SEISMIC STRENGTHENING OF TRADITIONAL CARPENTRY JOINTS

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ABSTRACT :

Traditional timber construction was based on using plain timber elements and carpentry joints that work by contact pressure and friction, often these joints had only minor reinforcement generically intended to avoid disassembling. In current practice, interventions for the upgrading of carpentry joints are mainly based on empirical knowledge according to tradition, are not specifically conceived for the case of seismic action and often result in overstrengthening. The mechanical behavior of the carpentry joints is examined here with emphasis on those most used in roof structures. The main connection type is the birdsmouth joint, connecting rafters to the tie beam; its behavior has been studied numerically and experimentally in monotonic and cyclic conditions. Other forms of connection are discussed as well. Some general criteria for the seismic strengthening of these joints are presented.

KEYWORDS: Timber roof structures, connections, seismic strengthening

1. INTRODUCTION

In newly designed timber structures, complying with seismic principles and regulations, the seismic response highly relies on the mechanical behavior of connections. Because of the brittleness intrinsic in the material, which prevents relying on any post-elastic resources, timber elements are kept within the elastic range. On the contrary, suitably designed connections may develop post-elastic dissipation capabilities exploiting the interaction between timber and the steel connectors.

The situation of existing buildings is different. In historical timber structures, carpentry joints transmit forces between timber elements by direct contact and friction, whereas metal connectors, when present, only safeguard the connection against exceptional actions that could relieve contact and separate parts. While for newly designed and modern-type connections mechanical models are available and research is devoted to develop and improve them, in general the mechanical characteristics of traditional connections are not well known, especially for what concerns their cyclic and post-elastic behavior that is most important in seismic conditions. The literature on this topic is limited to a small number of contributions (e.g. Bulleit et al., 1999). Some interesting works are from authors of Far Eastern Countries (e.g. King et al., 1996; Seo et al., 1999, Lam et al.) and indicate the strong interest in preserving timber buildings and monuments like pagodas in a long and very sophisticated tradition in timber construction. The work presented here is devoted to joints that are frequently found in the European tradition, both in monuments and more common buildings.

Most times, carpentry joints were built based on traditional practice based on empirical rules, or at most designed with extremely simplified models that are not supported by sufficient experimental and analytical evidence and consequently they may not comply with the safety levels that are now imposed, particularly for seismic action. In today's practice, interventions for the upgrading or strengthening of carpentry joints are still strongly based on empirical knowledge. Often they produce a general strengthening of the connection, but are not specifically conceived for the case of seismic action. Strengthening on heuristic bases may be only partially effective; often strength and stiffness increase excessively, inducing fragility as a particularly dangerous side-effect in seismic conditions.

Although in many seismic regions, particularly in southern Europe, buildings with full timber structures do not occur frequently, timber structures have been used extensively to support floors and roofs, making structural timber almost universally present in seismic areas. For this reason, a research program has been developed by the authors for characterizing the mechanical behavior of carpentry joints. The work is still in progress. The joint typologies most frequently used in timber structures, with special attention to roof trusses, have been considered in the study. In the following, some results are presented, with special attention to the rotational behavior of these joints and to the effects of possible seismic strengthening operations in the elastic and post-elastic field.

2. TYPICAL CARPENTRY JOINTS

A number of timber structures have been examined in different geographic areas in Italy to collect realistic data and observations for the study and to select significant cases. The joints described in the following cover most situations and have been considered in the study.

2.1 Joints at truss nodes

The most frequent layout of roof structures is given by a series of parallel trusses, usually connected by simple purlins or, seldom, by more elaborated transversal elements. Depending on the surface to be covered, trusses may reach considerable size and complexity.
The rafter-to-tie beam connection has always been considered the most important node of a truss, as may be seen, for instance, consulting traditional design manuals. Figure 1 presents the forms of this joint that are most frequently found.

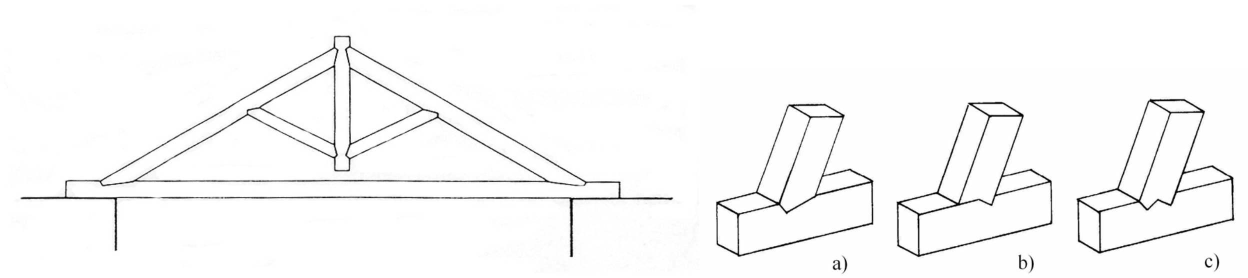

Figure 1. Typical roof truss containing birdsmouth connections at different locations (left); types of connection in roof trusses: (a) birdsmouth joint, (b) its reversed form, (3) double notch joint (right)

The birdsmouth joint, (a) in Fig. 1 is the most frequent solution and usually presents a skew angle of about 30° between the tie beam and the rafter. The same kind of joint, with an angle of about 60°, connects rafters and post; it is often used also at strut ends in the typical and most frequent “palladiana” truss, as in Fig. 1, in which diagonal struts link the rafters to the lower end of the post. The form (c) of the joint, with a double notch, is found in trusses covering a wide span, like in roofs of monuments and public buildings, where the rafter is particularly deep.

Again for large spans, occasionally a joint type intermediate between the single birdsmouth and the double notch is found, where a double indentation in the tie beam accommodates two parallel rafters.

These joints cover practically the majority of cases of timber trusses in the Italian territory, where this study has originated. Usually metal connectors were added to the joints in order to avoid disassemblage due to unexpected actions, yet the connection was always intended to work by contact and friction in service conditions.

2.2 Continuity joint

For trusses covering large spans, length limitations of the timber elements due to the natural origin of the material often imposed to head-join two elements to reach the required length of the tie beam. A scarf joint is usually adopted to reconstitute continuity. The joint may vary for the number of indentations and the scarf inclination. Figure 3 shows the joint area in a case studied and commented in the following section. Transversal reinforcement was present.

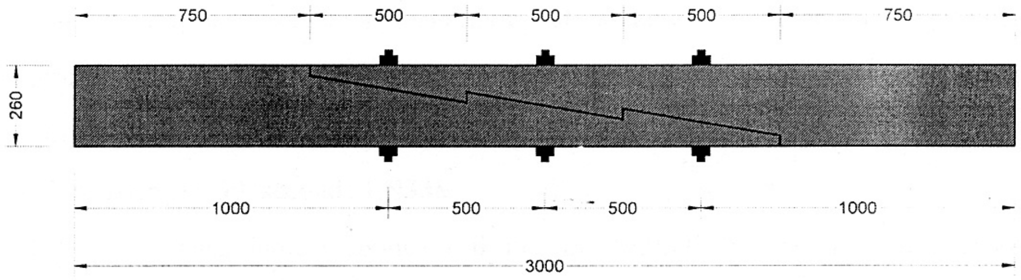

Figure 2. Scarf joint in a tie beam; dimensions are taken from a real case and are in mm.

2.3Other connections

The cross connection shown in Fig. 3, joins two elements at mid-length. The element thickness is reduced to half in the joint region, so that the elements are inserted one into the other and the assemblage is planar. This joint was not found in roofs, because scissor trusses are unusual in the Italian territory, but it is found connecting bracing elements as in the building of Fig. 3.

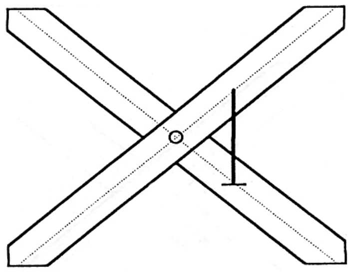

Figure 3. Half-depth cross joint in bracings of a timber structure (left); scheme of the jointed elements in a specimen, the vertical bar indicates the position of relative displacement measures (right)

3. TESTING AND NUMERICAL ANALYSES

The joints described above have been studied by numerical analysis or experimentally, or both, with a different level of deepening in terms of cases and variations examined that depended on the importance of the joint. Further studies for some types are in progress or planned.

The birdsmouth joint has been studied most extensively, because of its preminence among possible connection solutions. Its rotational behavior has been characterized by numerical analysis and experimentally in monotonic and cyclic conditions, in the elastic and post-elastic field up to failure (Parisi and Piazza, 2000 and 2002a). Monotonic tests were performed on unreinforced joints and on strengthened ones, cyclic tests on reinforced joints only. The effect and the possible inconveniences of different retrofitting methods where pointed out by direct experimental observation as well as by comparisons with numerical results. Figure 4 shows the testing apparatus for a rafter-to-tie beam node: a constant pressure is applied to the rafter, while a lateral force is applied to generate moment and rotation at the node. The experimental setting permits load reversal.

The static behavior of a double rafter has been examined experimentally in two queen trusses of different size under static loading. Experimental results were matched with numerical analysis that included semirigid modeling of the joints. The trusses were then analyzed assuming seismic conditions (Parisi and Piazza, 2002b).
The joint with a double notch, (c) in Fig. 1, has been recently investigated by means of numerical analysis in

monotonic conditions. The rafter size assumed in this case has been 34x34 cm, a fairly large value that permits to realize this kind of connection. An orthotropic elastoplastic material and friction along the contact surfaces between elements characterize this model as well as all the other numerical analyses on different types of connection. Further elaboration of results and the analysis of additional cases are under way. Physical testing of this assemblage has not been performed at this time.

Similarly, the scarf joint case has been analyzed numerically for monotonic loading in the configuration of Fig. 2, to assess the effective degree of continuity produced and examine the post elastic behavior with the development of the failure mechanism.

The cross joint has been investigated experimentally, together with parallel numerical analyses, following the post-elastic behavior, in order to ascertain the mechanism and derive possible strengthening approaches.

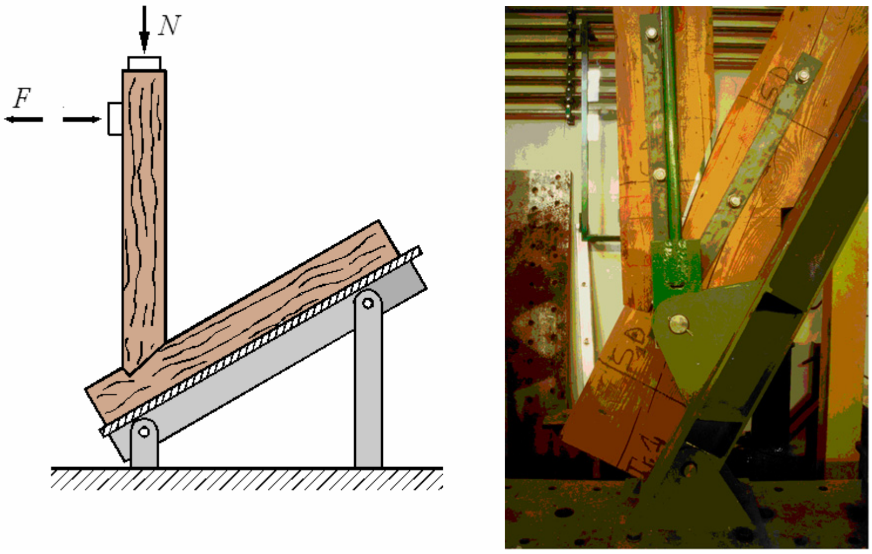

Figure 4. Testing of rafter to tie beam joints; layout of the apparatus, applying an axial force N and a transversal force, F, with possibility of reversal (left); a joint reinforced with lateral stirrups being tested (right).

4. EVALUATING THE SEMIRIGID BEHAVIOR

The two main objectives in investigating the behavior of carpentry joints are a) to assess their capability to undergo load cycles into the post-elastic field and b) to determine values of their elastic stiffness, to be used in practice for an accurate modeling of the structure in the design or redesign process. Indeed, according to design codes, newly designed timber connections are to be considered semi-rigid in structural analyses; stiffness values may be derived for different connection typologies.

For existing structures to be retrofitted, no indications or stiffness values are available. In carrying out analyses, joints are usually modeled either as hinges, or as full-moment transmitting connections when hinges seem inappropriate. Some structural configurations used in roofs cannot reach equilibrium if computed with hinges, confirming that connections are in reality moment resistant. Carpentry joints are characterized by intermediate levels of moment transmission and may be classified as semi-rigid. It is worth noting that the possibility of introducing semi-rigidity at a node between two beam elements is now offered by many commercial finite element systems used for professional applications, making it possible to develop more realistic structural models once the stiffness values are known. In this perspective, initial rotational stiffness values have been an objective of this study.

For the birdsmouth joint the initial stiffness and the elastic behavior in general has been obtained by numerical models of the node area and verified by experimentation. Values depend on many parameters, among which are the strength of the material and the skew angle: rotational stiffness values are approximately half at 60° compared to 30° all other conditions being the same. The height of the rafter has very significant influence, as may be seen in Table 1. The level of rafter compression also influences stiffness, but to a minor degree.

The rotational stiffness has been evaluated for birdsmouth joints in the unreinforced condition as well as in the presence of reinforcement, for the two reinforcement types that resulted more apt for strengthening, as described in the next section. Results are also summarized in Table 1 for a choice of parameter values. Only a moderate increase in rotational stiffness occurs in reinforced joints. The aim of reinforcement is, today as in the past, at protecting the connection toward exceptional actions, rather than modifying its normal mode of behavior in elastic conditions.

The rotational stiffness of the double notch joint, again in Table 1, was evaluated for an angle of 30°. Its comparatively high value derives from the deeper section that is adopted here and that corresponds to common cases in this typology. Due to non availability of the material, section dimensions hardly exceed these values.
The double rafter corresponds to two single birdsmouth joints and its global stiffness may be derived from theirs.

In the table, a reference value is given for the cross joint as obtained from a series of tests, as commented in the following section.

The effect of semi-rigid modeling of joints on the calculated structural response may be examined with reference to the modal analysis of a king-post truss, with span of 12 m, skew angle 30°, section of the elements 20×20 cm. The fundamental period resulted of 0.52 s in the all-hinge assumption and of 0.45 s considering the connections as semi-rigid with rotational stiffness values as above. Higher differences may be expected from other structural configurations.

Table 1 – Initial rotational stiffness of joints (+ = closing, - = opening), friction coefficient 0.30

type	Section, cm	skew angle	Rafter compr., (MPa)	Ki+ (kNm/rad)	Ki- (kNm/rad)
Birdsmouth, unreinforced	20x10	60°	1	310	320
Birdsmouth, unreinforced	20x15	60°	1	680	780
Birdsmouth, unreinforced	20x20	60°	1	1300	1540
Birdsmouth, unreinforced	20x20	30°	range 1-1.5	(mean) 1660	(mean) 2000
Birdsmouth, single bolt	20x20	30°	range 1-4	(mean) 1990	(mean) 2013
Birdsmouth, binding strip	20x20	30°	range 1-4	(mean) 2186	(mean) 2257
Double notch, unreinforced	34x34	30°	Range 1-2.5	(mean) 3870	(mean) 3863
half depth cross	15x10	-	-	160	160

5. POST-ELASTIC BEHAVIOR AND STRENGTHENING CRITERIA

The post-elastic behavior of these carpentry joints has been investigated to date with regard to the development of failure mechanisms for increasing loading. For the birdsmouth joint, the ultimate conditions and dissipation capabilities under cyclic loading with different types of reinforcement have been considered.

In older design manuals for traditional timber as well as in more recent publications, a number of strengthening interventions for carpentry joints, and particularly for the rafter-to-tie beam node, may be found. While in older references no mention is made to the seismic problem, more recent publications often present the strengthening of nodes within the seismic upgrading interventions for buildings. The interventions suggested are, however, based on traditional practice and on general principles and are not specifically conceived for seismic actions nor evaluated in terms of effectiveness on the seismic response. This consideration has suggested to investigate common typologies of intervention derived from traditional practice in view of assessing their effectiveness in the post-elastic field under cyclic action. Experimentation in these conditions has concerned birdsmouth joints reinforced with different types of metal connectors, as in Fig.5. Detailed numerical models of the node area with different reinforcement types, have also been developed.

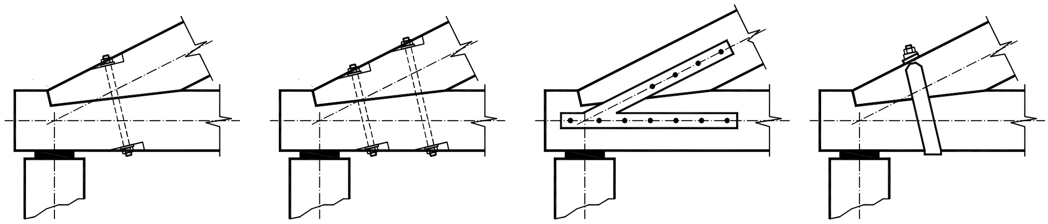

Figure 5 Reinforcement typologies; from left single bolt or two bolts placed transversally, two bolts placed longitudinally, stirrup, binding strip

A reinforcement typology tested consists of V-shaped metal connectors positioned on both sides of the joint, or "stirrups". The reinforced joint may be seen also in the picture of Fig. 4 while on the testing apparatus. At unloading after exceeding the elastic limit, the timber joint disassembled and no recovery of contact was possible. The connection response was subsequently based only on the stirrups. Yet, the stirrups may end in brittle failure, their ribs being welded and possibly subject to instability in the compressed area. A recent experimental campaign carried out at the University of Minho, Portugal, repeated the tests and did not observe this phenomenon (Tomasi et al. 2007), yet the strong change of behavior of the connection relying only on the metal elements and the possible brittle failure that could occur discouraged further study asnd use.

Binding strips have been in the past another widely adopted type of reinforcement. The version adopted today and used in the tests is a modern form that permits retightening. The strip is positioned in a notch in the timber elements. The first loading showed an extension of the elastic phase compared to the non reinforced specimens. No fragile modes were observed.

Finally, tests were performed on joints reinforced with one or more bolts passing through rafter and tie beam. Results from cyclic tests were different for the various combinations, as follows. When a single bolt, 20 mm in diameter, was used, a satisfactory cyclic behavior was obtained, with a number of fairly stable moment-to-rotation cycles. Afterwards, more serious degradation of stiffness and sliding due to a pinching effect started. No loss of connection, however, occurred also in degraded conditions. The pinching in the cycle was due to yielding of timber compressed by the bolt in the direction of fibers and especially by the washer across fibers, a weaker direction of the material.
In order to better distribute pressure and to reduce plastic deformation, the connection was also realized with two bolts of smaller diameter, 12 mm. Results in this case were very different depending on the position of the bolts. A high number of very stable cycles were obtained with the two bolts aligned transversally to the connection. When bolts were disposed along the rafter axis, the connection became excessively stiff. Brittle failure was reached before developing a cyclic response.

Additionally, brittle failure by sliding of the beam toe at the base of the notch was observed in some tests, particularly for low skew angles. This failure mode has been avoided by reinforcing the toe area with evenly distributed, small-diameter screws. Similar reinforcement was then applied also to the tip of the rafter, where compression is high and thickness small, to avoid local failure.

As a general result, testing has shown that suitably reinforced joints may develop a satisfactory cyclic behavior in the post-elastic field. Yet, excessive stiffening and local conditions triggering brittle failure modes must be carefully avoided. Excessive stiffening in some interventions, like those encapsulating the node in a sort of metal cage that were fairly popular some years ago, is straightforward to recognize. Yet, testing has shown that more subtle, yet dangerous increases of rotational stiffness may derive just from incorrect positioning of a small amount of reinforcement. Finally, adding local reinforcement in critical areas in the form of well distributed, small size steel elements may avoid developing brittle failure modes.

The positive result of strengthening interventions has been confirmed also evaluating for various earthquake records the behavior factor, q, in trusses where the joints have been modeled with the cyclic behavior obtained in the experimentally from reinforced connections. The results, with $q \cong 3$, confirmed a satisfactory seismic response.

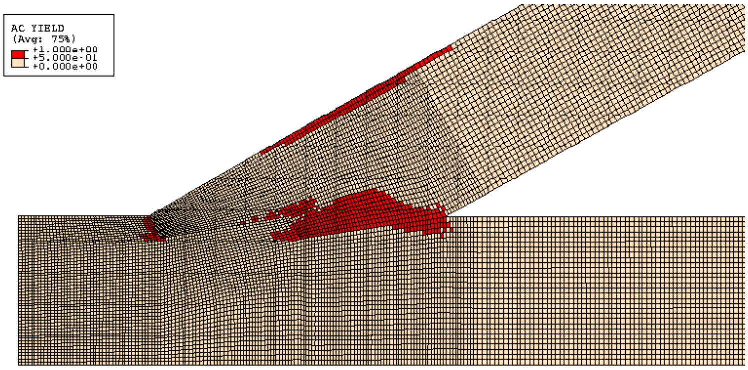

Figure 6. The specimen was tested in rotation closing the angle; red (dark) marks indicate areas of embedding.

For the double notch joint the post-elastic behavior has been to date examined numerically in monotonic conditions for a non reinforced case, with rotation closing or opening the skew angle (Parisi et al., 2008); analyses have been carried out with a range of different compression levels in the rafter. The effect of the friction coefficient has been also assessed. Figure 6 shows the spreading of plastic zones. The specimen was tested in relative rotation of the elements; red marks indicate areas of embedding. The joint has a high degree of interlocking and, unless the friction is annihilated, it is less at risk of disassembling than the single notch case, because extended yielding would occur first. Only small plastic deformations are possible in tension, limiting the value of the maximum moment that is realistically reachable. Experimental testing is planned for this connection. The intention is also at checking the validity of reinforcement suggested by the analysis to avoid splitting.

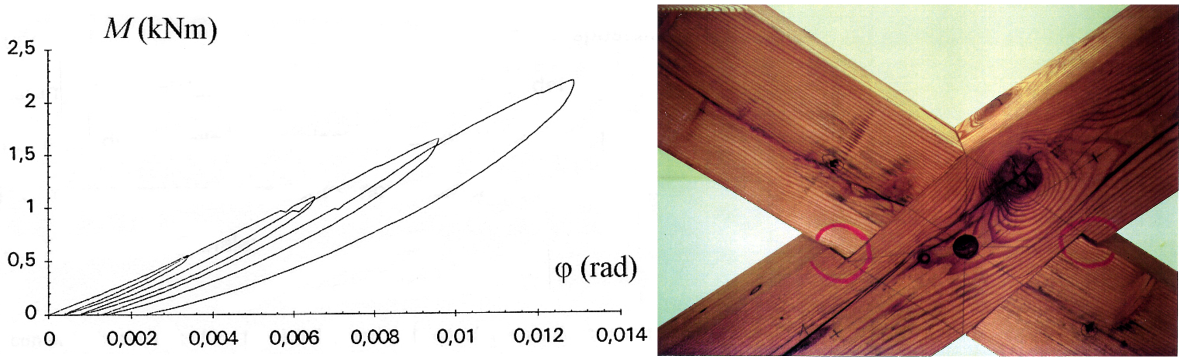

Figure 7. Experimental moment versus rotation results for a half-depth cross joint. The specimen was tested in relative rotation of the elements; red marks indicate areas of embedding

Experimentation on the cross joint has been carried out on specimens with section of 15 x 10 cm. The tests were performed generating moments by applying two opposite vertical forces at 2 extremes of the elements. Testing has pointed out especially the 3D behavior of the assemblage. Tests were performed on joints connected with a wood peg, a snug-fit unrestrained steel bolt, or a bolt with nut. All the cases tested presented a very similar stiffness, from which the value previously indicated in Table 1 was derived. Beyond a first stage where gaps are covered, the joint acquires rotational stiffness, deriving it from various effects that oppose rotation, as friction and plastic deformation of timber in transversal compression. For further increase of the moment, beyond values in Fig. 7, the curve becomes flatter. Tests were stopped before ultimate conditions. The nonsymmetrical geometry of the two connected elements in the joint area tends to separate them and disassemble the joint. This effect is unrestrained with the peg and the free bolt and better opposed by the nut.

Numerical analyses of the scarf joint have been carried out for now to assess the degree of continuity that may be expected in a reinforced layout as in Figure 2. The finite element model acknowledged the geometry of the notches, of the reinforcement, and the contact situation between wood elements and between wood and metal.
The degree of continuity, intended as ratio of the ultimate load for the jointed element and for a continuous timber element, was 0.2. The ultimate condition would result from shear failure at the notches. The presence of reinforcement, however, avoids a brittle failure mode. The behavior of the connection presented a good level of ductility, with an extended post-elastic branch, whereas an elastic-brittle behavior would characterize the full timber element.

6. CONCLUSIONS

In the seismic response of timber structures, joints are the key elements because they may either contribute to fragility, or develop, to some extent, a dissipating behavior. In the perspective of seismic strengthening of timber structures, general criteria for strengthening the connections to resist the effect of seismic action are needed, considering that traditional intervention methods in timber construction are generally conceived in view of a static behavior under vertical loads. The results of an experimental and numerical research program on the behavior of carpentry joints have been outlined here:
the semi-rigid behavior of carpentry joints has been examined and quantified;
their post-elastic behavior and some indications and considerations on the modalities of intervention have been discussed.

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