Influence of an In-Situ Inspection on the Reliability Analysis of an Ancient Timber Roof

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Abstract Despite the durability of timber and its efficient performance seen in the built heritage, it has become a common practice, in Portugal, to replace ancient timber roof structures by concrete or steel roof ones. The main reason may be attributed to the difficulty in assessing the real condition of timber structures with respect to its actual level of conservation. In this work a reliability analysis of an ancient timber roof, from a Portuguese neoclassic building of the eighteenth century, is made to evidence the importance of different levels of information taking into account visual and geometric inspections. The impact of posterior knowledge obtained from non-destructive tests is evaluated by comparing the probability of failure and the reliability index on two distinct scenarios. The first scenario considers only prior information for the mechanical properties of timber elements and apparent cross-sections for the structural members. On the other hand, the second scenario considers the results of an in-situ inspection that provides the residual cross-section of the principal members, as well as the updating of the modulus of elasticity and density, based on a Bayesian Updating procedure that takes advantage of the results of a database of non-destructive tests. Latin Hypercube Sampling (LHS) was used in this study to generate a set of structural models, in which each model corresponds to a realization of the assumed random variables. Apart from the mechanical properties, the uncertainties related to permanent, snow and wind loads, are included according to the provisions of the Joint Committee on Structural Safety (JCSS). The presented results indicate that in-situ inspections have to be a priority on the assessment of ancient timber structures. The absence of a careful assessment of deteriorated elements can lead to incorrect conclusions about the structural safety. Additionally, the use of a probabilistic framework allows to a better definition of intervention plans by providing the reliability of distinct critical elements.

Keywords Bayesian updating · Non-destructive tests · Reliability

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1 Introduction

The failure of several timber roof structures under expected snow loads has shown that design and/or construction errors, unexpected deterioration, and insufficient maintenance can lead to consequences far greater than the initial event [1]. On the other hand, when each structural member is properly designed and an adequate maintenance plan is put into practice, timber structures are durable and can remain in service for periods greater than the design life-span [2]. Despite the durability of ancient timber, it has become a usual practice, in Portugal, to replace existing timber roofs structures by concrete or steel roof structures, supposedly due to lower construction costs. This is mainly due to the difficulty in assessing the real condition of timber structures, with respect to its actual level of conservation. In fact, the geometric assessment and visual inspection of existing timber structures are two of the most important tasks in evaluating their integrity. The inherent variability of timber and its susceptibility to decay emphasize the need to carry out a characterization of the constitutive timber elements [3]. The information on the geometry and the state of degradation can be measured by means of non-destructive tests (NDT), mechanical tests, and other means [4]. However, the uncertainties related to the actual mechanical properties constitute a drawback for practitioners, which in turn may design ultra-conservative solutions or decide for a complete replacement of the structure. The existence of a database with correlations between NDT results and mechanical properties of wood from different species would constitute an important achievement. On the other hand, engineers can perform reliability-based analysis, where they can assume that mechanical properties, as well as the external loads, as random variables, thus accounting to different sources of uncertainty. In this regard, Köhler et al. [5] proposed distribution functions for the main mechanical properties of timber members, based on test programs and investigations considering European and North American softwoods, whereas the probabilistic models for loads can be derived based on the Probabilistic Model Code and other standards. Additionally, the parameters of these probabilistic models can be updated through the use of Bayesian methods, when new empirical or monitoring data is available.

The main objective of this paper is to implement a probabilistic methodology to evaluate the safety of an ancient timber roof, from a Portuguese neo-classic building of the eighteenth century, for partial collapse prevention limit state. The evaluation includes the inherent uncertainties of timber, as well as the uncertainties related to external loading. Distinct damage scenarios are considered, in order to stress the importance of performing adequate in-situ inspections before the design of solutions to retrofit the structure. The first assumed scenario (Scenario 1) considers the cross-sections with their apparent dimensions, and the mechanical properties as random variables represented through the probabilistic distributions proposed in [5] and considering the mean values presented in national standards [6]. The second reliability analysis takes advantage of various NDT, such as impact penetration, drilling resistance and ultrasounds tests, where effective cross-sections are considered for the different structural members, whereas a Bayesian Updating procedure

was performed to obtain new model parameters for density and modulus of elasticity (Scenario 2).

2   Description of the Structure

The two pitched roof structure is composed by four Maritime Pine (*Pinus pinaster*) collar beam trusses, spaced 3 m from each other, as illustrated in Fig. 1. The disposition of elements was based on the structural configuration of the Chimico Laboratory, a Portuguese neoclassic building from the eighteenth century.

In the case of the collar beam trusses considered in the current study, a geometrical assessment was performed at intervals of 40 cm for each individual member. The assessed sections were marked and identified so that the dimensions of these same sections could be measured using non-destructive tests in order to obtain the effective cross section. Information on the density, the presence of voids and their location, as well as the modulus of elasticity were obtained using impact penetration, drilling resistance and ultrasounds tests. Figure 2 illustrates a mapped diagram with measured sections of each member of one collar truss. Due to the biological attack found on the surface of the elements, significant coefficients of variation were found pertaining to the cross-section dimensions. Moreover, extensive wane was found affecting the rectangular section of the elements. This was especially found on the rafters and collar beams. as presented in Table 1.

One of the collar trusses was tested at the Structural Laboratory of the University of Minho, Portugal. The truss was subjected to two downward loads, applied on the rafters, at a displacement rate of 0.05 mm/s until failure. Further information is available in [9]. Three main failures were found during the mechanical test, namely, failure of both rafters in the sections near the connection with the tie-beam and failure near the connection of one of the rafters with the post. Those failures coincided with sections having lower visual grades due to the presence of significant knots.
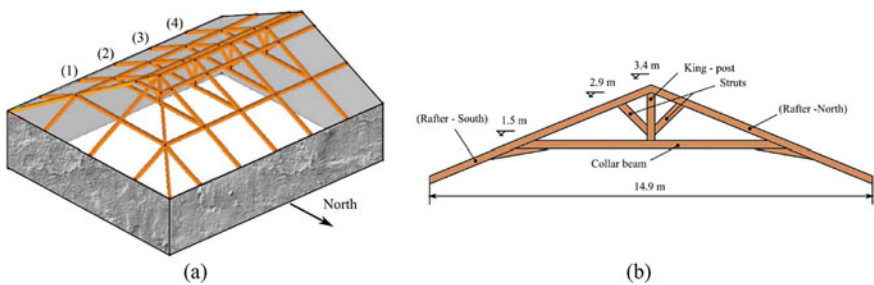
Fig. 1 Timber roof: **a** three-dimensional perspective and **b** planar collar beam truss [7]
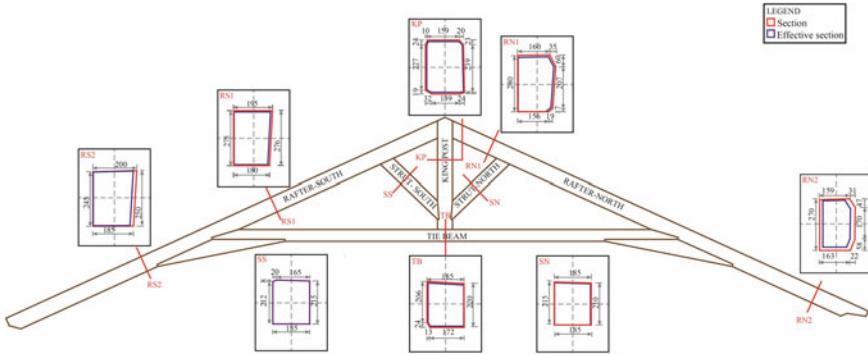
Fig. 2 The collar truss with representative sections obtained from the geometrical survey mapped [8]

3 Probabilistic Model

3.1 Structural Model, Limit States and Variables

For the structural analysis, 2D linear finite element models were developed within the OpenSees framework. The trusses were modelled using linear elastic frame elements connected with zero-length elements. Each structural element is divided in frame segments of in 40 cm length as per the locations of the non-destructive tests carried out. The stiffness between the collar beam and the rafters is considered as rigid for all the three degrees of freedom (translational and rotational), whereas the remaining ones were considered as hinged connections. The supports of the truss on the walls were also modelled by hinges. The connection between the purlins and the trusses is considered as weak. In this scenario, in case of failure of one of the trusses, the loads are not redistributed to another undamaged parts of the structure. Additionally, one can evaluate separately the reliability of each truss, while considering uniform distribution of loads within the roof. The considered failure modes are related with the ultimate limit state verifications for bending and combined axial and shear forces in the timber elements according to Eurocode 5 [10]. The failure of elements due to bending and combined axial forces, is given as follows:

$$g_i = X_R - \left(\frac{a_i \cdot G + b_i \cdot Q}{k_{mod} \cdot k_{c,z} \cdot R_{c,0} \cdot A} + k_m \cdot \frac{c_i \cdot G + d_i \cdot Q}{k_{mod} \cdot k_h \cdot R_m \cdot W} \right) = 0 \qquad (1)$$

where A is the cross-section area, W is the section modulus, and X_R is the model uncertainty, which is modelled through a lognormal distribution with an expected value of 1.0 and a coefficient of variation of 10% [11]. The internal bending moment, as well as the normal force, are given by linear combinations the permanent loads G, and the variable loads Q, which englobes imposed, wind, and snow loads. R_m is the bending strength parallel to the grain and $R_{c,0}$ is the compression strength parallel to

Table 1 Apparent and residual (in parenthesis) dimension of the structural elements cross-sections

Truss	Element	Height (mm)			Width (mm)		
		Min	Mean	Max	Min	Mean	Max
(1)	North rafter	255.0 (170.0)	271.3 (256.1)	280.0 (280.0)	180.0 (60.0)	189.4 (90.6)	200.0 (165.0)
	South rafter	260.0 (138.0)	280.0 (209.8)	290.0 (285.0)	165.0 (50.0)	183.1 (101.9)	190.0 (190.0)
	Collar beam	180.0 (90.0)	217.2 (170.2)	240.0 (214.0)	190.0 (130.0)	195.0 (176.7)	200.0 (195.0)
	King-Post	265.0 (227.0)	265.0 (252.3)	265.0 (265.0)	200.0 (200.0)	260.0 (260.0)	380.0 (380.0)
	North strut		215.0 (205.0)			190.0 (185.0)	
	South strut		210.0 (210.0)			185.0 (185.0)	
(2)	North rafter	215.0 (90.0)	240.0 (179.9)	250.0 (235.0)	180.0 (180.0)	186.7 (185.7)	190.0 (190.0)
	South rafter	225.0 (123.0)	243.3 (150.3)	260.0 (190.0)	195.0 (100.0)	202.9 (155.0)	220.0 (220.0)
	Collar beam	200.0 (70.0)	215.0 (184.5)	225.0 (225.0)	190.0 (115.0)	199.6 (186.4)	205.0 (205.0)
	King-Post		260.0 (230.0)			425.0 (425.0)	
	North strut		195.0 (195.0)			190.0 (100.0)	
	South strut		225.0 (125.0)			200.0 (200.0)	
(3)	North rafter	245.0 (165.0)	262.8 (230.9)	270.0 (270.0)	190.0 (110.0)	195.0 (163.9)	200.0 (200.0)
	South rafter	240.0 (200.0)	270.0 (242.5)	290.0 (262.0)	180.0 (140.0)	190.8 (176.7)	200.0 (190.0)
	Collar beam	215.0 (163.0)	231.7 (216.0)	240.0 (237.0)	190.0 (185.0)	195.8 (194.2)	200.0 (200.0)
	King-Post	260.0 (230.0)	260.0 (245.0)	260.0 (260.0)	195.0 (195.0)	250.0 (201.3)	415.0 (220.0)
	North strut	235.0 (199.0)	237.5 (211.5)	240.0 (224.0)	195.0 (195.0)	195.0 (195.0)	195.0 (195.0)
	South strut		230.0 (230.0)			200.0 (160.0)	
(4)	North rafter	260.0 (175.0)	267.5 (230.5)	280.0 (268.0)	190.0 (125.0)	200.4 (195.0)	205.0 (205.0)
	South rafter	250.0 (130.0)	265.0 (206.9)	270.0 (236.0)	190.0 (95.0)	195.0 (159.5)	205.0 (195.0)

Table 1 (continued)

Truss	Element	Height (mm)			Width (mm)		
		Min	Mean	Max	Min	Mean	Max
	Collar beam	180.0 (175.0)	207.9 (203.7)	220.0 (220.0)	190.0 (100.0)	196.4 (167.9)	200.0 (200.0)
	King-Post	265.0 (205.0)	267.5 (205.0)	270.0 (205.0)	195.0 (190.0)	312.5 (190.0)	430.0 (190.0)
	North strut		220.0 (170.0)			190.0 (190.0)	
	South strut		230.0 (150.0)			190.0 (190.0)	

the grain. k_{mod} (equal to 0.9) is a modification factor taking into account the effect of the duration of load and moisture content. $k_{c,z}$ is an instability factor, which is dependent of the slenderness of each structural element, and k_h is a size effect factor [10]. On the other hand, the shear failure of elements is modelled with the following short-term ultimate limit state function:

$$g_i = X_R - \left(\frac{e_i \cdot G + f_i \cdot Q}{k_{mod} \cdot R_v \cdot A} \right) = 0 \tag{2}$$

where R_v is the shear strength, and the internal shear force is given by linear combinations of the variable loads Q and the permanent loads G. As presented in Table 2, seven random variables are evaluated for each timber element. The distribution parameters of the mechanical properties (bending strength, bending modulus of elasticity, and density), used for the first scenario assumed, are computed based on characteristic values and mean values, defined in NP 4305:1995 [6] for structural Maritime

Table 2 Random variables for timber material properties

X	Distribution parameters				Correlation matrix						
	Dist.	E[X]	CoV[X]		R_m	E_m	ρ_{den}	$R_{t,0}$	$R_{c,0}$	G_v	R_v
R_m	LN	27.8	0.25	R_m	1.0	0.8	0.6	0.8	0.8	0.4	0.4
E_m	LN	7500	0.13	E_m		1.0	0.6	0.6	0.6	0.6	0.4
ρ_{den}	N	580	0.13	ρ_{den}			1.0	0.4	0.8	0.6	0.6
$R_{t,0}$	LN	18.6	0.30	$R_{t,0}$				1.0	0.5	0.4	0.6
$R_{c,0}$	LN	25.4	0.20	$R_{c,0}$					1.0	0.4	0.4
G_v	LN	470.0	0.13	G_v						1.0	0.6
R_v	LN	3.1	0.25	R_v							1.0

Description: R_m—Bending strength to the grain (N/mm^2); E_m—Bending modulus of elasticity (N/mm^2); ρ_{den}—Density (kg/m^3); $R_{t,0}$—Tension strength to the grain (N/mm^2); $R_{c,0}$—Compression strength to the grain (N/mm^2); G_v—Shear modulus (N/mm^2); R_v—Shear strength (N/mm^2)

Table 3 Random variables for timber material properties

Variable	Distribution	$E[X]$ (kN/m^2)	$CoV[X]$
Permanent load, P	Normal	1.0	0.25
Live load, LL	Gumbel	0.32	0.40
Wind load—upwind, W_{up}	Gumbel	0.23	0.35
Wind load—downwind, W_{down}	Gumbel	0.32	0.35
Snow load, S	Gumbel	0.20	0.40

Pine swan timber. Table 2 also presents the intra-element correlation coefficients considered, which were also taken from the work developed by Köhler [6].

The loads considered are the permanent (i.e., weight of the trusses, roof tiles and sheeting), imposed (i.e. live load), wind loads (i.e., upwind and downwind), and snow load. The expected values (mean values), coefficient of variation (CoV), and probabilistic distributions used for each load type are provided in Table 3.

3.2 Bayesian Updating of Mechanical Properties

The updating procedure was detailed for the case of existing timber structures in [12], following a brief description is provided. When samples or measurements of a stochastic variable X are provided, the probabilistic model may be updated and, thus, also the probability of failure. Considering a stochastic variable X with density function $f_X(x)$, and if q denotes a vector of parameters defining the distribution for X, the density function of the stochastic variable X can be written as $f_X(x,q)$. When the parameters q are uncertain then $f_X(x,q)$ can be considered as a conditional density function: $f_X(x|Q=q)$ where q denotes a realization of Q, therefore q is a vector of distribution parameters (e.g. mean μ, and standard deviation σ). The initial density function for the parameters Q is denoted $f_Q'(q)$ and is termed the *prior* density function.

Taking into account new information, it is assumed that n observations or measurements of the stochastic variable X are available making up a sample $\hat{x} = (\hat{x}_1, \hat{x}_2, \ldots, \hat{x}_N)$. The measurements are assumed to be statistically independent. The updated density function $f_Q''(q|\hat{x})$ of the uncertain parameters Q given the realizations is denoted the *posterior* density function and is given by:

$$f_Q''(q|\hat{x}) = \frac{f_N(\hat{x}|q)\, f_Q'(q)}{\int f_N(\hat{x}|q)\, f_Q'(q)\mathrm{d}q} \tag{3}$$

where the *Likelihood function* $f_N(\hat{x}|q) = \prod_{i=1}^{N} f_X(\hat{x}_i|q)$ is the probability density of the given observations assuming that the distribution parameters are q. Then the updated density function of the stochastic variable X given the realization $\hat{x}$ is denoted the *predictive* density function and is defined by:

$$f_X(x|\hat{x}) = \int f_X(x|q) f_Q''(q|\hat{x}) dq \tag{4}$$

The *prior* and *posterior* distributions are often chosen according to the available data and to the importance of the analysis. Normal distributions are often used for that purpose. Assuming that X is Normal distributed and both mean value, μ and standard deviation, σ are uncertain then the *prior* distribution of the resistance function R is denoted $f_Q'(q) = f_R'(\mu, \sigma)$ and can be defined as Eq. (5):

$$f_R'(\mu, \sigma) = k\sigma^{-(v'+\delta(n')+1)} \exp\left(-\frac{1}{2\sigma^2}\left(v'(s')^2 + n'(\mu - m')^2\right)\right) \tag{5}$$

with $\delta(n') = 0$ for $n' = 0$ and $\delta(n') = 1$ for $n' > 0$. The *prior* information about the standard deviation σ is given by parameters s' and v'. The expected value $E(\sigma)$ and coefficient of variation $COV(\sigma)$ of σ can asymptotically (for large v') be expressed as:

$$E(\sigma) = s' \tag{6}$$

$$COV(\sigma) = \frac{1}{\sqrt{2v'}} \tag{7}$$

The *prior* information about the mean μ is given by parameters m', n' and s'. The expected value $E(\mu)$ and coefficient of variation $COV(\mu)$ of μ can asymptotically (for large v') be expressed as:

$$E(\mu) = m' \tag{8}$$

$$COV(\mu) = \frac{s'}{m'\sqrt{n'}} \tag{9}$$

Another possible way to interpret the *prior* information is to consider the results of a hypothetical prior test series, for mean and standard deviation analysis. For that case the standard deviation is characterized by s' (hypothetical sample value) and v' (hypothetical number of degrees of freedom for s'). The information about the mean is given by: m' (hypothetical sample average) and n' (hypothetical number of observations for m').

Usually the degrees of freedom for the number of observations n is given by $v = n - 1$, but the *prior* parameters n' and v' are independent from each other. When new information is available, the resistant model given by the *prior* distribution $f_R'(\mu,$

σ) may be updated according to Eq. (3), with the parameters:

$$n'' = n' + n \tag{10}$$

$$v'' = v' + v + \delta(n') \tag{11}$$

$$m''n'' = n'm' + nm \tag{12}$$

$$v''(s'')^2 + n''(m'')^2 = v'(s')^2 + n'(m')^2 + vs^2 + nm^2 \tag{13}$$

With this procedure the predictive value of the resistance R is given by:

$$f_R = m'' - t_{v''}s''\sqrt{\left(1 + \frac{1}{n''}\right)} \tag{14}$$

where $t_{v''}$ has a central t-distribution [11]. The new information may be considered from the data gathered in non-destructive tests and with that data reliability may be updated.

3.3 Data for Updating

In this work, Bayesian updating methods are used to update two key mechanical properties of the material, namely bending modulus of elasticity and density. The updating data was obtained through NDT's results collected using pin penetration tests, conducted on one of the trusses, as well as from tests allowing to estimate the correlations between those results and the mechanical properties. Further detail regarding the experimental results and linear correlation analysis can be found in [9, 13, 14].

3.4 Updated Values for the Key Mechanical Properties (Scenario 2)

Information is considered by vague prior information on both mean value and standard deviation. Therefore, the prior information parameters can be presented as hypothetical sample average, m', and sample standard deviation, s', are not relevant; hypothetical number of observations for m', $n' = 0$; hypothetical number of degrees of freedom for s', $v' = 0$. Thus, the posterior parameters become: $n'' = n$; $v'' = n - 1$; $m'' = m$ and $(s'')^2 = s^2$. The predictive value of r is given by:

Table 4 Updated values for bending modulus of elasticity and density for the tested truss, assuming vague information on both mean and standard deviation

Element	MOE		Density	
	Mean (kN/mm^2)	CoV (%)	Mean (kg/m^3)	CoV (%)
North rafter	5596	7.8	575	1.0
South rafter	8218	1.9	601	0.5
Collar beam	8992	1.9	613	0.5
Post	8672	1.6	608	0.47
North diagonal	10,004	0.9	626	0.3
South diagonal	11,344	0.6	645	0.2
Truss (global)	7830	4.8	601	0.8

$$r_d = \exp(m(Y)) \cdot \exp\left(-t_{vd} \cdot s(Y) \cdot \sqrt{1 + \frac{1}{n}}\right) \tag{15}$$

where t_{vd} has a central t-distribution.

The results of the updated values for bending modulus of elasticity and density are provided for the tested truss, for each element and globally on Table 4. By analysis of the results it is seen that a low variation is found within elements with exception of the North rafter. This is consistent with the visual inspection carried out to the trusses which showed that this element was severely decayed on localized segments (near the wall support).

From the results obtained, one concluded that for Scenario 2, the modulus of elasticity and the density should be modelled considering the global updated parameters (expected value and coefficient of variation), presented in Table 4, whereas the remaining random variables are represented with the same parameters and distributions, already presented in Table 2.

4 Reliability Analysis

The Latin Hypercube Sampling (LHS) was used to generate 100,000 structural models for each scenario considered in this analysis. As mentioned above, the four trusses where analyzed separately through 2D linear finite element models. Thus, the reliability analysis implied the performance of 800,000 numerical analyses. A load controlled method was applied with a load factor increment $\Delta\lambda_L = 0.1$. For each structural realization, the analysis finished when the short-term ultimate limit state function (Eqs. 1–2) did not hold, which is associated to the partial collapse limit state. During the analysis, a structural failure is considered when the load factor is lower or equal to 1.0, or when the structure is not able to sustain the applied loads. The probability of failure is then given by the ratio between the number of failures

(z_0) and the number of realizations (z):

$$p_f = \frac{z_0}{z} \tag{16}$$

Given the probability of failure (p_f), it is possible to determine the structural reliability index (β) through the inverse cumulative distribution function (Φ^{-1}) of standard normal distribution:

$$\beta = \Phi^{-1}\left(1 - p_f\right) \tag{17}$$

Steenbergen et al. [15] proposed a target reliability β_0, which defines the minimum reliability for which not to upgrade the structure is assumed as acceptable. The index β_0 is dependent on the lifetime period, collapsed area, and consequence class. If one considers a lifetime of 5 years and a high consequence class, the structure under study needs an intervention when the reliability index is lower than 3.6. From the results presented in Table 5, one can conclude that in-situ inspections have to be a priority on the assessment of ancient timber structures. The use of apparent cross-sections and timber mechanical properties given by NP 4305:1995 [6], could lead to the decision of postponing an intervention. However, when the analysis considers the effective cross-section, as well as the updated mechanical properties, one can conclude that the structure needs urgent interventions. The impact of assessing the deterioration of timber elements can be also evaluated through the cumulative distribution functions presented in Fig. 3, which were obtain by fitting a lognormal distribution to the values of load factor measured at failure for each truss. The reduction of the effective

Table 5 Reliability indices and probabilities of failure for distinct Scenarios

Scenario	p_f	β
(1) Without in-situ inspection	9×10^{-5}	3.75
(2) After an in-situ inspection	0.96	−1.72
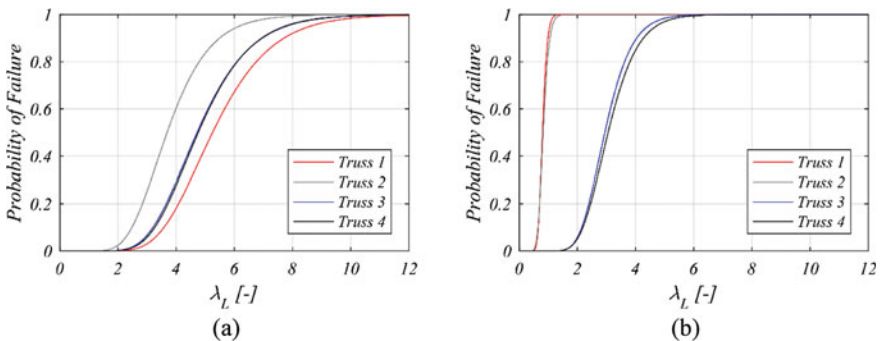
Fig. 3 Cumulative distribution functions for load factors obtained at the partial collapse limit state: **a** Scenario 1 and **b** Scenario 2

cross-section of the rafters of trusses 1 and 2 (see Fig. 1), are compromising their strength, especially for bending and compression forces. On the other hand, one can conclude that trusses 3 and 4 are still able to sustain the applied loads. Thus, a reliability analysis can be a useful tool to plan future interventions in ancient timber structures.

5 Conclusions

A reliability analysis was presented for an ancient timber roof. The evaluation was based on the results of an in-situ inspection including non-destructive tests. The inspection allowed to gather the dimensions of apparent and effective cross-sections, while the results of impact penetration, drilling resistance and ultrasounds tests, permitted the Bayesian update of mechanical properties, such as the elastic modulus and wood's density. The presented results are indicative that in-situ inspections have to be a priority on the assessment of ancient timber structures in order to have an optimized intervention plan. The absence of a careful assessment of deteriorated elements can lead incorrect conclusion about the global structural safety. For the presented study case, the use of a probabilistic framework would allow to define the intervention plan by providing the reliability of distinct primary trusses. Future studies shall assess the impact of defect on the reliability of ancient structures, given that it is difficult to measure in-situ the size and dispersion of knots.

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