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Safety Assessment of Historic Timber Structural Elements / Barozzi, G.; Cosentino, N.; Lanzoni, L.; Tarantino, A. M.. - In: CASE STUDIES IN CONSTRUCTION MATERIALS. - ISSN 2214-5095. - 8:(2018), pp. 530-541. [10.1016/j.cscm.2018.04.006]

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# Safety assessment of historic timber structural elements

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## Abstract

Dealing with the safety assessment of existing buildings engineers often have to face the diagnosis of old timber structures. The current standards framework does not provide clear prescriptions about the evaluation of these kinds of structures, so the principal aim of this work **is** to outline an alternative methodology that leaves the concept of “Knowledge Level” and “Confidence Factor”, usually applied for existing buildings. An experimental campaign carried out on old timber joists supplied a sample of homogeneous data that were the support to the theoretical reasoning.

*Keywords:* Old timber; Mechanical behaviour; Safety assessment; Existing buildings; Confidence factor.

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## 1. Introduction

The assessment of existing structures is dealt by several studies. Starting from the assumption that **Confidence Factor** ( $CF$ ) values do not rest on solid theoretical foundation, Alessandri et al. [1] proposed a method for the calculation of two types of  $CF$ , one for the geometry and one for the materials. The subject of [1] was the reinforced concrete, for which the codes do not make a distinction between the two materials involved, that are completely different in terms of behaviour and in terms of techniques of investigation. The calibration of these new kinds of  $CF$  is done by using the Bayesian method that allows the inclusion of prior information and ex-post results (investigations on the materials and on the structural

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elements). This procedure is interesting because it gives the adequate relevance to the non-destructive tests that can be carried on existing structure. Franchin et al. [8] investigated the soundness of the  $CF$  by a simulation of the entire assessment procedure and the evaluation of the distribution of the assessment results on the acquired knowledge. Based on this distribution, a criterion is employed to calibrate new  $CF$  values. This procedure was applied to three reinforced concrete frame structures of increasing sizes, employing the nonlinear static and dynamic analysis methods and considering all Knowledge Levels ( $KLs$ ). An analysis of the reliability of the  $CF$  for seismic safety assessment was proposed in [18]. Such a study outlines a procedure for the assessment of the material properties by combining different sources of information. By using a Bayesian framework and considering the case of normal distributed strength, the obtained results lead to the conclusion that, when the prior knowledge and the new test data are in agreement, the necessary  $CF$  decreases as compared to the value obtained in the absence of a prior knowledge. An extensive literature is available on testing old timber elements. As an example, Piazza et al. [17] presented an experimental campaign on disassembled old roof beams, whereas [15], [3] and [7] deal with the correlation between non-destructive and destructive methods for the evaluation of timber properties. A testing activity on 130 years old timber beams can be found in [2]. Machado et al. [19] reports a review of the application of Visual Strength Grading ( $VSG$ ) and the way the information obtained can be combined with information provided by other NDT/SDT methods and [21] presents an experimental campaign on 20 old chestnut beams in order to define the correlations between bending modulus of elasticity in different scales of timber members in combination with visual grading analysis.

The aim of this paper is to define an alternative method for the safety assessment of old timber structures. As just the first step for the processing of a new methodology, this work has the objective of outlining the general way to progress, so some hypotheses are restrictive and some parameters are not taken into account. This work starts with an experimental activity based on destructive tests on old timber joists which were recovered from existing buildings. Before samples were tested it was performed the  $VSG$  according to the current Standards. The  $VSG$  was carried out in a more accurate way with respect to the prescriptions of the Standards in order to take into account some aspects that are relevant during the assessment *in situ* of timber

members. Both bending and compression tests were carried out. The results of the tests were elaborated in order to determine their characteristic values on the basis of the prescriptions given in [23], [24]. Then it was possible to perform a statistical analysis in order to evaluate the variance of the strength results for each type of test. The values obtained by tests were used as a support for the development of a new method for the evaluation of the design strength of old timber elements. A procedure based on the concept of "Knowledge Levels" ( $KL$ ) and "Confidence Factors" ( $CF$ ) is proposed based on the combination of  $VSG$  and the direct determination of strength provided by experimental tests on the samples.

It is important to remark that the proposed approach is directed to evaluate the design values of the strength based on two possible strategies: The first one through experimental test on samples extracted from the existing structure and the second one through the visual grading. The calibration of the coefficients involved in these two procedures is based on a study case; nevertheless, further study cases will improve the calibration itself in a future development of the research program. Finally, in order to promote a deeper comprehension of the material properties and to improve the reliability of the design parameters, a combined (mixed) procedure will be proposed. By this "third way", the design values are determined by using both the tests and the visual grading, so that the uncertainties and, in turn, the consequent  $CF$ s will be reduced, thus increasing the design strength.

## 2. Confidence Factors

### 2.1. The significance of the Confidence Factors

Once all the investigations on the structures have been carried out the main task becomes the definition of the design values. Concerning existing timber structures there is not a defined standardization, for this reason we refer to the Italian Standards [26]. The prescriptions of the Italian Standards [26] for the assessment of existing building are based on the concept of  $KL$ , that is defined by the quantity and the quality of the information gained. Depending on the  $KL$ , the  $CF$  plays the role of an additional safety coefficient that contains the uncertainties about the existing structure in terms of geometry, details and materials. In order to understand the physical significance



of  $CF$  it is necessary to focus on the difference between the determination of the design value in case of new buildings and in case of existing buildings. The design value for new structures  $f_d$  is obtained by the ratio between the characteristic value  $f_k$  of the material and the safety coefficient  $\gamma_M$

$$f_d = \frac{f_k}{\gamma_M}. \quad (1)$$

On the other hand, for the assessment of existing buildings the mean value  $f_m$  is used instead of the characteristic value, and it is reduced by the  $CF$

$$f_d = \frac{f_m}{\gamma_M CF}. \quad (2)$$

The safety coefficient  $\gamma_M$  assumes the same value in both cases. By comparing (1) and (2) it is clear that the ratio  $f_m/CF$  for the existing structures should be the equivalent of the  $f_k$  for the new

$$f_k = \frac{f_m}{CF}. \quad (3)$$

The eq (3) shows that  $CF$  should account for both the standard deviation of the material strength as well as the incomplete knowledge of the structure. As a matter of fact, the  $CF$  value defined by standard codes does not account for the strength variance, so that it results inappropriate. Since it was at the Authors' disposal a homogeneous sample of old timber joists, some destructive tests have been carried out in order to calculate the actual standard deviation of mechanical properties and to estimate the subsequent value of  $CF$  according to eq (3).

### 3. Testing materials and layout

#### 3.1. Samples

The test material derives from fifteen fir tree timber joists that were recovered from some buildings of the first half of the 20<sup>th</sup> century, damaged by the Emilia Romagna earthquake of May 2012 (see Fig. 1). These joists constituted the secondary warp of roof. Full-size joists were used for bending tests, from the undamaged rests the samples for compression tests were sawn<sup>1</sup>.

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<sup>1</sup>Recent contributions to the damage theory in the framework of finite elasticity can be found in [12], [13], [14].



Figure 1: Some images concerning a building damaged by the Emilia Romagna earthquake of May 2012: a) An image of the building, b)-d) images of the timber roof elements.



Figure 2: Subdivision of the sample.

### 107 3.2. Grading

108 Strength grading was carried out according to the Italian Standards [28],  
 109 [29]. Since for new products their load configuration is unpredictable, Stan-  
 110 dards prescribe to value each defect without considering its position through  
 111 the element, so it is the worst defect that defines the strength class. But the  
 112 probability of failure actually depends on load configuration that is known  
 113 for elements *in situ*, as well as for elements subjected to a bending test. Thus  
 114 it was decided to carry out also a visual grading of smaller portions of each  
 115 joist in order to observe in a more accurate way its behaviour at different  
 116 positions along the element. Standards were applied to each joist as a whole  
 117 and further to three 40 cm long marked sections (see Fig. 2).

### 118 3.3. Equipment and method

119 Bending and compression tests were carried out according to the Euro-  
 120 pean Standards [22] (see Figs. 3, 4).

121 Even though the Standards require samples of specific dimensions for each  
 122 type of compressional test it was decided to use samples of the same dimen-  
 123 sions (80x80x160 mm) for all tests, essentially because they were extracted  
 124 from the rests of the already broken joists and so there was not enough tim-  
 125 ber to obtain longer ones. However this limitation on dimensions of samples  
 126 length was established warranting the proportion between base and height  
 127 and at the same time avoiding any stability problems during the longitudi-  
 128 nal compression test. The three types of compression tests configuration are  
 129 represented in Fig. 4.

## 130 4. Results

131 The present Section deals with the main results provided by the exper-  
 132 imental tests on wood samples, with special reference to ultimate bending  
 133 and compression strengths.

134 Table 1 shows all test results in comparison with Standards values for  
 135 samples under bending. In "Failure Description" it is described the type of

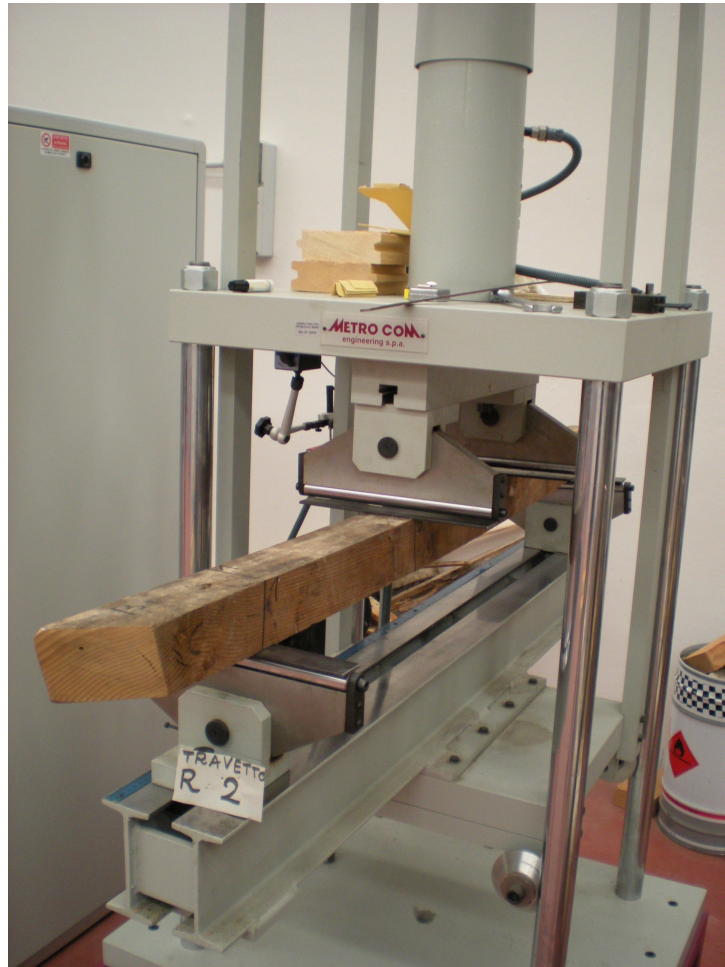


Figure 3: Bending test set up.



Figure 4: A timber sample under compression test (before the test). a) Longitudinal compression. b) Radial compression. c) Tangential compression.

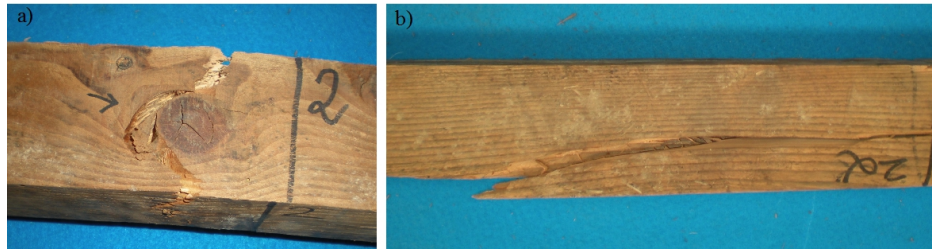


Figure 5: Samples after the bending test. a) Failure around a knot. b) Failure caused by fibre inclination.

136 failure by indicating the defect from which it started and its correspondent  
 137 class. When the failure started from a point where no visible defect was  
 138 present the class of the entire portion is reported<sup>2</sup>. Two main kinds of failure  
 139 are expected. The first kind is due to the presence of knots. Knots usually  
 140 pass through the beam interrupting or deviating the fibre flow and conse-  
 141 quently compromising local mechanical properties, as shown in Fig.5a. A  
 142 further kind of failure is fibre inclination, as shown in Fig.5b. During flexure  
 143 the lower part is subjected to traction parallel to the longitudinal axis. In  
 144 such a case, the failure was caused by the orthogonal component of traction  
 145 stress, in which direction the traction strength is lower.

146 The results for the longitudinal, tangential and radial compression tests  
 147 are listed in Tables 2, 3 and 4, respectively.

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<sup>2</sup>Timber structural elements can be subjected to relevant viscous effect. For their computation the approach reported in [4], [5], [6] can be performed.

Joist n.	UNI 11035	Test values			UNI 11035	Failure description
	Class	$Q_{max}$ (kN)	$v$ (mm)	$f_m$ (MPa)	$f_{m,k}$ (MPa)	
1	S2	14.74	13.48	34.55	25	Knot in 1D (S2)
2	S3	7.29	13.22	17.09	18	Knot in 1D (S3)
3	S2	17.21	19.33	40.33	25	Knot in 2B (S2)
4	S3	8.41	15.96	19.71	18	Knot in 2A (S3)
5	S3	14.85	15.04	34.80	18	Knot in 2C/D (S2)
6	S2	13.64	19.38	31.97	25	Knot in 2C (S2)
8	S2	15.74	19.16	36.89	25	Knot in 2B/D (S2)
9	S3	12.88	19.38	30.19	18	Middle of 2C (S3)
10	S2	16.32	18.49	38.24	25	Fibre in 2C/D (S2)
11	S2	13.26	19.93	31.07	25	Between 1 and 2 (S2)
12	S3	11.18	13.13	26.19	18	Knot in 2A/C (S2)
13a	S2	14.10	18.51	33.06	25	Knot in 1A (S2)
13b	S3	13.68	17.71	32.03	18	Knot in 2B (S2)
14	S2	10.90	18.96	25.54	25	Knot in 2B (S2)
15	S3	12.82	15.47	30.04	18	Knot in 2C (S3)

Table 1: Bending tests results.

Sample n.	Class*	Test values		UNI 11035
	UNI 11035	$Q_{c,0,max}$ (kN)	$f_{c,0}$ (MPa)	$f_{c,0,k}$ (MPa)
1	S2	219.70	34.33	21
2	S3	131.95	20.62	18
3	S2	210.98	32.97	21
4	S3	202.73	31.66	18
5	S3	145.02	22.69	18
6	S2	237.58	37.12	21
9	S3	176.84	27.63	18
10	S2	186.85	29.20	21
11	S2	204.49	31.95	21
12	S3	173.01	27.03	18
13a	S2	203.28	31.76	21
15	S3	243.62	38.07	18

Table 2: Longitudinal compression test results. \*Samples are divided into the two classes S2 and S3 considering the class of the entire joist from which they were extracted.



Sample n.	Class* UNI 11035	Test values		UNI 11035 $f_{c,90,k}$ (MPa)
		$Q_{c,90t,max}$ (kN)	$f_{c,90t}$ (MPa)	
1	S2	61.70	4.82	2.5
2	S3	43.10	3.37	2.2
3	S2	46.00	3.59	2.5
4	S3	47.30	3.70	2.2
5	S3	26.90	2.10	2.2
6	S2	45.20	3.53	2.5
9	S3	50.10	3.91	2.2
10	S2	33.20	2.59	2.5
11	S2	33.10	2.59	2.5
12	S3	38.80	3.03	2.2
13a	S2	35.00	2.73	2.5
15	S3	48.00	3.75	2.2

Table 3: Tangential compression test results. \*Samples are divided into the two classes S2 and S3 considering the class of the entire joist from which they were extracted.

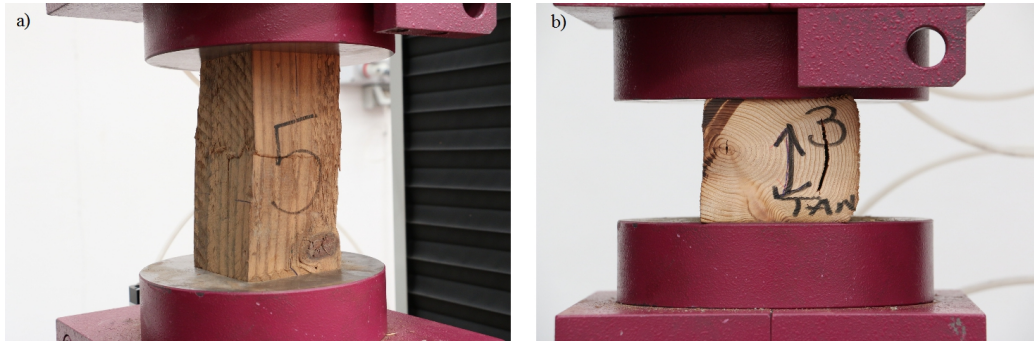


Figure 6: A sample under a) longitudinal compression and b) tangential compression.

Sample n.	Class*	UNI 11035	Test values		UNI 11035
			$Q_{c,90r,max}$ (kN)	$f_{c,90r}$ (MPa)	$f_{c,90,k}$ (MPa)
1		S2	36.20	2.83	2.5
2		S3	52.30	4.09	2.2
3		S2	36.80	2.88	2.5
4		S3	55.10	4.30	2.2
5		S3	25.50	1.99	2.2
6		S2	41.20	3.22	2.5
9		S3	56.50	4.41	2.2
10		S2	40.10	3.13	2.5
11		S2	34.50	2.70	2.5
12		S3	34.70	2.71	2.2
13a		S2	27.50	2.15	2.5
15		S3	45.00	3.52	2.2

Table 4: Radial compression test results. \*Samples are divided into the two classes S2 and S3 considering the class of the entire joist from which they were extracted.

148 A sample under longitudinal compression test is shown in Fig.6a. In the  
149 first case, the fibre started collapsing in correspondence of a quite horizon-  
150 tal plane until the failure of the entire section. At the same time also the  
151 zone around the knot was subjected to a strong deformation due to the fact  
152 that knots fibre is basically orthogonal to fibre direction and so it offers a  
153 compression strength significantly lower with respect the rest of the element.

154 In the case of tangential compression, the horizontal traction stress that  
155 takes place in the middle zone causes the breaking of the sample for separa-  
156 tion of the ring surfaces (see Fig.6b), where traction strength is lower.

157 A sample under radial compression is shown in Fig.7. In this situation,  
158 the accentuate vertical shift is combined with a strong lateral expansion that  
159 caused the expulsion of the softer material. Note also that, for the sample  
160 at hand, a strong fracture started close to the knot.

#### 161 4.1. Observations on the results

162 A comparison between two different graphs regarding the bending tests is  
163 now presented, the first one was obtained by considering the entire elements  
164 grading, the second one shows the strength values reorganized with respect  
165 to the class of the defect from which failure started. In both plots (Fig. 8a  
166 and 8b) the values are distinguished in two sets, one for the visual strength





Figure 7: A sample under radial compression.

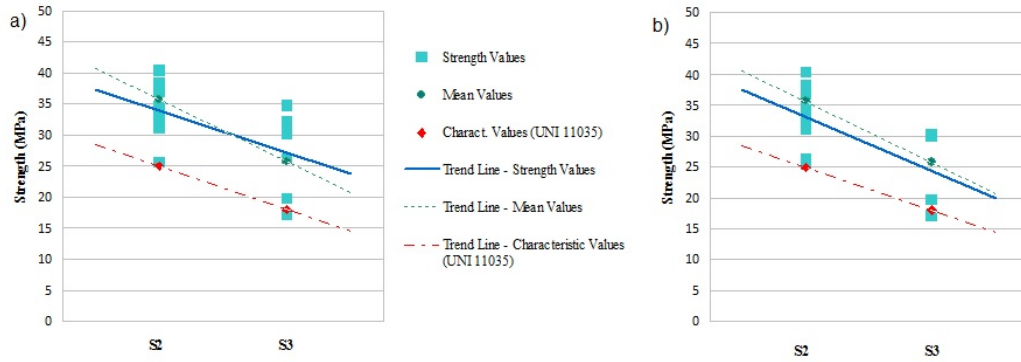


Figure 8: Ultimate bending strengths provided by the experimental tests. a) Subdivision between S2 and S3 considering full-size joists. b) Subdivision between S2 and S3 considering the class of the point from which failure started.

grade S2 and one for S3. The red square corresponds to the characteristic value according to the visual grading standard, the green circle represents the estimated mean value, that is obtained dividing the characteristic value by 0.7.

It can be observed that in Fig. 8a the experimental "trend line" (blue solid line) is quite far from the theoretical "trend line" (green dotted line). On the other hand, in Fig. 8b the theoretical and the experimental "trend lines" are quite parallel, even if the experimental mean strength is a little smaller than the theoretical one. Hence, not only the presence of a defect is important for the ultimate strength but also its location through the element is fundamental, and it should be taken into account by the operator during the visual inspection of the structure. For example the heads of the bottom chord of a timber truss or the point of maximum bending stress in a beam are the most dangerous locations for a defect. Another aspect that should be underlined is that the results obtained are slightly lower than the values given by [29]. This is because these joists are old and a loss of their capacity is expected, so the values given by [29] would overestimate the strength of old timber joists. For this reason the introduction of a coefficient higher than 1 that corrects the standards values appears to be necessary.

## 5. Determination of the characteristic values

The characteristic value of strength was determined according to [24]. The 5% fractile of a property  $X$  should be found by using the general formula:

$$X_k = m_X (1 - k_n V_X), \quad (4)$$

where  $m_X$  is the mean of the  $n$  samples results,  $k_n$  is the characteristic fractile factor and  $V_X$  is the coefficient of variation of  $X$ . The coefficient  $k_n$  depends on the number of samples  $n$  and on the  $V_X$ .

n.	1	2	3	4	5	6	8	10	20	30	$\infty$
$V_X$ unknown	-	-	3.37	2.63	2.33	2.18	2.00	1.92	1.76	1.73	1.64

Table 5: From the Eurocode.

According to [23], if samples height is less than 150 mm the characteristic values of bending strength should be adjusted to the reference condition by dividing by the coefficient  $k_h$

195 -  $k_h = \min((\frac{150}{h})^{0.2}, 1.3)$ , where  $h$  is the height of the samples.

196 Since the bending test set up is not in line with [22] (i.e. span  $l=18h$  and  
197 distance between inner load points at  $=6h$ ), then the 5-percentile bending  
198 strength shall be adjusted by dividing by the factor

199 -  $k_l = (\frac{45h}{l_{et}})^{0.2}$ ,

200 with

201  $l_{et} = l + 5a_f$

202 where  $a_t$  and  $l$  assume the respective values for the test. In this case,  $k_l$  was  
203 not applied because the sample geometry was perfectly in line with [22].

204 Therefore, the characteristic values of bending strength were obtained by  
205 the following formula:

$$f_{mk} = f_{m,Mean} \frac{1 - k_n V_s}{k_h}. \quad (5)$$

All (MPa)	S2 (MPa)	S3 (MPa)
16.68	21.73	11.86

Table 6: Bending strength characteristic values. “All” stands for “All samples without distinction between classes”.

206 Similarly, the characteristic values of longitudinal, tangential and radial  
207 compression strengths were obtained by these formulas

$$f_{colk} = f_{col,Mean} (1 - k_n V_f), \quad (6)$$

$$f_{c90tk} = f_{c90t,Mean} (1 - k_n V_f), \quad (7)$$

$$f_{c90rk} = f_{c90r,Mean} (1 - k_n V_f). \quad (8)$$

	All (MPa)	S2 (MPa)	S3 (MPa)
Longitudinal compression	20.48	27.0	14.21
Tangential compression	1.92	1.41	1.85
Radial compression	1.68	1.99	1.39

Table 7: Compression strength characteristic values. “All” stands for “All samples without distinction between classes”.

## 6. Statistic analysis

Some observations on the statistical trend of the results are now presented. Because of the small number of values it was not possible to better perform a statistical distribution and the Gaussian distribution was assumed

$$F(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{\frac{-(x-f_{Mean})^2}{2\sigma^2}}, \quad (9)$$

where  $f_{Mean}$  is the arithmetic mean,  $\sigma^2$  is the variance and  $\sigma$  is the standard deviation.

S2				S3			
$f_{m,Mean}$	$\sigma$	$V_f$	$f_{mk}$	$f_{m,Mean}$	$\sigma$	$V_f$	$f_{mk}$
33.96	4.66	0.14	21.79	27.15	6.55	0.24	11.86
32.89	2.68	0.08	27.06	27.95	6.30	0.23	14.21
3.31	0.87	0.26	1.41	3.31	0.67	0.20	1.85
2.82	0.38	0.13	1.99	3.50	0.97	0.28	1.39

Table 8: Bending, longitudinal compression, tangential compression, radial compression strengths.

## 7. Considerations about the Confidence Factors

In order to show the inadequacy of the methodology indicated by [26], the value  $CF$  is calculated with the test results in respect of relation (3)

$$CF' = \frac{f_{mTest}}{f_{kTest}}, \quad (10)$$

where  $f_{mTest}$  and  $f_{kTest}$  are respectively the mean value and the characteristic value (for the determination of  $f_{kTest}$  see Section 4). Table 9 shows

		$CF'$
Bending	All	1.85
	S2*	1.56
	S3*	2.28
Long Comp	All	1.49
	S2**	1.22
	S3**	1.97
Tang Comp	All	1.73
	S2**	2.34
	S3**	1.79
Rad Comp	All	1.88
	S2**	1.42
	S3**	2.53

Table 9: Values of  $CF'$ . Here “All” stands for “All samples without distinction between classes”. \*Subdivision depending on the grading done after failure. \*\*Subdivision depending on the visual grading

the results obtained for  $CF'$ . The laboratory campaign is comparable to a comprehensive *in situ* inspection and comprehensive *in situ* testing (KC3). But the results of  $CF'$  are much larger than 1, and even than 1.35 (that are the extreme values suggested by the standards, depending on the  $KC$ ). This means that the  $CF$ , as it is formulated by the Standards, actually does not cover all the uncertainties due to the limited knowledge of the existing buildings. In the seismic case the great dispersion around the mean value is compensated by the phenomena of stress redistribution, but this is not true for the static case where the failure of one element could compromise the entire structure, which is usually isostatic. For this reason, even though for existing buildings safety levels are accepted to be lower than for the design of new buildings, this way to define the design value appears improper.

## 8. Proposal

Since the design value  $f_d$  should be the result of the procedure, the methodology proposed consists in its direct determination leaving the concept of  $KL$ . A new method, easily applicable in the context of an *in situ* inspection, which includes two possibilities, was developed for the determination of  $f_d$ .

### 237 8.1. Design value obtained by testing

238 This procedure consists in the execution of some tests on structural ele-  
 239 ments, that could be destructive or not, depending on the case considered.  
 240 The design value could be obtained according to [24]

$$f_{dTest} = \frac{f_{kTest}}{\gamma_M} \eta_d, \quad (11)$$

241 where  $f_{kTest}$  is the characteristic value of the test results;  $\eta_d$  is the design  
 242 value of the conversion factor and it includes some corrective factors which  
 243 take into account the effects of volume and scale, humidity, temperature  
 244 and load duration [9]. The effects of volume and scale have already taken  
 245 into account by the corrective factors given by [23], the remaining factors  
 246 (humidity, temperature and load duration) are considered included into the  
 247 coefficient  $k_{Mod}$ , [26].  $k_{Mod}$  is determined by the Class of Load Duration, and  
 248 the Service Class. Thus relation (11) becomes

$$f_{dTest} = \frac{f_{kTest}}{\gamma_M} k_{Mod}. \quad (12)$$

### 249 8.2. Design value obtained by visual grading

250 The characteristic value  $f_{kClass}$  is obtained by *SVG* according to the  
 251 approach presented in Section 4. The prescriptions of the Italian Standards  
 252 [28] and [29] should be applied considering that the load configuration is  
 253 known and so it should be important to focus on the most stressed zones  
 254 in order to avoid the underestimation of the strength of the timber member  
 255 due to the presence of a defect in low stress zones, so taking into account  
 256 the indications of the Italian Standards [27]. The design value should be  
 257 determined as follows

$$f_{dClass} = \frac{f_{kClass}}{\gamma_M k_c} k_{Mod}, \quad (13)$$

258 where  $k_c$  should be applied to  $f_{kClass}$ . In fact,  $f_{kClass}$  is defined for new  
 259 timbers, so it should be better to correct it with the coefficient  $k_c$ , which  
 260 takes into account the divergence between class values of new timber and  
 261 real strength of old timber. In the present work  $k_c$  was calculated using the  
 262  $f_{kTest}$  obtained by the tests executed in order to outline a way to process.  
 263 In a future step the determination of  $k_c$  with a sufficiently high number of

264 tests on homogeneous samples will be performed, so that each homogeneous  
 265 material class could have its own  $k_c$

$$k_c = \frac{f_{kClass}}{\gamma_M f_{dInf}} k_{Mod}. \quad (14)$$

266 Then, by substituting (12) in (14) and considering that in laboratory condi-  
 267 tions  $k_{Mod} = 1$ , relation (14) becomes

$$k_c = \frac{f_{kClass}}{f_{kInf}} = \frac{f_{kClass}}{f_{kTest}}. \quad (15)$$

		$k_c$
Bending	All	-
	S2*	1.15
	S3*	1.52
Long Comp	All	-
	S2**	0.77
	S3**	1.27
Tang Comp	All	-
	S2**	1.77
	S3**	1.19
Rad Comp	All	-
	S2**	1.26
	S3**	1.59

Table 10: Values of  $k_c$ .

### 268 8.3. A mixed procedure to determine the design values

269 Based on the experience of the Authors, and to promote a deeper com-  
 270 prehension of the material properties, we finally suggest to calculate both  
 271  $f_{kClass}$  and  $f_{kTest}$  and choose the maximum between them, providing that

$$f_d < 1.3 \min(f_{dClass}, f_{dTest}). \quad (16)$$

272 The coefficient 1.3 (whose value could be better calibrated in further  
 273 developments of the research) is introduced to account for both: (i) the  
 274 possibility to accept, in existing buildings, a low safety level than in new  
 275 buildings, as also stated by Italian Standards; (ii) the fact that the safety  
 276 coefficient  $\gamma_m$ , calibrated for new building, results overestimated for existing  
 277 building because the execution uncertainties have been already overcome.

## 9. Conclusion

The tests carried out were the support for the outlining of a new methodology for the assessment of existing timber structures. In this context some considerations were developed about the role of the  $CF$ s defined according to [26].  $CF$  depends on the  $KL$  gained by the operator in terms of quantity and quality of information collected about the existing structure and it is applied to the mean strength as an additional safety coefficient. Making a comparison between the determination of the design value for new structures and for existing structures and by using the results provided by the experimental tests, the inadequacy of the  $CF$  values was demonstrated. Furthermore the Standards does not differentiate  $CF$  values with respect to the type of structure, the size of timber members, the essence of timber or its class. Therefore the idea is to follow a different approach, based on the direct determination of the design value. The methodology proposed comprises two possibilities.

- *Testing*: It consists in carrying out tests on timber members that constitute the existing structure. The design value is calculated following [24]. This procedure was adapted to the specific case of timber samples (see Section 8.1), by using the correction coefficients defined both in the European Standards [23] and Italian Standards [26].
- *Visual Strength Grading*: The visual grading has to be carried out according to prescriptions reported in [28] and [29] but with an approach that should be similar to that defined by [27]. The idea is to do the visual grading giving more importance to the defect individuated into the most stressed zones. In fact, as it emerged from the tests, the performance in terms of strength strictly depends on the localization of the most important defects and not simply on their presence through the element (see Section 4.1). Another aspect to take into account is the fact the  $f_{kClass}$  were defined for new timber, in fact from the tests it came out that there is a difference between the strength values actually reached by the old timber samples and those provided by Standards. For this reason  $f_{kClass}$  should be reduced with an additional coefficient called  $k_c$ . Currently the values of  $k_c$  were calculated with the results obtained by testing in order to give a way to process that should be developed in the future with several tests. The objective is to obtain  $k_c$  values for each type of strength (bending, compression orthogonal and parallel to grain), timber species and classes.



314 Finally, based on the experience and to promote a deeper comprehension  
315 of the material properties, the Authors suggested a mixed procedure to de-  
316 termined the design values by using both the tests and the visual grading.  
317 By this "third way", the design values are determined by using both the tests  
318 and the visual grading, so that the uncertainties and the consequent CFs will  
319 be reduced, thus increasing the design strenght.

## 320 Acknowledgements

321 Financial support from the Italian Ministry of Education, University and  
322 Research (MIUR) in the framework of the Project Project PRIN "COAN  
323 5.50.16.01" (code 2015JW9NJT) is gratefully acknowledged.

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