Full-Size Truss Joints Made of Old Wood: Laboratory Tests on Shear Failures

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1. Introduction

Traditionally, timber truss is one of the most used load bearing structures for roofs in Italy, especially in historical buildings; if properly realized and regularly checked and maintained[1], it could represent the best structural solution for a very long time [Bonamini *et al.*, 2001]. On the contrary the neglect and the defect of maintenance, combined with a bio-deterioration or an increase of the load, could lead to very important structural problems [Ceccotti *et al.*, 1998, Follesa e Lauriola, 2004], which sometimes require hard work and thorough knowledge to be solved and to avoid the risk of failure [Uzielli, 2001-2004, Bonamini *et al.*, 2002].

In some cases the presence of wood "defects" like knots, shrinkage fissures, slope of grain, or wood damage (e.g. insect attacks) in critical locations of the truss such as joints, may affect the mechanical performance of the structure, inducing an early failure at low load levels, in other cases the strength of the structural system is not affected by specific defects, but may be difficult for the timber surveyor to state.

Understanding the failure behaviour of full dimension joints is an important goal, particularly in order to optimize on-site examination and diagnosis of timber structures[2]; by such means, the visual assessment and evaluation of strength properties of timber elements and joints can be performed with greater reliability, with the objective of saving old timber members from replacement or heavy strengthening, if not strictly necessary.

This paper reports about the first part of a wider study in progress; it presents an experimental research aiming at determining the strength of nodal joints connecting bottom and top chords in traditional trusses, and to verify how shear failures occur in heel; then to test the shear strength of little full dimension specimens, according to EN 1193, made with the timber of the same structural elements.

Main purposes are to assess the pattern of failure in shear of heel connections, to relate it with defects and wood damage, usually present in full size ancient timber beams, and to compare the mode of shear failure between the two different test modalities. The strength values are also compared and discussed, in view of evaluating possible design values.

2. Materials and methods

The testing methodology has been planned starting from the need to test some

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[1] The on-site examination and diagnosis of timber structures, by means of visual assessment, evaluating strength properties of timber elements and joints, repeated at regular intervals, generally is very opportune, if conceived in a logic and coherent strategy of monitoring and maintenance [ICOMOS, 1999].

[2] As pointed out in the Principles for the preservation of Historic Timber Structures "...a thorough and accurate diagnosis of the condition [...] of the timber structure should precede any intervention..." [ICOMOS, 1999]. This approach has been confirmed by some recent Italian rules about criteria for inspection and assessment of timber structures relevant for Cultural Heritage, in order to collect all necessary data ahead of every kind of intervention [UNI 11119:2004, UNI 11138:2004].

full dimension elements. The focus of interest was the connection between the top- and the bottom-chord, for its fundamental role and for the risk of failure, due to bio-deterioration, wood damage or high load. The timber members have been prepared to reconstruct the portion of a typical truss: a heel joint.

The connection has been loaded by the axial force (compression) applied to the top-chord. The component of the load perpendicular to the top-chord (e.g. load of the purlins) has been omitted because its contribution to the failure of the node has been considered not significant.

After the tests to failure of the joints, the timber has been re-used to prepare other full dimension specimens for testing the shear strength according to EN 1193.

2.1 Timber and processes

Experimentation has been carried out on old timber elements, made of poplar (*Populus sp.*), fir (*Abies alba* Mill.) and spruce (*Picea abies* Karst.), coming from ancient buildings undergoing restoration in Tuscany. The information collected about the structures in restoration dated the beams more than fifty years old: poplar beams which have been under load from the last years of the XIX century, while fir and spruce date from the Forties [Bevilacqua, 2001]. This material has been used to realize six full dimension heel joints of different wood:

- 3 joints of poplar,
- 2 joints of fir,
- 1 joint of spruce.

Three further joints have been made with some pieces of more recent chestnut beams (*Castanea sativa* Mill.), used for some laboratory tests in the Nineties [Bonamini e Togni, 1999].

Formerly all the joists had been subjected to bending.

Then the beams were truncated to the length required to compose the joint: 100cm for the top-chord and 130cm for the bottom-chord; each single element has been planed and reduced to a section of 15x15cm. Top and bottom chords, made with the same wood, have been connected as shown in the first picture. The bottom-chords have been prepared with a notch about 4cm deep at a distance of 20cm from the end of the beam, to bear the top-chord.

The joint has been made to obtain a pitch of the top-chord of 30°.

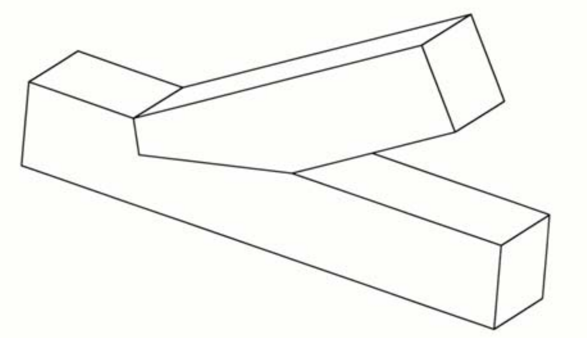


Figure 1. Illustration of the heel joint.

The moisture content of timber has been periodically controlled by an electric moisture meter; after a period of some months inside labs, during the tests the MC has been measured in the range 11-14%, close to the normal condition (12%).

2.2 Experimental device for testing joints

This experimental apparatus has been designed for testing the top-chord of the joints loaded axially, in vertical position, by consequence, due to the shape of members, the bottom-chord has a 30° slope on the vertical (60° on the horizontal) as shown in Figure n.2.

A steel device has been specifically designed and built [Thoma, 2002] to support the

bottom-chord during the loading phase of the test, and to load the top chord.
It has been made by two main parts entirely built with 20mm thick steel plates, properly stiffened by steel ribs: one horizontal base and one oblique plate. The flat plate has been provided with two wings to connect, through two swivel bolts, the oblique steel plate. Two sleeve couplings, with double screw, joining the two steel plates, have been designed to allow the changing of the slope of the oblique plate, rotating on the hinges.
This plate supports the bottom chord and the modification of its slope permits to uphold the top-chord in vertical position, so joints with different pitch could be tested as well.

The bottom-chords have been hung up to the high part of the steel device by means of six bolts, passing through two steel plates, connected to a rod, spherical joint ended and screwed to a small ribbed steel plate.

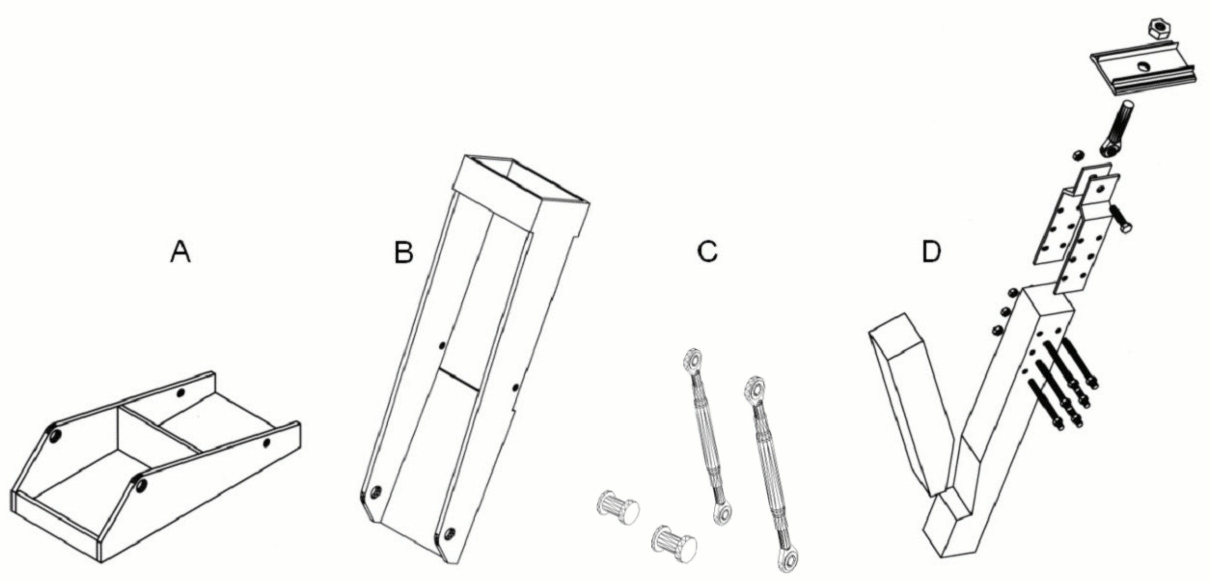
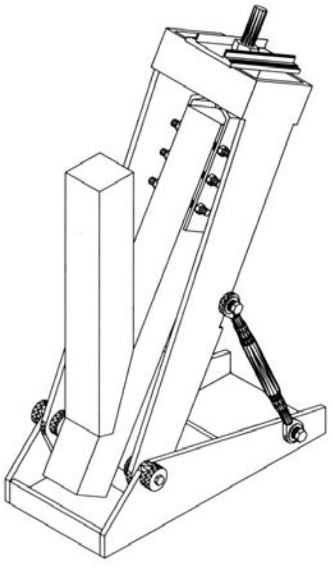
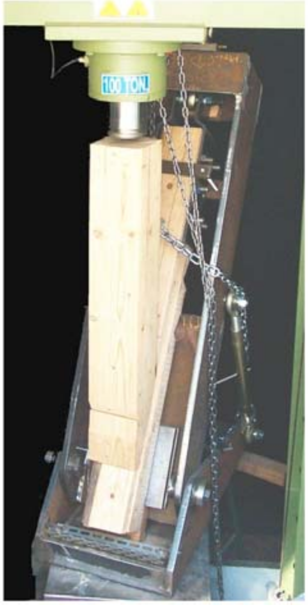


Figure 2. Main components of the device for testing: reinforced steel base (A), steel swivel frame with support plate (B), bolts and sleeve couplings, connecting the two steel frames (C), specimen with the structure to suspend the bottom-chord to the support frame B (D).

Under the notch, twin steel sheets, which can slip one against the other without friction, have been put as support of the beam so the bottom chord can elongate freely during the test and transfer the load perpendicular to the bottom chord to the plate of the frame B (Picture n.2).





The entire mechanism for testing has been fitted in a 1000kN hydraulic press, manually controlled, used as testing machine.

Figures 3, 4. Illustration of the joint on the support and a photo of the device installed in the testing machine (chains were for safety).

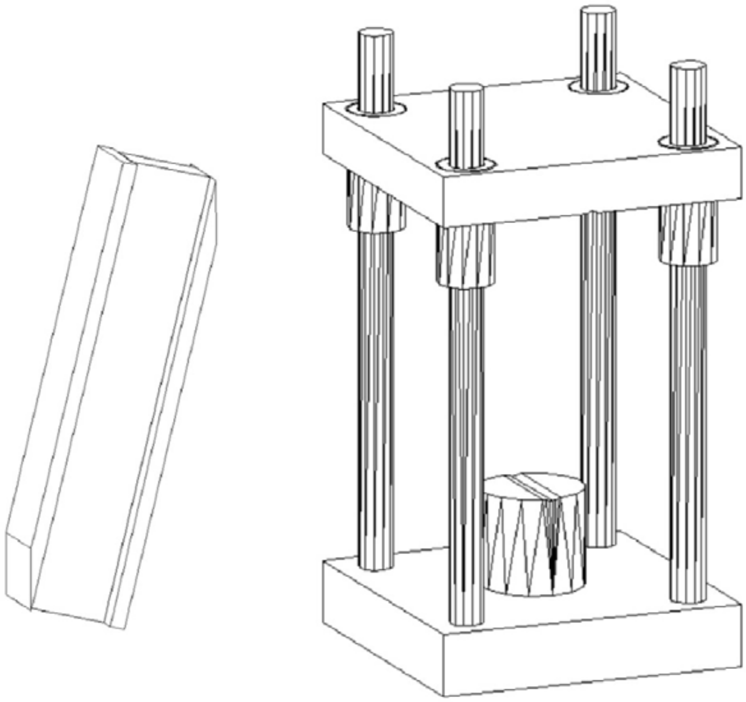
2.3 Measurement apparatus

During the tests, loads applied and strains have been measured using different devices: three load cells (working electrical strain gauges), supporting the steel base, connected to a digital recording system; LVD Transducers have been applied on the heel and on the end of the top-chord to record the relative displacements. The signals have been digitally recorded after an ADC[3] transformation.
The sequence of loads has been formulated as indicated by the standard UNI EN 380:1994.

2.4 Shear test

The shear tests on timber specimens have been performed according to EN 1193:1999[4]: specimens have been cut from the timber members used previously, excluding those parts which were affected by failure and close to high stressed portions. Each specimen has been planed to the cross section 32x55mm and cut to a length of 300mm. Then two steel plates, necessary to apply the load to the specimen, have been glued by means of epoxy resin. The plate-specimen-plate systems have been loaded along the diagonal, in line with the two steel corners. The other two corners of the steel plate have been previously profiled to obtain the chamfered ends. This particular shape of plates is necessary to avoid the increasing of the stress at the ends of specimen [Feldborg, 1991].
A steel frame[5] has been used to support the specimens and to correctly apply the load. A notch has been cut on the upper plate and on the cylindrical basement, to keep the correct position during the test, the steel plates glued to the specimen, edgeways. The two notches have been built on the same plane, in the geometrical centre of the frame, on line with the vertical axis and with the loading point.

52 specimens, differently distributed by species, have been prepared for testing.
As expected, during the test, a load component, perpendicular to the grain of the specimen, increases proportionally to the vertical load. The component is easily quantified because the angle between vertical axis (the direction of the load) and the longitudinal direction of the specimen (14.6 degree) has been determined geometrically.



Figures 5, 6. A specimen with steel glued plates and the loading steel frame

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[3] ADC: Analog to Digital Converter
[4] The rule EN 1193 has been withdrawn. Nowadays the shear test is enclosed in the reworking of EN 408.
[5] An ordinary die holder, whose parallel steel plates slide precisely on four steel columns, by means of ball bearing sleeves.

The load has been applied with a 200kN hydraulic universal testing machine, electronic controlled. The deformations have been measured with a LVD Transducer, the load has been measured with the load cell integrated inside the testing machine.

3. Results and discussions

3.1 Shear failures of heel joint

The full size truss joints failed by shear with the simple sliding of the heels, as expected, except in fir-2 and poplar-3, where a small portion of the cross section of the bottom-chord have split from the heel by a failure in tension, so that these failures can be described as a combination of shear and tension.

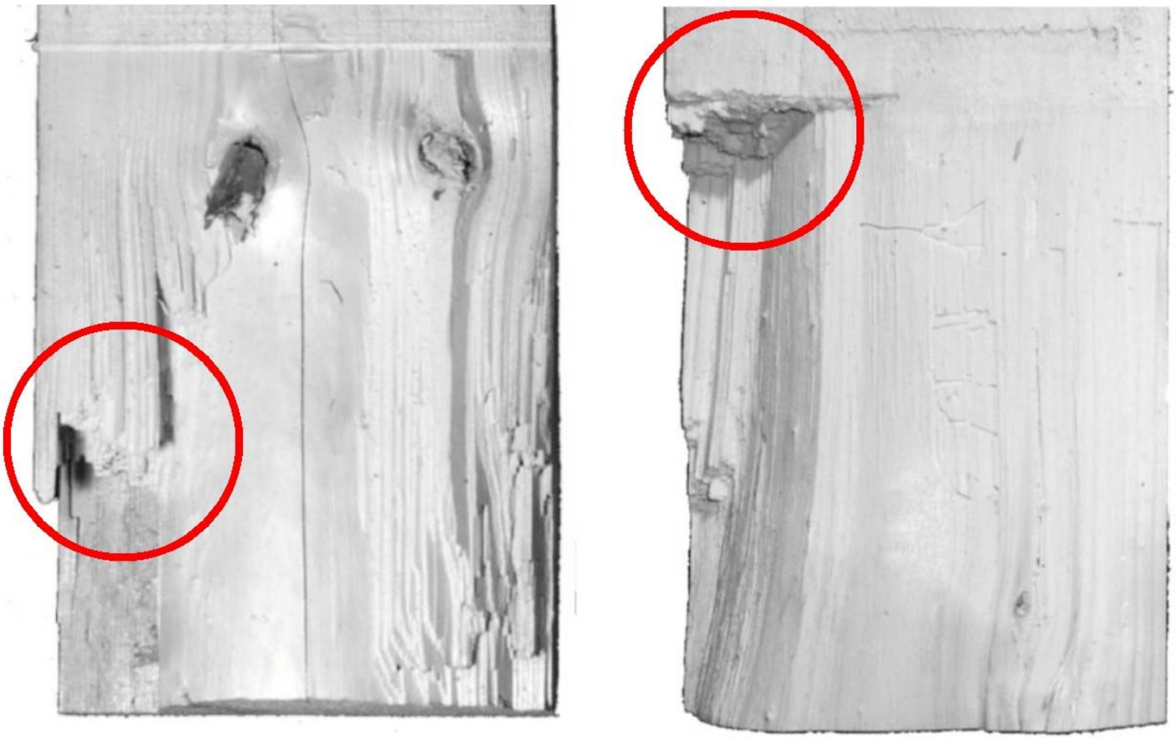

Figure 7. Partial failure in tension in two bottom-chords (after failure, view from above).

The sliding layer has never coincided with the plane corresponding to the nominally stressed surface, because of the geometry of the wood anatomic elements, the direction of the growth rings as well as the presence and the distribution of defects and anomalies in the timber of the bottom chord.
Shear failures have been developed on longitudinal surfaces as following:
- three resulted on one plane parallel to the rings (LT plane[6]), as in fir-1, fir-2 and poplar-3 where more than 50% of the shear surfaces were along one ring, on the limit of the annual ring between spring- and summer-wood,
- three others perpendicular to the ring (LR plane[7]), as shown in chestnut-2, poplar-2 and poplar-1,
- the last three ones combining the two planes like a "stairs shape", as in spruce, chestnut-1 and chestnut-3, as the weakness of the 2 planes were similar, but firstly for geometrical reasons.

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[6] Longitudinal-tangential plane
[7] Longitudinal-radial plane

Figure 8. More than 50% shear along the ring

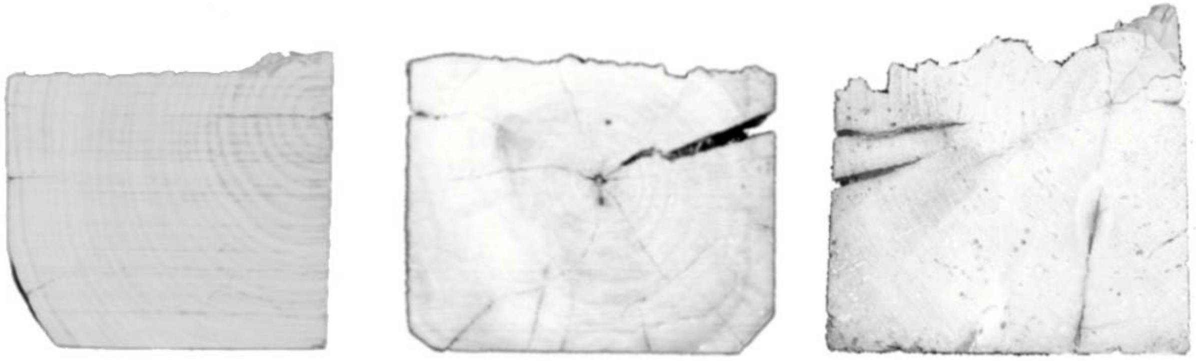

Figure 9. More than 50% shear along the LR plane (radial and subradial surface)

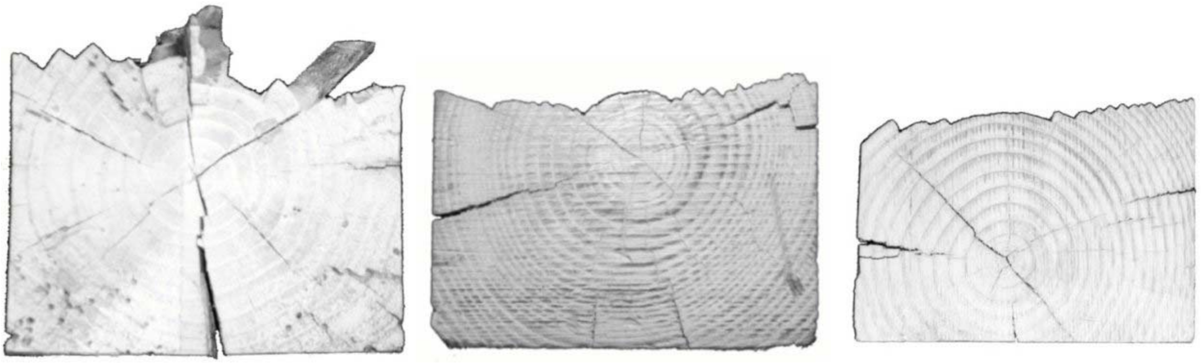

Figure 10. Intermediate shear failures

The failures have developed following the grain and its general longitudinal slope. In all the softwood specimens tested and in poplar-1 the grain deviation due to the knots and in poplar-3 the local deviation of the grain, meant a general increase of the dimensions of sliding surfaces.

Figure 11. General slope of the grain

The same variety of behaviour has been observed in the shear failure of the EN 1193 specimens, principally depending on the three-dimensional orientation of the wood.

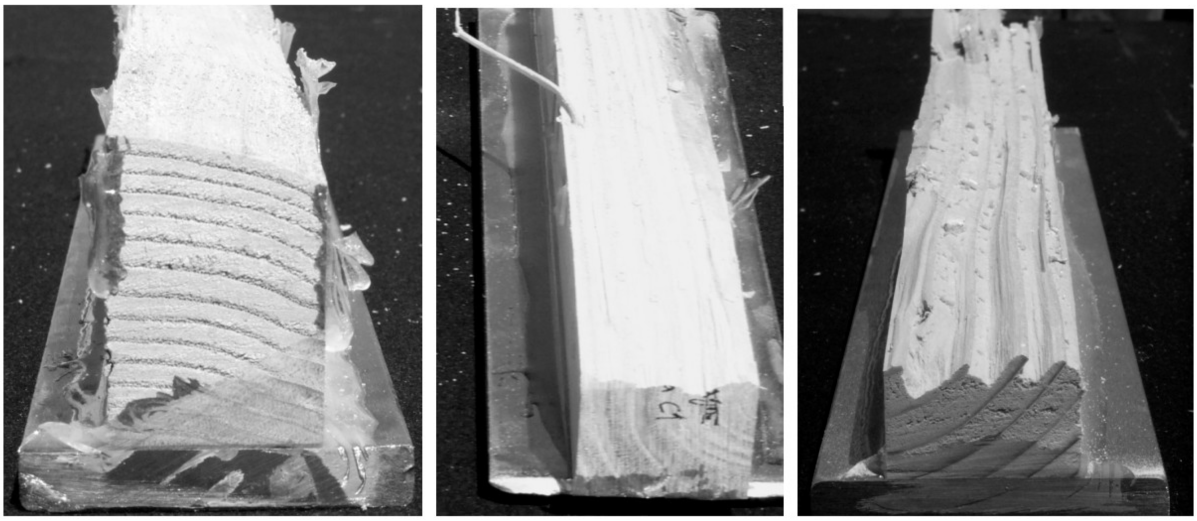

Figure 12. Shear on a ring (LT plane), on a LR plane, and intermediate

Similarly the other defects have produced the same effect on the way failure has occurred, as local slope of the grain; some partial failures in tension have been also possible.

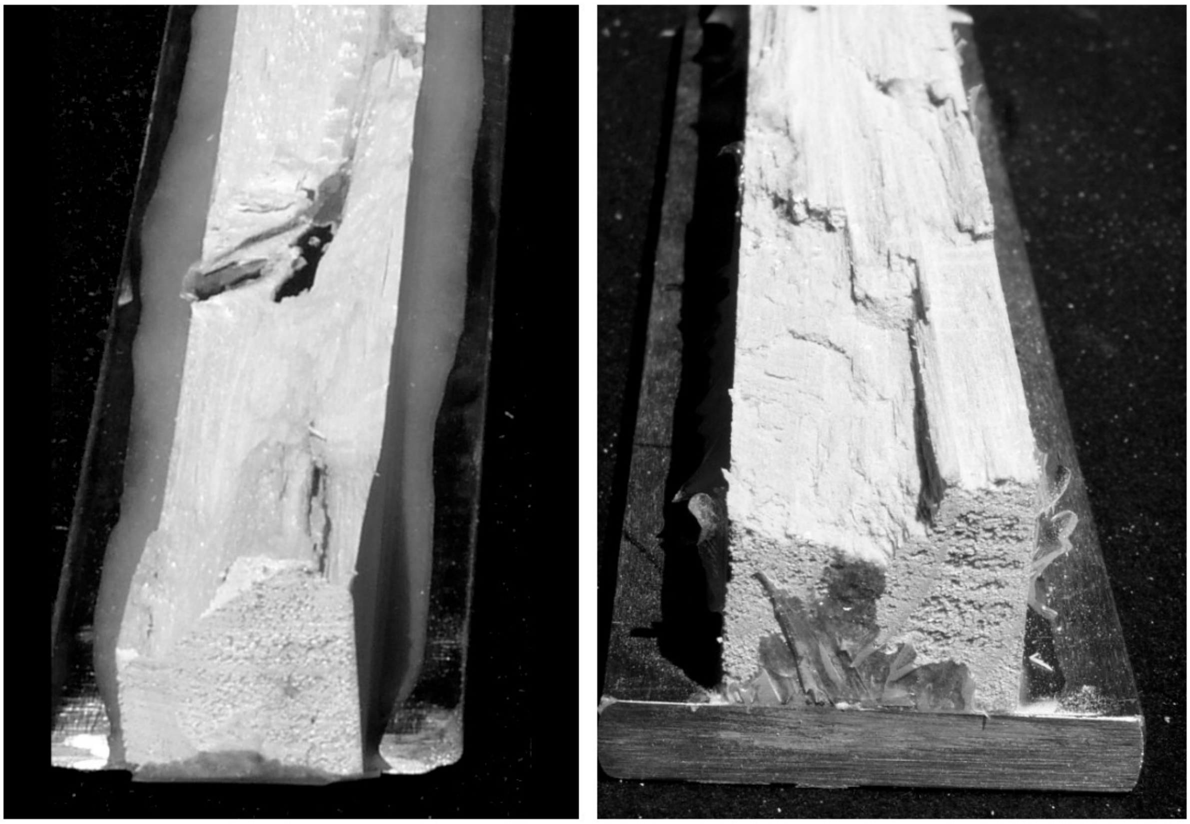

Figure 13. Local slope of the grain and partial failure in tension

At the same time some tunnels of wood-boring beetle**s** have been revealed on the failure shear surfaces.

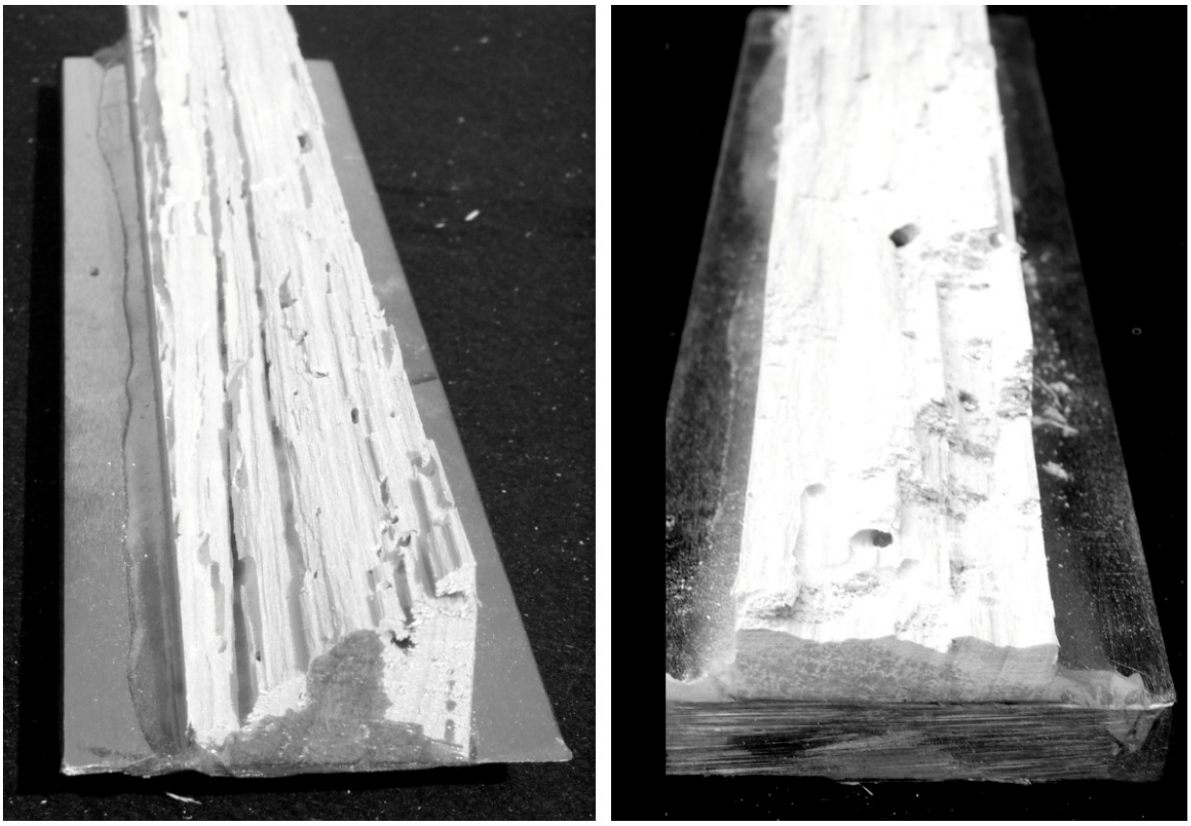

Figure 14. Tunnels of wood boring beetle**s** (Fam. *Anobidae*) parallel and perpendicular to the grain.

Here the images of the results and the synthesized description of the joints after the breaking tests have been listed.

Spruce: heel with 3 knots and 1 pitch pocket

Fir-1: nominal surface opposing to the stress, decreased by the large wane. Failure mainly on 1 only ring.

Fir-2: shear failure mainly on 1 ring; concurrent a partial failure on tension. 2 knots and consequent local grain deviation.

Poplar-1: longitudinal tunnels of wood boring beetles, 1 knot and the local grain deviation. Shrinkage fissure close to stressed surface.

Poplar-2: beetles longitudinal tunnels. Shrinkage fissure very close to stressed surface.

Poplar-3: local slope of the grain; concurrent partial failure on tension. Failure on 1 only ring.

Chestnut-1: shrinkage fissure close to the shear surface.

Chestnut-2: general slope of the grain. Shrinkage fissure close to the shear surface.

Chestnut-3: general slope of the grain.

3.2 Shrinkage fissures

Through the results of the tests it has been pointed out that the presence of some shrinkage fissures (typical, on a full size timber member containing the pith) by itself has no effect on the shear strength and on the way of failure. No "attraction effect" [8] on the shear surface by the nearby fissure has been found.

3.3 The values of shear strength

The strength values obtained by the full test of the joints have been compared with the

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[8] This expression has been used to describe the tendency of the shear breaking plane to join the shrinkage fissures of the beam. If the distance between shear plane and fissure is enough, the strength in tension of wood does not permit this joining. In the test the closest shrinkage fissure was in chestnut-1, at a distance of 11 mm from the shear plane.

results of EN 1193 test on timber specimens. The results showed that the values of the specimens were higher than the results of the heel joints[9]. The reason for the differences has to be attributed to the test procedure: specifically the component of the load perpendicular to the grain concurred to the strength cohesion of the specimens and made the shear strength higher.

The presence of tunnels due to wood boring beetles did not have any notable effects on the final strength. Also the way of failure (on LR, or LT plane, or intermediate) did not have particular consequences on the resulting values. The slope of grain and the knots seem to have an improving effect on strength, or differently a null effect; no reduction effect has been discovered due to these characteristics.

The Table 1 summary of the resulting shear data.

shear strength						
	heel joint		**specimens according to EN 1193**			
N/mm²	**nominal strength**	**actual strength**	**min**	**mean**	**max**	**n. of specimen**
spruce	3,27	3,01	4,50	5,43	6,25	7
fir-1	*2,48*	*3,29*	1,84	4,11	5,30	7
fir-2	3,87	3,63	5,21	6,08	7,15	5
poplar-1	3,38	3,13	3,03	5,13	6,72	13
poplar-2	3,22	3,01	3,70	4,96	6,10	10
poplar-3	2,76	2,60	-	-	-	-
chestnut-1	3,94	3,81	5,69	7,09	7,76	4
chestnut-2	3,20	2,79	6,58	7,40	7,63	5
chestnut-3	4,48	4,19	-	-	-	-

Table 1. Shear strength values

Actual strength values have been calculated measuring the effective breaking surface after the failure. Mainly these surfaces are greater than the nominal area, except for fir-1 in which wane reduced the nominal surface 24%.

Chart n.1 shows the experimental values compared with:

- the characteristic values listed in the Italian *visual strength grading rule* UNI 11035 (category S1, S2 and S3 for fir and spruce, S for poplar and S for chestnut), used as reference for the new Structural technical regulation [MIT, 2005],
- the characteristic values of the strength classes (EN 338) for softwood and poplar[10],
- the values obtained by the 1st class of *working stress design*[11] values by Prof. Guglielmo Giordano [Giordano *et al.*, 1999], multiplied by 2.75 for comparison (frequently used as a conversion factor [Uzielli, 2002]).

The chart demonstrates that all the experimental strength values (except fir-1) have been established to be higher than characteristics strength values of the 1st class of the recent technical rules, and than the 1st class of the working stress design values. The anomaly outcome of fir-1 was due to the particularly low quality of the timber component of the bottom-chord, in which the large wane had to be graded in the S3 grade (UNI 11035): the value was lower than S1, but satisfied S2 grade. Poplar-3 value is approximately equivalent to characteristic strength value of the grade S for poplar (UNI 11035).

Shear values achieved according to EN 1193 have been much higher than reference values.

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[9] After the tests of the heel joint, poplar-3 and chestnut-3 were not re-cut, because the remaining portions of the beams have been used for different tests.

[10] Chestnut was excluded by softwood strength classes. Recent studies [Bonamini, Togni, 1999] proved that EN 338 Strength Classes for hardwood are not able to represent correctly the mechanical properties of chestnut sawn timber.

[11] Allowable stress method

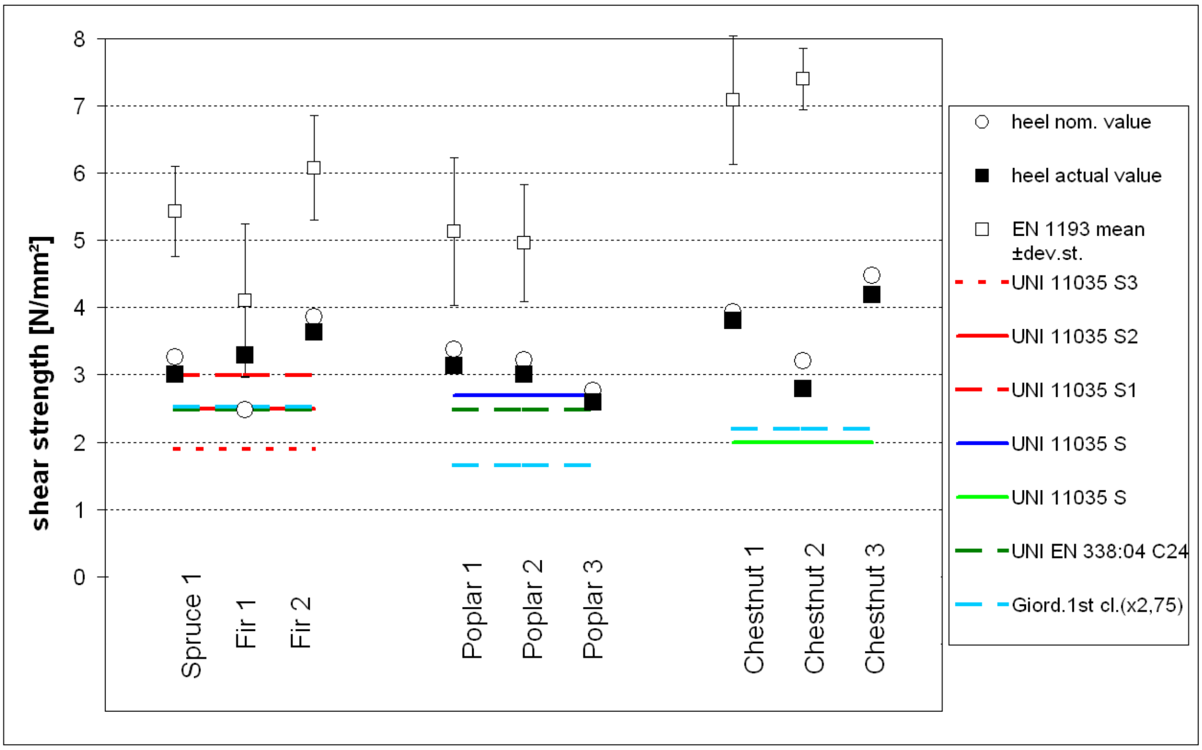

Chart 1. Comparison of the strength shear values

4. Conclusions

The laboratory tests on the different shear failures allow us to formulate the following conclusions:

- the shear failure of the heel joints has been widely verified: the behaviour of the full size timber specimens corresponds properly to the performances expected on structural timber elements. The test proposed is able to reproduce the actual *in situ* loading condition;
- the shrinkage fissures have no weakening effect on the shear strength of the heel, if they are not precisely in the breaking plane;
- some strength reducing characteristics, such as the slope of the grain and the knots, ordinarily evaluated for grading full size timber elements (e.g. UNI 11035), have no negative effects on shear strength; on the contrary wane, pitch pockets, tunnels of wood-boring beetles and, more generally, *"lack of wood",* can influence the correct shear performance;
- it is not possible to exclude a positive effect of knots and slope of the grain (local or general), on heel shear strength and stiffness of the full size truss joint;
- the strength values are not enough numerically to be statistically evaluated; they show that the timbers tested, although defects and insect tunnels are present, have shear mechanical performances comparable to the literature data about new timber;
- the shear strength values by EN 1193 specimens, resulting systematically higher, even if the way of shear breaking was very similar to the full size joints, indicates the limits of the test in the reproduction of shear stress on timber elements.

Further tests on full size heel joints on similar sized timber elements, but having different slopes of top-chord, species and quality, are programmed.

Acknowledgements

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