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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.

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

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

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

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

71 2. Confidence Factors

72 2.1. The significance of the Confidence Factors

73 Once all the investigations on the structures have been carried out the
74 main task becomes the definition of the design values. Concerning existing
75 timber structures there is not a defined standardization, for this reason we
76 refer to the Italian Standards [26]. The prescriptions of the Italian Standards
77 [26] for the assessment of existing building are based on the concept of KL ,
78 that is defined by the quantity and the quality of the information gained. De-
79 pending on the KL , the CF plays the role of an additional safety coefficient
80 that contains the uncertainties about the existing structure in terms of geom-
81 etry, details and materials. In order to understand the physical significance

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

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

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

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

88 The safety coefficient γ_M assumes the same value in both cases. By compar-
 89 ing (1) and (2) it is clear that the ratio f_m/CF for the existing structures
 90 should be the equivalent of the f_k for the new

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

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

99 **3. Testing materials and layout**

100 *3.1. Samples*

101 The test material derives from fifteen fir tree timber joists that were re-
 102 covered from some buildings of the first half of the 20th century, damaged
 103 by the Emilia Romagna earthquake of May 2012 (see Fig. 1). These joists
 104 constituted the secondary warp of roof. Full-size joists were used for bend-
 105 ing tests, from the undamaged rests the samples for compression tests were
 106 sawn¹.

¹Recent 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

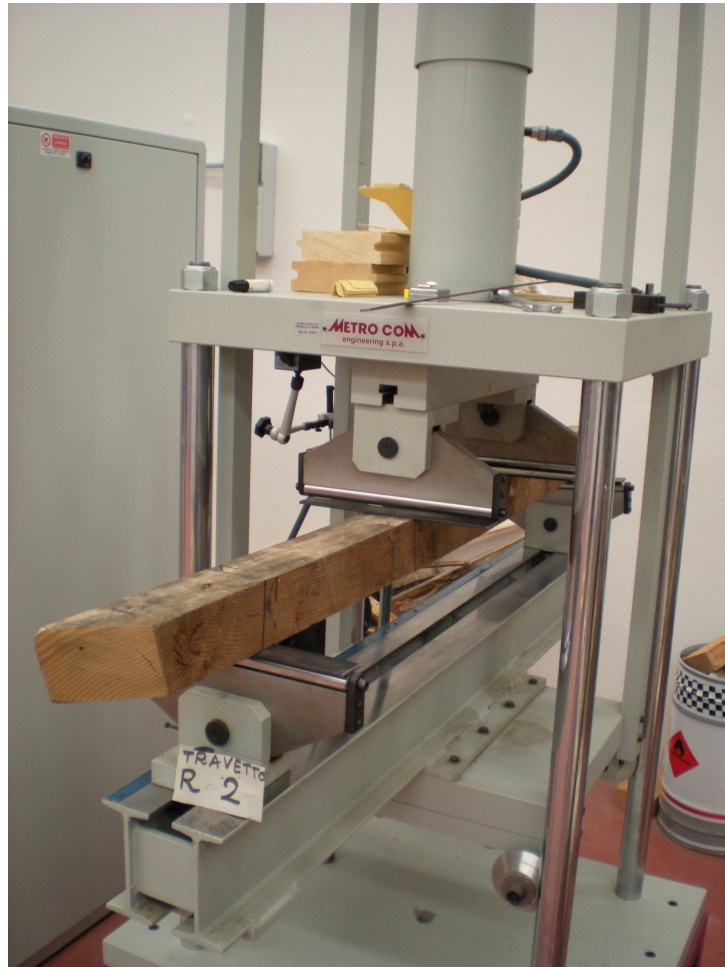


Figure 3: Bending test set up.



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

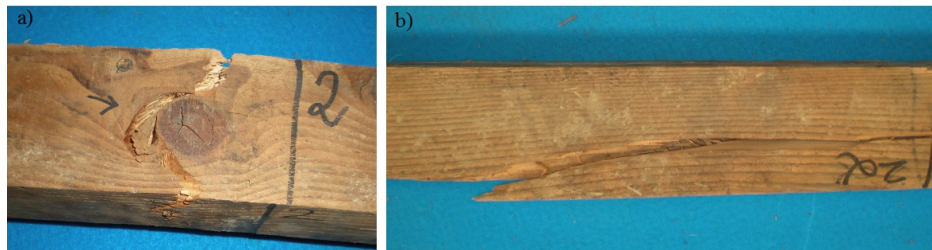


Figure 5: Samples after the bending test. a) Failure around a knot. b) Failure caused by fibre inclination.

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 reported². 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.

²Timber 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
		$Q_{c,90t,max}$ (kN)	$f_{c,90t}$ (MPa)	$f_{c,90,k}$ (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.

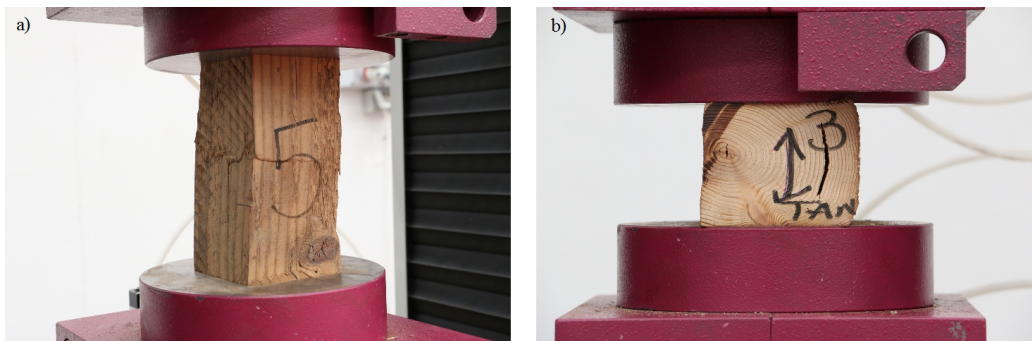


Figure 6: A sample under a) longitudinal compression and b) tangential compression.

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.

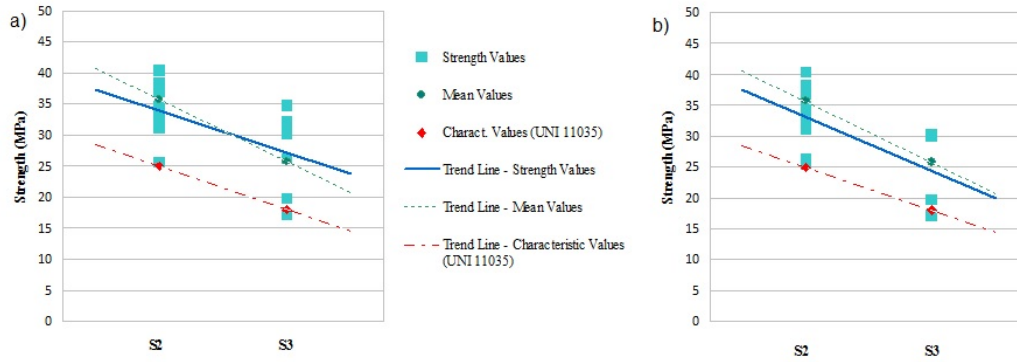


Figure 8: Ultimate bending strengths provided by the experimental tests. a) Subdivision between S2 and S3 considering full-size joists. b) Subdivision between S2 and S3 considering the class of the point from which failure started.

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

171 It can be observed that in Fig. 8a the experimental "trend line" (blue
 172 solid line) is quite far from the theoretical "trend line" (green dotted line).
 173 On the other hand, in Fig. 8b the theoretical and the experimental "trend
 174 lines" are quite parallel, even if the experimental mean strength is a little
 175 smaller than the theoretical one. Hence, not only the presence of a defect is
 176 important for the ultimate strength but also its location through the element
 177 is fundamental, and it should be taken into account by the operator during
 178 the visual inspection of the structure. For example the heads of the bottom
 179 chord of a timber truss or the point of maximum bending stress in a beam
 180 are the most dangerous locations for a defect. Another aspect that should
 181 be underlined is that the results obtained are slightly lower than the values
 182 given by [29]. This is because these joists are old and a loss of their capacity
 183 is expected, so the values given by [29] would overestimate the strength of
 184 old timber joists. For this reason the introduction of a coefficient higher than
 185 1 that corrects the standards values appears to be necessary.

186 5. Determination of the characteristic values

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

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

189 where m_X is the mean of the n samples results, k_n is the characteristic fractile
 190 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	∞
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.

191 According to [23], if samples height is less than 150 mm the characteristic
 192 values of bending strength should be adjusted to the reference condition by
 193 dividing by the coefficient k_h
 194

195 - $k_h = \min\left(\left(\frac{150}{h}\right)^{0.2}, 1.3\right)$, 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 - $k_l = \left(\frac{45h}{l_{et}}\right)^{0.2}$,

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

$$f_{c0lk} = f_{c0l,Mean} (1 - k_n V_f), \quad (6)$$

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

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

	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”.

208 6. Statistic analysis

209 Some observations on the statistical trend of the results are now pre-
 210 sented. Because of the small number of values it was not possible to better
 211 perform a statistical distribution and the Gaussian distribution was assumed

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

212 where f_{Mean} is the arithmetic mean, σ^2 is the variance and σ is the standard
 213 deviation.

S2				S3			
$f_{m,Mean}$	σ	V_f	f_{mk}	$f_{m,Mean}$	σ	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.

214 7. Considerations about the Confidence Factors

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

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

217 where f_{mTest} and f_{kTest} are respectively the mean value and the character-
 218 istic 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

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

231 8. Proposal

232 Since the design value f_d should be the result of the procedure, the
233 methodology proposed consists in its direct determination leaving the con-
234 cept of KL . A new method, easily applicable in the context of an *in situ*
235 inspection, which includes two possibilities, was developed for the determi-
236 nation 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; η_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 γ_m , calibrated for new building, results overestimated for existing
 277 building because the execution uncertainties have been already overcome.

278 **9. Conclusion**

279 The tests carried out were the support for the outlining of a new method-
280 ology for the assessment of existing timber structures. In this context some
281 considerations were developed about the role of the CF s defined according to
282 [26]. CF depends on the KL gained by the operator in terms of quantity and
283 quality of information collected about the existing structure and it is applied
284 to the mean strength as an additional safety coefficient. Making a compar-
285 ison between the determination of the design value for new structures and
286 for existing structures and by using the results provided by the experimental
287 tests, the inadequacy of the CF values was demonstrated. Furthermore the
288 Standards does not differentiate CF values with respect to the type of struc-
289 ture, the size of timber members, the essence of timber or its class. Therefore
290 the idea is to follow a different approach, based on the direct determination
291 of the design value. The methodology proposed comprises two possibilities.

- 292 • *Testing*: It consists in carrying out tests on timber members that con-
293 stitute the existing structure. The design value is calculated following
294 [24]. This procedure was adapted to the specific case of timber samples
295 (see Section 8.1), by using the correction coefficients defined both in
296 the European Standards [23] and Italian Standards [26].
- 297 • *Visual Strength Grading*: The visual grading has to be carried out ac-
298 cording to prescriptions reported in [28] and [29] but with an approach
299 that should be similar to that defined by [27]. The idea is to do the
300 visual grading giving more importance to the defect individuated into
301 the most stressed zones. In fact, as it emerged from the tests, the per-
302 formance in terms of strength strictly depends on the localization of
303 the most important defects and not simply on their presence through
304 the element (see Section 4.1). Another aspect to take into account is
305 the fact the f_{kClass} were defined for new timber, in fact from the tests it
306 came out that there is a difference between the strength values actually
307 reached by the old timber samples and those provided by Standards.
308 For this reason f_{kClass} should be reduced with an additional coefficient
309 called k_c . Currently the values of k_c were calculated with the results
310 obtained by testing in order to give a way to process that should be
311 developed in the future with several tests. The objective is to obtain
312 k_c values for each type of strength (bending, compression orthogonal
313 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.

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