided by a series of images. Space is left in the form for the user to insert and draft a type that is not present. Approximate dimensions are estimated and collected. The presence of unrestrained thrusts is a major cause of unsatisfactory seismic response. The type of cover determines the weight of the roof and thus affects inertia forces.

The description of the roof supporting structure follows at point 5. Supporting ridge beam without trusses, trusses of different kind (simple king truss, two-level queen truss), rectangular trussed beams may be selected, or other situations may be described.

An indication of the three dimensionality of the structural conception is also given at this point. Very often roof structures composed with a series of parallel trusses are observed with connecting elements that do not offer similar stiffness in the direction orthogonal to the truss. The level of out-of-plane stiffness available may vary often according to regional constructional traditions. In many cases, however, the out-of-plane deformability is controlled by ad-hoc sub-structures, as in the example shown in Figure 1. This roof, discussed also in the following section, presented additional trusses in the orthogonal direction. Figure 1 shows a partial image of the roof, while being dismantled. Lastly, cases where the structural conception is fully three dimensional are also found. The presence of significantly different lateral stiffness in different directions negatively affects the seismic response.

At this point of the evaluation form, indication may be given of the presence of evident conceptual errors, leaving, for instance, some degrees of freedom unrestrained. These situations are not so infrequent in older vernacular construction and are occasionally detected when inspecting roofs. They constitute a danger in every case, regardless of seismic conditions.

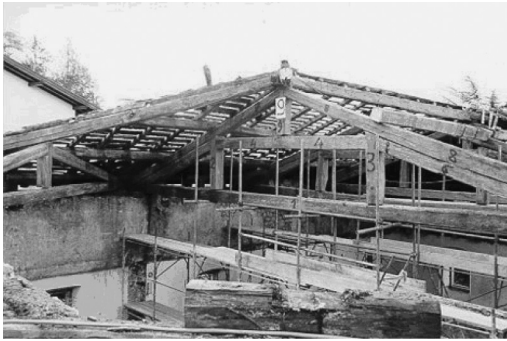


Figure 1. Roof structure with trusses in two directions.

Table 3. Structural elements (trusses).

1.	Ridge beam
	– section
	– length
	– constraints
	– conservation state (decay, settlements)
2.	Purlin
3.	Rafter
4.	Second (parallel) rafter
5.	Tie beam
6.	Post
7.	Strut
8.	...

The following point describes in detail the situation of the individual elements of the structure, or each substructure. Structural elements for trusses (Table 3), require some downbranching and multiple answers are possible. For elements occurring more than once in a truss, the location is identified (e.g. right rafter, left rafter); reference to similar elements already described may be made avoiding repetitions.

When trussed beams are used, an infrequent choice, similar information is collected for the corresponding elements (diagonals, etc.).

The last point of Table 1 covers the description of connections (e.g. carpentry joints). These may be realized in very different ways, according to the location and function. Within a same location in the structure different choices may derive from different requirements related for instance to structural size, and to workmanship, giving rise to a variety of situations.

The connections coupling the various truss elements have been considered and for each the different types of carpentry joint typically adopted have been listed (Table 4). For each connection, various choices are suggested; for each joint type, details descend in subsequent branches. The table reports only an example of subsequent branching.

Table 4. Carpentry joints.

Connection	Joints
1. Rafter-to-tie beam	a. birdsmouth – connection angle – type single notch, double notch reverse – notch depth – tenon present – metal elements old recent type – strip – bolt – stirrup ... – state of connection
2. Rafter-to-post	a. tenon-mortice ... b. birdsmouth ...
3. Strut-to-rafter	...
4. Rafter-to-rafter	...
5. Strut-to-post	...
6. Post-to-tie beam	...
7. Tie-beam scarf joint (to increase length)	...

Table 5. Vulnerability indicators.

1. Unrestrained thrusts
2. Supports
3. Structural typology
4. Carpentry joints
5. Elements
6. Conservation state

Some characteristics typical of timber construction are not analyzed, because of their lower impact on seismic vulnerability and in order to reduce surveying and form-filling time. Durability is not analyzed per se, yet indication of critical situations can be given for joints and elements and as final remarks on the structure.

#### 4.2 Advanced level: assessing the seismic indicators

The second part of the analysis directly concerns the aspects that may be considered as significant indicators for seismic vulnerability.

Making reference to the elements and qualities that permit a favorable seismic behaviour, as listed in section 3, a series of indicators has been considered, (Table 5). The data collected at the first level are used for their assessment.

The negative effect of unrestrained thrusts is amplified under seismic conditions, easily triggering collapse of walls and roof.

Ineffective connection with supporting walls or insufficient extension of supports is the main cause of the relatively frequent collapse for roof structures, often generating a domino effect collapse of the underlying stories.

The effects of structural typology and of connections, listed at points 3 and 4, are discussed in detail further in the paper.

Occurrence of an earthquake modifies and generally increases the level of stress in structural elements. In general, this additional stress was not considered by the original design but may be accommodated by the large sections and consequent safety factors that most traditional structures present. Particularly slender elements with sections deemed insufficient may be acknowledged at point 5 of Table 5.

Finally, the conservation state, and any other specific observation concerning the state of the structure, is evaluated at point 6. A poor level of maintenance has been recognized as a vulnerability contributor for other typologies. Its role is expected to be significant particularly for timber.

#### 4.2.1 The rating process

Following a criterion adopted in other procedures for vulnerability assessment (Petrini, 1999) each indicator in the list is ranked according to a scale of values ranging from A to D, where A represents a satisfactory situation and D indicates a high vulnerability level, for which an improvement would be strongly recommended. Intermediate situations are indicated by values B and C, depending on whether the case is closer to one or the other extreme.

In spite of class borders being at times fuzzy, attributing classes is a fairly objective operation. For each indicator, a numerical value may then be associated to classes, permitting to express a partial vulnerability index. Combining partial indices yields, then, a global value for the case under examination.

This procedure involves two delicate tasks, i.e. attributing numerical values to classes and defining well balanced combination coefficients for the partial values to obtain a global index. Separating the selection of an appropriate class for an element and the attribution of a numerical value to the class permits redefinition of the latter at later times. Combination weights are necessary because the indicators have different roles and importance in inducing or reducing vulnerability. Calibration of these values is still under development by the authors for most cases.

#### 4.2.2 The structural typology

This indicator grades the roof structure from the point of view of its typology being more or less suitable to respond to seismic excitation.

First of all, evident deficits either present in the original conceptual design of the structure or introduced by subsequent remodeling and possibly by inappropriate

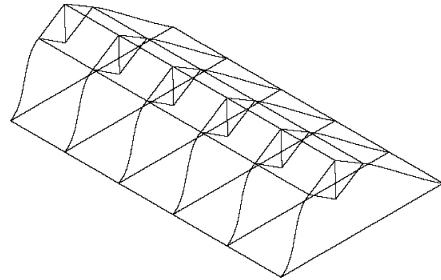


Figure 2. Longitudinal modal shape in a structure with one purlin for each rafter; first mode.

strengthening operations, if pointed out in the first level survey, are acknowledged here.

One significant issue from the seismic point of view is the capability of responding in different directions of excitation. Most often roofs are supported by a series of parallel trusses transversally connected in more or less efficient ways. The response in the two horizontal directions, parallel and orthogonal to the truss system, may be very different.

Seismic analyses performed for numerous structural typologies and cases are necessary to constitute a basis for evaluating the three-dimensional conception. For this purpose and for general calibration of the vulnerability procedure, various structures were surveyed and data collected in four different Italian regions (i.e. Lombardy, Trentino, Southern Tyrol and Calabria). Cases ranged from simple pent roofs in a Calabrian location to articulated three-dimensional cases in the Trentino region.

Modal analysis is an expressive, yet simple method to examine the attitude of the structure toward directional response. Here, two simple but frequent gable roofs are compared. The two structures, composed of 5 parallel king trusses, are 20 m long and span 12 meters. They differ for the number of purlins, either one or two, at each pitch. Element sections are  $20 \times 20$  cm. For sake of simplicity and in order to consider the lowest grade of joint stiffness joints have been modeled as fully hinged where applicable, without introducing semirigidity. Figures 2 and 3 show the first longitudinal mode for the two structures. The single-purlin case, in figure 2, has low stiffness in the longitudinal direction, where the first modal form appears with practically total participating mass and a relatively long period of 0.65 seconds.

The second mode is lateral, as in figure 4, with participating mass of 15.23 percent and a period of 0.47 seconds. The second assemblage is longitudinally stiffer and yields a more restrained longitudinal mode, as in figure 3, which also consolidates virtually all the participating mass in that direction. Yet, this is the third mode to appear for the structure, the fundamental mode being transversal, as in figure 5. Periods are 0.51

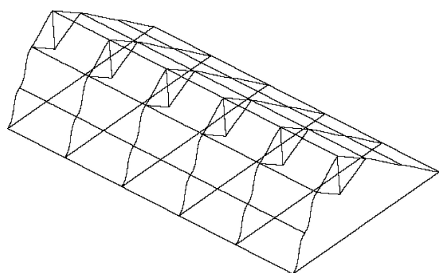


Figure 3. Longitudinal modal shape in a structure with two purlins for each rafter; third mode.

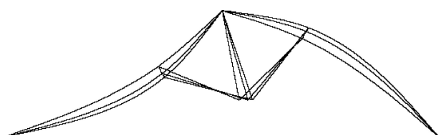


Figure 4. Transversal modal shape in a structure with one purlin for each rafter; second mode.

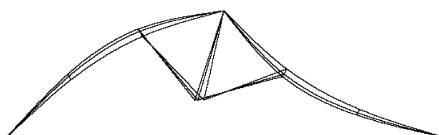


Figure 5. Transversal modal shape in a structure with two purlins for each rafter; first mode.

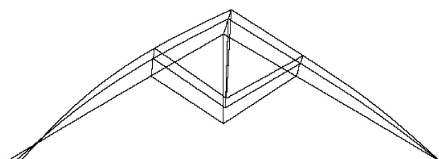


Figure 6. Vertical modal shape in a structure with one purlin for each rafter; ninth mode.

seconds and 0.33 seconds for the first and third mode, respectively. The structure appears better balanced in its three-dimensional behaviour.

Longitudinal displacements may be actually restrained by the usually slender gable walls, which in change become loaded outside their plane.

Vertical modes develop for higher frequencies in both models. Figure 6 shows mode 9, the first vertical mode, for the single-purlin structure, with a period of 0.09 seconds. These modes are less significant for general structures, yet they may be worth investigating further for their possible effects on pressure joints.

The structure previously shown in figure 1 and modeled for modal analysis in figure 7 is a case of three-dimensional conceptual design, with secondary diagonal trusses stiffening the longitudinal direction.

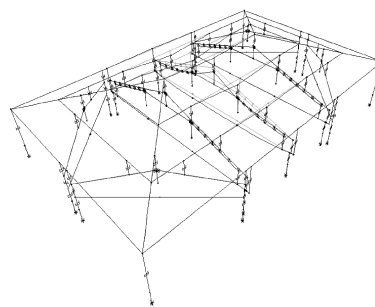


Figure 7. Numerical model of the roof structure shown in Figure 1.

Dimensions are comparable with those of the structures analyzed above, the span being of 14 m, the length around 22 m, and corresponding sections. For sake of completeness, the roof structure was dismantled and eliminated, when the building underwent major renovation and an increase in the number of stories. Its timber structures were donated to the University of Trento for research purposes.

The first modes involve the central part of the structure. The figure shows the fundamental mode that is in the vertical direction, with a period of 0.31 seconds. In this model, joints have been considered semi-rigid on the basis of experimental values, which increases the general stiffness. The main effect, however, derives from the structural configuration.

Better or less proficient structural configurations are recognized in the vulnerability procedure by comparison with example cases supplied in the manual, without performing specific calculations.

#### 4.2.3 The carpentry joints

The key points for the evaluation of carpentry joints are the maintenance of connection during alternate excitation and their expected post-elastic behaviour. The results of a parallel research program on mechanical behaviour of carpentry joints are the basis for the following evaluation.

At the worse side of the scale, D, are joints that are unrestrained or ineffectively restrained against disassembling. According to the period of construction, the level and quality of node binding may vary significantly, with absence of metal connectors in very old cases as one extreme. At the same time, excessive stiffening of the node may derive from reinforcing interventions carried out in a more recent past, as in the example of figure 8. Eliminating the possibility for the connected elements to rotate modifies the original conceptual design of the node and structure, forcing a different behaviour and very likely triggering brittle failure under extreme conditions, as in the case of an earthquake.



Figure 8. Very high stiffness is introduced at the node and in the tie beam by this strengthening intervention dating back to the 1960s.

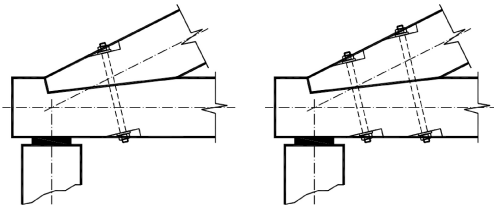


Figure 9. Birdsmouth joint reinforced with 2 bolts either in the transversal direction (left) or longitudinally (right).

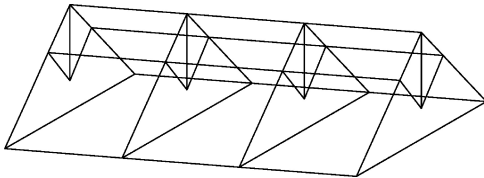


Figure 10. Example of roof structure.

Excessive strengthening that precludes rotation and may induce brittle failure may also derive from the use of a limited amount of connectors as an effect of their positioning. In figure 9 two bolts are used to reinforce a birdsmouth joint either transversally, as in the image on the left, or along the rafter axis, as in the right image. Experimental analysis of the two solutions has given a very positive outcome on the first layout and a very discouraging one on the second.

At the better end of the vulnerability scale, A, are connections safeguarded against separation of the connected elements due to sudden and temporarily decrease of pressure or loss of contact, yet maintaining their original semi-rigid hinge behaviour. Intermediate scale values are covered by a variety of situations to be evaluated with respect to these two limits, considering the effectiveness of connection, possible fragilities, as well as the reliability of the connecting elements.

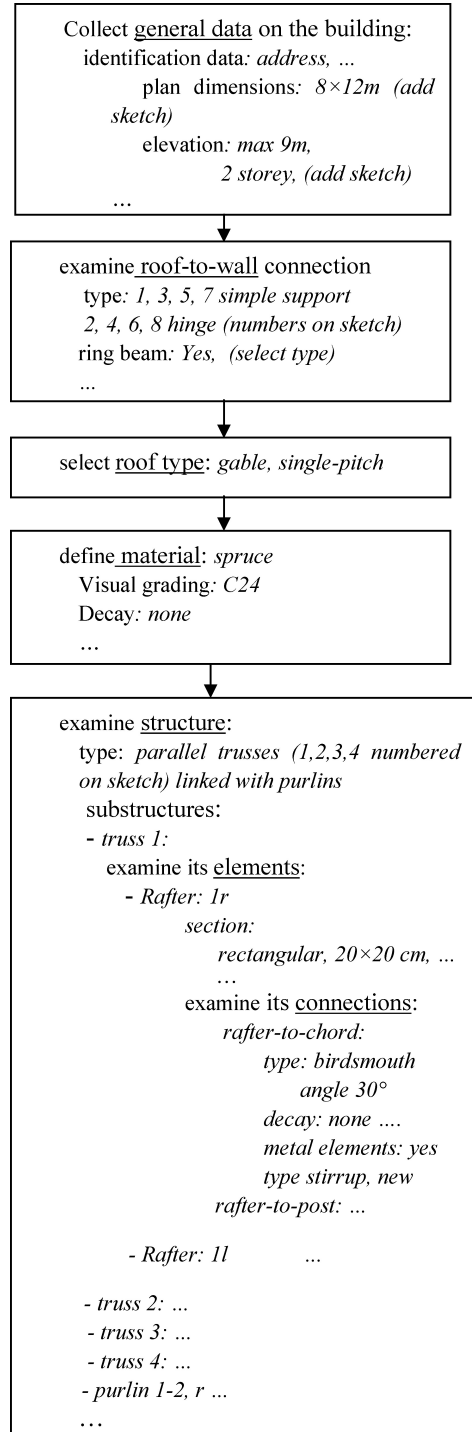


Figure 11. Flow chart of a vulnerability assessment, level 1.

#### 4.2.4 An example

Exemplifying the actual vulnerability assessment of a roof, which requires an extended series of operations and involves a considerable amount of data, is not possible here. With reference to the roof scheme in Figures 10 and 11 summarizes the order of operations for the first level, with the supposed operator's answers in italics. In the structural block, components are numbered with reference to the substructure (truss) and to their position.

The vulnerability indicators would then be evaluated interpreting the collected data according to the given guidelines. Many details are missing in this example. Grossly, no unrestrained thrust being present and with satisfactory supports, the first two would be rated "A"; the structural typology could possibly be rated "B", given the different lateral stiffness in the two main directions; the rating of the rafter-to-chord node highly depends on the reinforcement: a "C" may be assumed here, if no special provisions to prevent brittle failure were taken, because stirrups may develop this kind of ultimate behaviour; elements are evaluated on the basis of their dimensions and of the seismic action expected at the site; here, elements and conservation state could be rated "A".

#### 4.2.5 Future developments

The procedure makes use of an evaluation form listing the items to be inspected. The form has undergone various modifications from its first layout, as suggested by trial and error application to a number of structures. In this preliminary stage, a paper form seemed more suitable for experimentation and calibration. The use of an information technology approach would offer the possibility of direct data control and organization and the flexibility of an interactive interface that could simplify data collection. It is being considered for a future development of this work.

## 5 CONCLUSIONS

The need for methods to assess the seismic vulnerability of timber roof structures has been discussed, in the perspective of preserving both monumental artifacts and existing structures of more common character

that may represent architectural and constructional traditions. A two-level procedure has been proposed and its main features described. Data from roof structures, collected in different regions, are being used to calibrate the procedure.

## ACKNOWLEDGEMENTS

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