Safety assessment of historic timber structural elements

G. Barozzi[a], N. Cosentino[a], L. Lanzoni[a,b,], A.M. Tarantino[a]

[a]*DIEF-Department of Engineering "Enzo Ferrari", University of Modena and Reggio Emilia, 41125 Modena, Italy*
[b]*DESD - Dipartimento di Economia, Scienze e Diritto, University of San Marino, Salita alla Rocca 44, Republic of San Marino, 47890 San Marino*

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

Email address: `luca.lanzoni@unimore.it` (L. Lanzoni)

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 CFs 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 γ_M

$$f_d = \frac{f_k}{\gamma_M}. \tag{1}$$

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

$$f_d = \frac{f_m}{\gamma_M CF}. \tag{2}$$

The safety coefficient γ_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

$$f_k = \frac{f_m}{CF}. \tag{3}$$

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^{th} 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[1].

[1]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.

3.2. Grading

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

3.3. Equipment and method

Bending and compression tests were carried out according to the European Standards [22] (see Figs. 3, 4).

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

4. Results

The present Section deals with the main results provided by the experimental tests on wood samples, with special reference to ultimate bending and compression strengths.

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

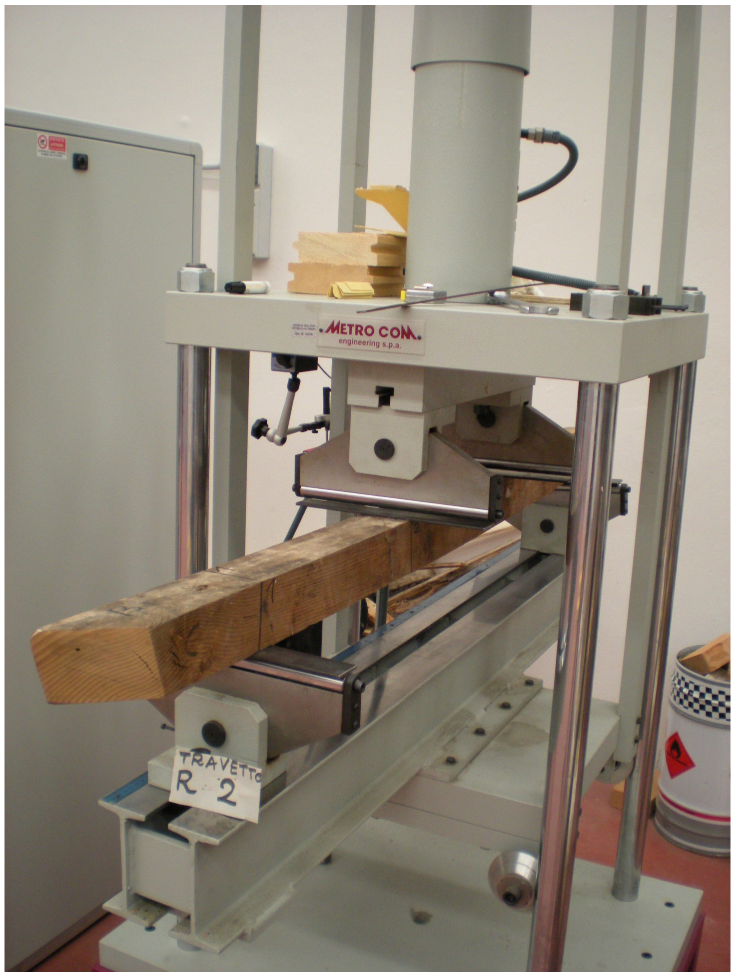

Figure 3: Bending test set up.

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

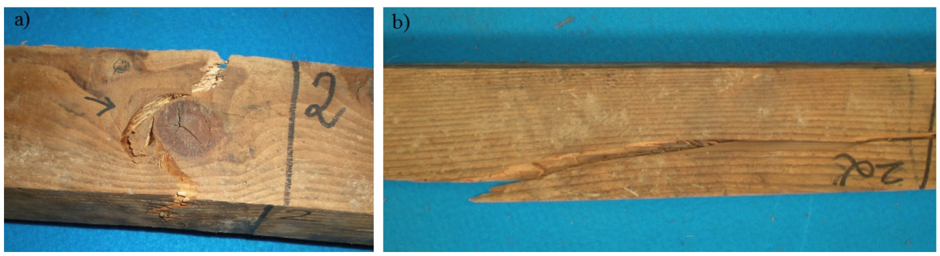

Figure 5: Samples after the bending test. a) Failure around a knot. b) Failure caused by fibre inclination.

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

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

[2]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 Class	Test values Q_{max}(kN)	v(mm)	f_m (MPa)	UNI 11035 $f_{m,k}$ (MPa)	Failure description
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* UNI 11035	Test values $Q_{c,0,max}$(kN)	$f_{c,0}$ (MPa)	UNI 11035 $f_{c,0,k}$ (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 $Q_{c,90t,max}$(kN)	$f_{c,90t}$ (MPa)	UNI 11035 $f_{c,90,k}$ (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.

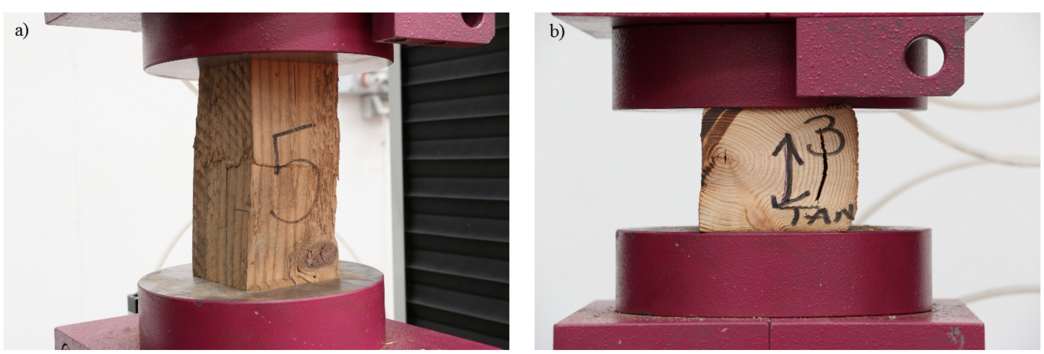

Figure 6: A sample under a) longitudinal compression and b) tangential compression.

		Test values		UNI 11035
Sample n.	Class* 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.

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

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

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

4.1. Observations on the results

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

Figure 7: A sample under radial compression.

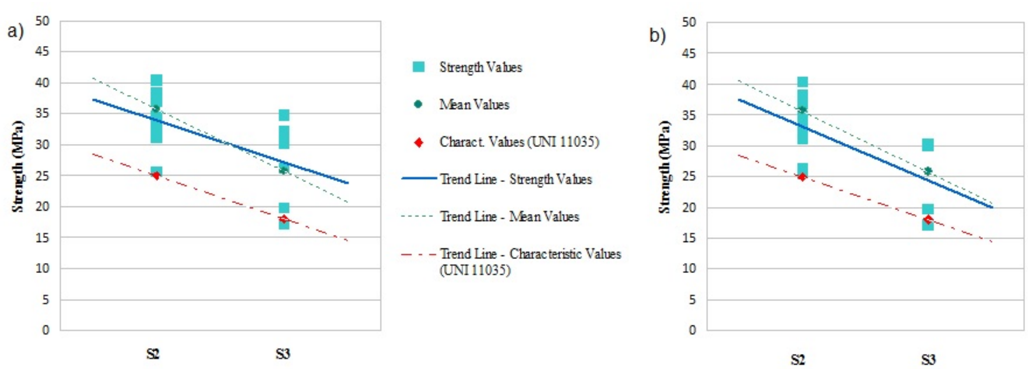

Figure 8: Ultimate bending strengths provided by the experimental tests. a) Subdivision between S2 and S3 considering full-size joists. b) Subdivision between S2 and S3 considering the class of the point from which failure started.

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" (blu solid line) is quite far from the theoretical "trend line" (green dotted line). On the other hand, in Fig. 8b the theoretical and the expiremental "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:

$$X_k = m_X \left(1 - k_n V_X\right), \tag{4}$$

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	∞
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 si less than 150 mm the characteristic values of bending strength should be adjusted to the reference condition by dividing by the coefficient k_h

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

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

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

with

$l_{et} = l + 5_{af}$

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

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

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

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”.

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

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

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

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

	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

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

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

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)

$$CF' = \frac{f_{mTest}}{f_{kTest}}, \tag{10}$$

wheref_{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.

8.1. Design value obtained by testing

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

$$f_{dTest} = \frac{f_{kTest}}{\gamma_M}\,\eta_d, \tag{11}$$

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

$$f_{dTest} = \frac{f_{kTest}}{\gamma_M}\,k_{Mod}. \tag{12}$$

8.2. Design value obtained by visual grading

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

$$f_{dClass} = \frac{f_{kClass}}{\gamma_M\,k_c}\,k_{Mod}, \tag{13}$$

where k_c should be applied to f_{kClass}. In fact, f_{kClass} is defined for new timbers, so it should be better to correct it with the coefficient k_c, which takes into account the divergence between class values of new timber and real strength of old timber. In the present work k_c was calculated using the f_{kTest} obtained by the tests executed in order to outline a way to process. In a future step the determination of k_c with a sufficiently high number of tests on homogeneous samples will be performed, so that each homogeneous material class could have its own k_c

$$k_c = \frac{f_{kClass}}{\gamma_M \, f_{dInf}} \, k_{Mod}. \tag{14}$$

Then, by substituting (12) in (14) and considering that in laboratory conditions $k_{Mod} = 1$, relation (14) becomes

$$k_c = \frac{f_{kClass}}{f_{kInf}} = \frac{f_{kClass}}{f_{kTest}}. \tag{15}$$

		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.

8.3. A mixed procedure to determine the design values

Based on the experience of the Authors, and to promote a deeper comprehension of the material properties, we finally suggest to calculate both f_{kClass} and f_{kTest} and choose the maximum between them, providing that

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

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

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 CFs 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.

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

Acknowledgements

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

References

[1] Alessandri S, Monti G, Goretti A, Sbaraglia L, Sforza G. *Metodi non distruttivi: Livelli di Conoscenza e Fattori di Confidenza*, Convegno Sperimentazione su Materiali e Strutture 2006; Venice, Italy.

[2] Brol J, Dawczyski S, Malczyk A, Adamczyk K. *Testing timber beams after 130 years of utilization*, Journal of Heritage Conservation 2012;32.

[3] Calderoni et al. *Experimental correlations between destructive and non-destructive tests on ancient timber elements*. Engineering Structures 2010; 32:2 442-448.

[4] Dezi L, Menditto G, Tarantino A M. *Homogeneous structures subjected to successive structural system changes*. ASCE Journal of Engineering Mechanics 1990; 116:8 1723-1732.

[5] Dezi L, Tarantino A M. *Time dependent analysis of concrete structures with variable structural system*. ACI Materials Journal 1991; 88:3 320-324.

[6] Dezi L, Menditto G, Tarantino A M. *Viscoelastic heterogeneous structures with variable structural system*. ASCE Journal of Engineering Mechanics 1993; 119:2 238-250.

[7] Faggiano B, Grippa M R, Marzo A, Mazzolani F M. *Combined Non-destructive and Destructive tests for the mechanical characterization of old structural timber elements*; 3rd International Conference on Advances in Experimental Structural Engineering 2009; 657-666.

[8] Franchin P, Pinto PE, Rajev P. *Confidence factor?*, Journal of Earthquake Engineering 2010;14: 989-1007.

[9] Gulvanessian A, Calgaro J A, Holick. *Desiner's guide to EN 1990 eurocode: basis of structural design*. Telford, London, 2002.

[10] Lanzoni L, Nobili A, Tarantino AM, *Performance evaluation of a polypropylene-based draw-wired fibre for concrete structures*, Construct. Build. Mater. 28 (2012) 798806.

[11] Lanzoni L, Soragni M, Tarantino A M, Viviani M. *Concrete beams stiffened by polymer-based mortar layers: Experimental investigation and modeling*, Construction and Building Materials 105 (2016) 321335.

[12] Lanzoni L, Tarantino A M. *Damaged hyperelastic membranes*, Int. J. NonLinear Mech. 60 (2014) 922.

[13] Lanzoni L, Tarantino A M. *Equilibrium configurations and stability of a damaged body under uniaxial tractions*, ZAMP Zeitsc. Angew. Math. Phys. 66(1) (2015) 171–190.

[14] Lanzoni L, Tarantino A M. *A simple nonlinear model to simulate the localized necking and neck propagation*, Int. J. NonLinear Mech. 84 (2016) 94-104.

[15] Loureno P B, Feio A O, Saporiti Machado S. *Chestnut wood in compression perpendicular to the grain: Non-destructive correlations for test results in new and old wood*. Construction and Building Material 2007; 1617-1627.

[16] Nobili A, Lanzoni L, Tarantino A M, *Experimental investigation and monitoring of a polypropylene-based fiber reinforced concrete road pavement*, Construct. Build. Mater. 47 (2013) 888-895.

[17] Piazza M, Riggio M. *Visual strength-grading and NTD of timber in traditional structures*, Journal of Building Appraisal 2008;3(4):267-296.

[18] Romo X, Gonalves R, Costa A et al. *Evaluation of the EC8-3 confidence factors for the characterization of concrete strength in existing structures*; Earthquake Engineering and Structural Dynamics 2010; 39 473-499.

[19] Saporiti Machado J, Feio A, Malczyk A. *In-situ assessment of timber structural members: Combining information from visual strength grading and NTD/SDT methods - A review*. Construction and Building Materials 2015.

[20] Saporiti Machado J, Loureno P B, Palma P. *Assessment of the structural properties of timber members in situ - a probabilistic approach*. International Conference on Structural Health Assessment of TimberStructures 2011; Lisbon, Portugal.

[21] Sousa H S, Branco J M, Loureno P B. *Use of bending tests and visual inspection for multi-scale experimental evaluation of chestut timber beams stiffness*. Journal of Civil Engineering and Management 2016; 22:6 728-738.

[22] CEN 408 (2004). *Timber Structures. Structural Timber and Glued Laminated Timber. Determination of Some Physical and Mechanical Properties*, European Committee for Standardization, Brussels.

[23] CEN 384 (2005). *Structural Timber - Determination of Characteristic Values of Mechanical Properties and Density*, European Committee for Standardization, Brussels. bibitemCEN 338 CEN 338 (2016). *Structural Timber. Strength Classes*, European Commettee for Standardization, Brussels.

[24] EN 1990. *Eurocode 0 - Basis of Structural Design - Annex D: Design Assisted by Testing*, European Commettee for Standardization, Brussels.

[25] EN 1998-3. *Eurocode 8 - Design of Structures for Earthquake Resistance - Part 3: Assessment and Retrofitting of Buildings*, European Commettee for Standardization, Brussels.

[26] NTC 2008. *Norme tecniche per le costruzioni*, Consiglio Superiore dei Lavori Pubblici. Gazzetta Ufficiale della Repubblica Italiana.

[27] UNI 11119 (2004). *Cultural Heritage - Wooden Artifacts - Load Bearing Structures - On Site Ispections for the Diagnosis of Timber Members*, Ente Nazionale di Unificazione; Milan, Italy.

[28] UNI 11035-1 (2010). *Structural Timber - Visual Strength Grading: Terminology and Measurements of Features*. Ente Nazionale di Unificazione; Milan, Italy.

[29] UNI 11035-2 (2010). *Visual Strength Grading and Characteristic Values for Italian Structural Timber Population*. Ente Nazionale di Unificazione; Milan, Italy.

List of Figures