Reliability based robustness of timber structures through NDT data updating

Hélder S. Sousa[1], John D. Sørensen[2], Poul H. Kirkegaard[3], Paulo B. Lourenço[4], Jorge M. Branco[5]

Abstract This work presents a framework for reliability-based assessment of timber structures / members using data gathered from non-destructive test results. These results are used for modeling an update of the mechanical characteristics of timber, using Bayesian methods. These methods are suitable to be used for parameter estimation and also allow updating model uncertainties. From the updated model, decisions upon the life-cycle reliability of existing structures may be taken and maintenance or strengthening actions may be considered. In this work, results gathered from ultrasound testing, Resistograph® and Pilodyn® conducted on chestnut wood specimens were used, as well as correlations between those results and compression strength parallel to the grain tests' results. The resistant characteristics are also updated assuming deterioration models applied to specific key elements of the structure, thus, being possible to evaluate reliability based in time dependent factors, as well to categorize that structure in terms of robustness.

Keywords timber, NDT, reliability, Bayesian methods, existing structures

1. INTRODUCTION

Timber is a rather complex construction material, due to its anisotropic behavior. Moreover, its properties also vary on space and time. For instance, the material properties of a timber element vary both in different parts of the same cross-section as well as along the element itself. Differences are also visible when comparing different timber elements even if they are from the same species. Therefore, timber structures are better analyzed using probabilistic models rather than deterministically when considering a structural safety evaluation.

Reliability methods are prone to be used in this assessment as they allow describing the properties of timber elements by random variables. The probability density functions of timber's mechanical properties are defined in various codes and guidelines and may be updated with the results of mechanical tests. A grading methodology based on visual inspection associated with NDT results is also a suitable source of information for an updating data model. When dealing with existing structures, data updating may be regarded as an important tool in the assessment of its reliability

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parameters. For that purpose, Bayesian methods are often applied to implement new information into a probabilistic structural assessment due to their simplicity of use. Bayesian methods allow quantifying an approximation about the statistical uncertainty related to the estimated parameters, regarding both the physical uncertainty of the considered variables as well as the statistical uncertainty related to the model parameters. Therefore, they offer a suitable method for parameter estimation and model updating. However, for making this possible, it is necessary to take into account the measurement and the model uncertainties in the probabilistic model formulation.

The purpose of this work is to present a framework for reliability based assessment of timber structures using data gathered from non-destructive tests (NDT) results. For that purpose, results gathered from non-destructive techniques via ultrasound testing, Resistograph® and Pilodyn® conducted upon a batch of chestnut wood specimens are used, as well as correlations between those results and compression strength values taken from Feio (2005). The uncertainties connected to the NDT are modeled and included in the assessment.

The structural resistance characteristics are also updated assuming deterioration models applied to some specific key elements of the structure, thus, being possible to evaluate reliability based on time dependent factors, as well to categorize that structure in a robustness framework.

2. UPDATING METHODS

2.1. Bayesian updating methods

When assessing existing structures a variety of information may be gathered from several distinct sources, which may be available or can be made available at a given cost. Qualitative together with quantitative information may allow defining the general condition of an existing structure. In the assessment of existing structures, this information can be taken into account and combined with prior probabilistic models resulting in so-called posterior probabilistic models. Regarding design assisted by results taken from tests, Eurocode 0 in Annex D (EN 1990:2002), provides different procedures to statistically determine a single property, in terms of design and characteristic values, and also provides information to statistically determine resistance models with use of additional *prior* information. Throughout their lifetime, structures change due to many aspects, from natural causes (such as material deterioration, environmental exposure and long term effects of loads), to human decisions (such as modification of the structure or changing of use) or even by accidental actions. Thus, the assessment of existing structures should be regarded as a successive process of model updating and consequent evaluation regarding new information. The Bayesian probabilistic assessment for structures is illustrated schematically in Figure 1.

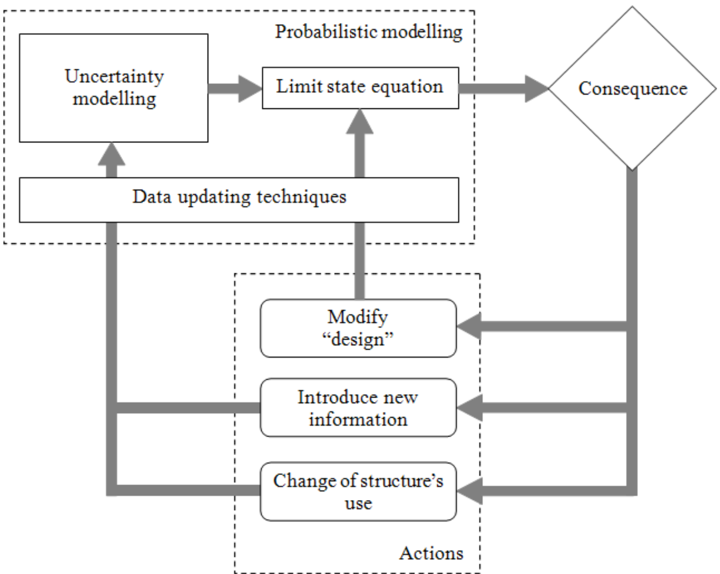


Figure 1 – Bayesian probabilistic assessment for structures, adapted from JCSS (2001).

2.2. Updating data

In this paper, the updating data is obtained through results collected from NDT's using ultrasound testing, Resistograph® and Pilodyn®, conducted upon a batch of chestnut wood specimens, as well as from tests allowing to estimate the correlations between those results and the compression strength parallel to grain, $f_{c,0}$. These data were gathered in Feio (2005) and is represented in Figure 2. The resistographic measure represents the ratio between the integral of the area of the diagram, *Area*, and the height, *h*, of the test specimens, which is therefore an average value. This value is given as:

$$RM = \frac{\int_0^h Area}{h} \tag{1}$$

For the case of the Pilodyn®, the considered measure parameter is the needle penetration depth taken directly from the tests. In the case of the ultrasound pulse velocity method the (elasto) dynamic modulus of elasticity, E_{din}, was calculated by:

$$E_{\text{din}} = V^2\,\rho \tag{2}$$

where E_{din} represents the (elasto) dynamic modulus of elasticity (N/mm^2), V is the propagation velocity of the longitudinal stress waves (m/s) and ρ is the density of the specimens (kg/m^3). It must be noted that the indirect method was used in the ultrasound tests.

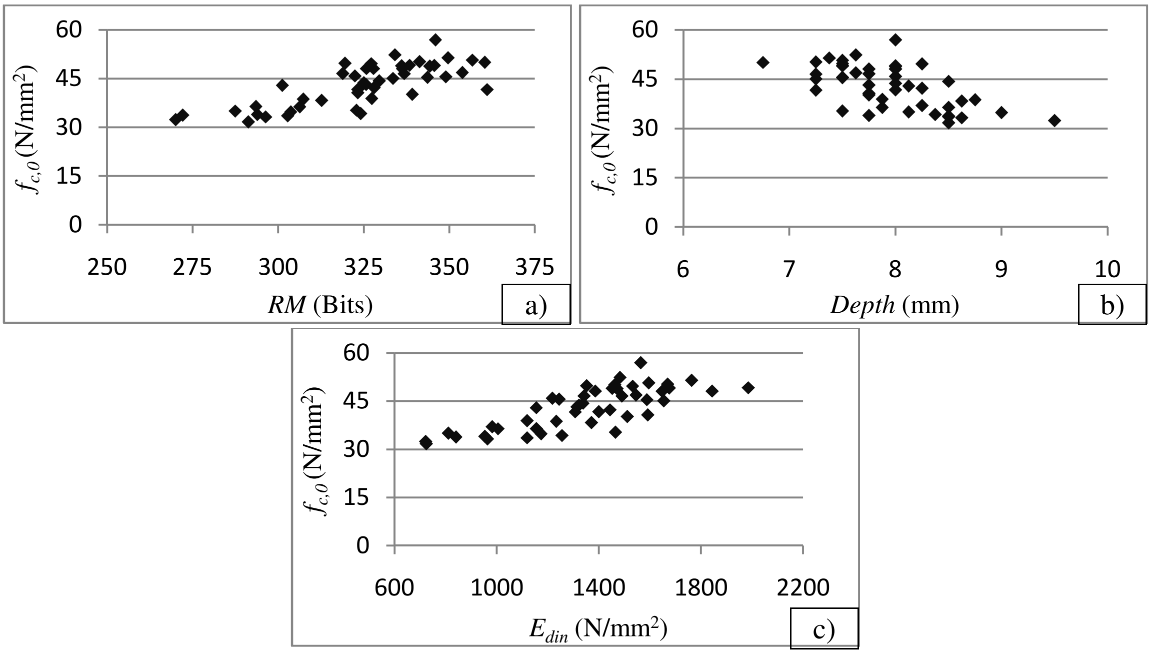


Figure 2 – Correlation between $f_{c,0}$ and NDT results, adapted from Feio (2005): a) Resistograph®; b) Pilodyn ®; c) ultrasound.

2.3. Model uncertainty

The uncertainties connected to the NDT methods are modeled and included in the assessment through a Maximum Likelihood method. For parameter estimation of linear regression lines, the following linear regression model in x_1, ..., x_m –space was considered:

$$y = \alpha_0 + \alpha_1 x_1 + \ldots + \alpha_m x_m + \varepsilon \tag{3}$$

where α_0, α_1, ..., α_m are the regression parameters and ε models the lack-of-fit. ε is assumed to be Normal distributed with expected value 0 and standard deviation σ_ε. The regression parameters for each correlation between $f_{c,0}$ and the NDT results are given in Table 1.

Table 1 – Regression parameters and lack-of-fit standard deviation for the non-destructive tests results in correlation with the $f_{c,0}$ given by destructive tests.

	a_0	a_1	σ_ε
Resistograph®	-30.69497	-0.22750	4.12217
Pilodyn®	102.59179	7.50664	5.12435
Ultrasound	19.24686	-0.01761	4.06081

3. DETERIORATION MODELS

To assess the evolution of the timber elements deterioration, a bi-parametrical idealized decay model proposed by Leicester (2001) was considered. The two parameters correspond to an initial propagation period of the deterioration phenomenon, t_{lag} (year) and an annual penetration ratio, r (mm/year). Considering a probabilistic analysis, the t_{lag} parameter is defined as a deterministic variable, while the r parameter should be defined by a lognormal distribution with a coefficient of variation between 0.5 and 1 according to the timber durability class (Wang et al. 2008).

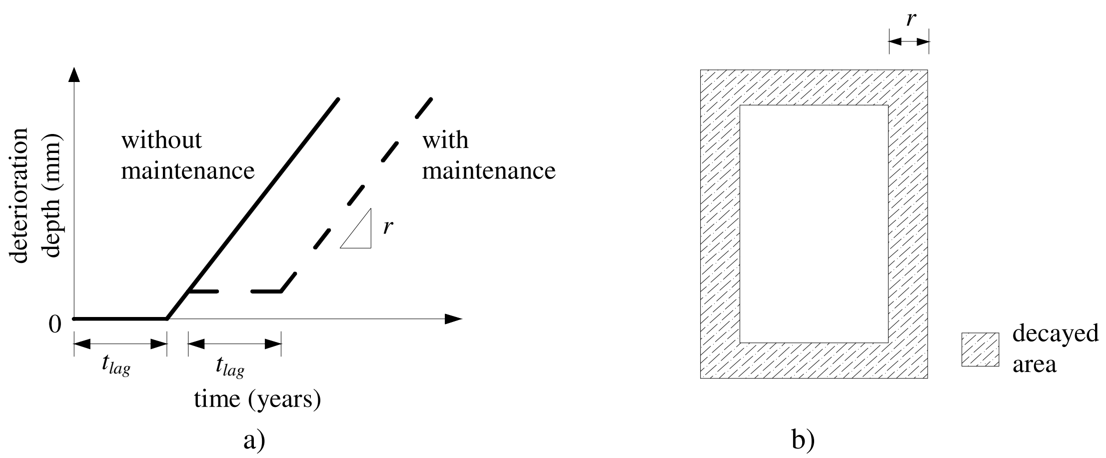


Figure 3 – Progress of decay: a) idealized model, adapted from Leicester (2001); b) damage penetration on a decayed cross-section.

4. RELIABILITY BASED ROBUSTNESS

In Frangopol and Curley (1987) and Fu and Frangopol (1990), some probabilistic measures related to structural redundancy are proposed. These measures can also be indicators of the level of robustness in some occasions. A redundancy index, RI, is defined by:

$$RI = \frac{P_{f(dmg)} - P_{f(sys)}}{P_{f(sys)}} \tag{4}$$

where $P_{f(dmg)}$ is the probability of failure for a damaged structural system and $P_{f(sys)}$ is the probability of failure of an intact structural system. The redundancy index provides a measure on the robustness / redundancy of the structural system. The following related redundancy factor, β_R, is also considered:

$$\beta_R = \frac{\beta_{intact}}{\beta_{intact} - \beta_{damaged}} \tag{5}$$

where β_{intact} is the reliability index of the intact structural system and $\beta_{damaged}$ is the reliability index of the damaged structural system.

Robustness may be considered as the ability of the system to suffer an amount of damage not disproportionate with respect to the causes of the damage itself (Biondini 2008). Therefore, in the scope of this work, deterioration must be defined in a period of time in order to assess if the effects in respect to this phenomenon are in proportion to the causes. Depending on the importance of a specific structure and regarding to structural and durability performance, different design lifetimes are considered. Since degradation is a long duration cause of damage, a time index, *TI*, was considered as:

$$TI = \frac{T_{lim} - T_d}{T_d} \tag{6}$$

where T_{lim} is the time given by the deterioration model for a defined limit of reliability, and T_d the time lifetime used for design. The values of *TI* may be equal:

- to -1, if the structure has a lower reliability than the required reliability level at time = 0. However this value must be disregarded for an existing structure with deterioration being the damage cause. This value can only be obtained if a wrong design procedure (or execution) was taken;
- to]-1;0[, if the reliability limit time is obtained before the design time;
- to 0, if the reliability limit time is obtained exactly at the design time;
- to > 0, if the reliability limit time occurs after the design time, which is normally the required condition for safety conditions.

5. EXAMPLES

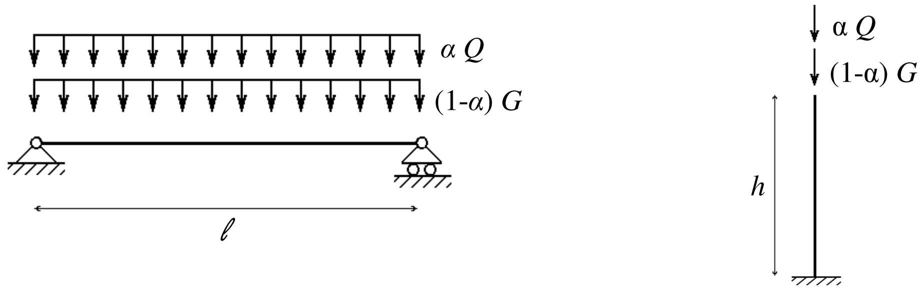
In this section, practical examples are given regarding structural safety assessment of timber structures. Life-cycle structural reliability is considered with respect to deterioration models. Those models are then updated with consideration of different scenarios given by possible NDT results. Depending on different assumptions a reliability based robustness assessment is also proposed, with respect to time evolution of the deterioration process. Resistance properties of timber for stochastic models were assumed from JCSS (2006) when no information was available.

5.1. Single element structures

The first example consists in a simple supported beam with rectangular cross-section. The loads are assumed uniformly distributed along the beam length, *l*. The second example consists in a column, with square cross-section. The loads are considered as concentrated loads applied at the top of the column, as shown in Figure 4. Therefore, the column transmits through compression, the effects of loading to the support elements at the bottom of the column. Both structures are composed by elements of solid timber and the load combinations are modeled by:

$$S = (1 - \alpha)G + \alpha Q \tag{7}$$

where *G* is the permanent load and *Q* is the variable load (live load in this case), α is a factor between 0 and 1, modeling the relative fraction of variable load and permanent load. The modification parameter regarding the effect of load duration and moisture content of timber, k_{mod}, was considered with respect to the load with smaller duration.



a) b)

Figure 4 – Structural models: a) simple supported beam; b) column.

A sensitivity analysis of the parameters used for the reliability assessment procedure was performed regarding the importance of the coefficient of variation (COV) for loads and resistant parameters, for both examples. The results given by this analysis show that when the coefficient of variation for load parameters is increased, the reliability of a given structure decreases, mainly due to the fact that the uncertainty of the problem is also increased. The same procedure was considered regarding the coefficient of variation of the resistance strength. From this analysis resulted that for all values of α, different values of reliability are obtained, however, the higher differences are found for lower values of α. As for the load sensitivity analysis, when the uncertainty is increased lower values of reliability are obtained.

5.1.1. Simple supported beam

The limit state equation was considered to be related to the maximum bending moment at mid-span of the beam. The following limit state equation was used:

$$g = \frac{1}{6} b h^2 \ k_{mod} \, f_m - \frac{1}{8} l^2 \big((1-\alpha)G + \alpha Q\big) \tag{8}$$

where, the bending strength is given by f_m.

The corresponding design equation, according to the combination of loads in Eq. (6.10) of Eurocode 0 (EN 1990:2002), can be written considering the height, h, of the cross-section as the design parameter:

$$\frac{1}{6} b h^2 \ k_{mod} \, \frac{f_{m,k}}{\gamma_m} - \frac{1}{8} l^2 \left((1-\alpha)G_k \, \gamma_G + \alpha \, Q \, \gamma_Q\right) \geq 0 \tag{9}$$

The partial factors used are values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies (EN 1990:2002). Partial factors allow to obtain design values from characteristic values.

Based in a semi-deterministic approach through partial safety factors, the load variables were defined through their characteristic values, and the remaining parameters deterministically. The reliability obtained in consideration to the design of timber elements subjected to bending, as given in Eq. (6.11) of Eurocode 5 – EC5 – (EN 1995-1-1:2004), is presented in Figure 5a), i.e. the design parameter h is determined from (9) for each value of α. However, when assessing existing structures it is necessary to define the present conditions and therefore a design parameter is not considered. Figure 5b) also presents the reliability obtained regarding the parameters, given in Table 2, for a practical example purpose, i.e. the design parameter h = 400mm is fixed for all values of α.

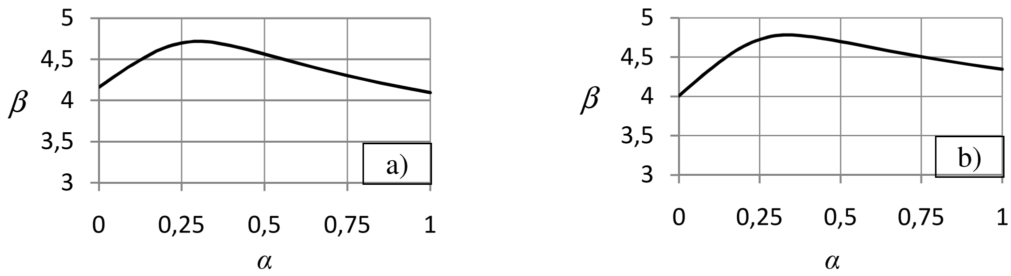


Figure 5 – Reliability index with reference time one year with respect to: a) design assessment with Eq. (9); b) example assessment of the model given in Table 2.

Table 2 – Variables used in the stochastic model for a simple supported beam example (JCSS, 2006).

Variable [X]	*Distribution*	*E [X]*	*COV [X]*	*Description*	*Characteristic values Eq.(9)*
f_m	Lognormal	25 N/mm^2	0.25	Bending strength	5%
G	Normal	6 N/mm	0.10	Permanent load	95%
Q	Gumbel	4 N/mm	0.40	Annual maximum live load	98%
h	Deterministic	400 mm	-	Height of the cross-section	-
b	Deterministic	200 mm	-	Width of the cross-section	-
l	Deterministic	6000 mm	-	Length of the beam	-

Timber is highly dependent of the climatic conditions of the surrounding environment. Also the attack of pathological agents is more significant in a specific range of values for humidity, temperature and solar exposure. In order to assess the differences that climatic factors may have in the reliability level of a structure, an analysis was conducted varying the climatic zones, assuming the models proposed in Wang et al. (2008) (climatic zones A to D, being A the less hazardous). The structural element is given by Table 2 with consideration of durability class 1. The results for the different deterioration curves regarding different climatic zones are presented in Figure 6, where a stochastic degradation model presented in Brites et al. (2008), with r = 1 mm/year and COV = 0.5, is also plotted, for lifetime reference period. In all models a α = 0.5 was considered.

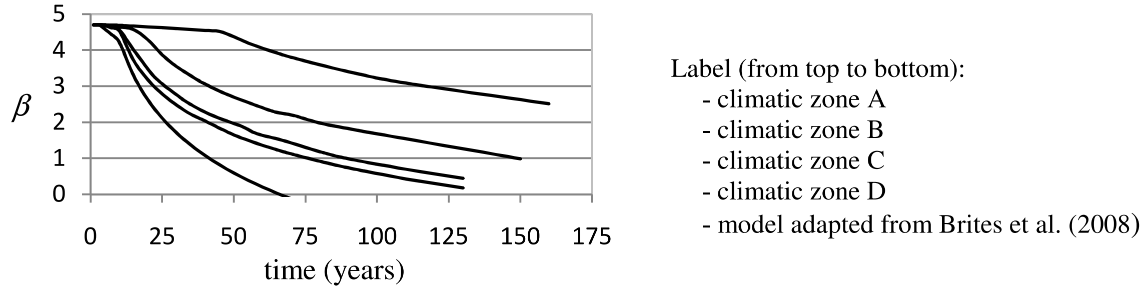


Figure 6 – Reliability comparison between deterioration models for different climatic zones.

The reliability index decreases faster when the climatic conditions are more hazardous because the propagation of deterioration in the timber element is also faster. When comparing the different models, it is visible that the most unsafe curve is given by the stochastic model of Brites et al. (2008) with r = 1 mm/year and COV = 0.5. The main reason is due to the fact that the mean value of penetration ration for the other models is inferior to 1 mm/year.

To evaluate the advantage of using an updating scheme for deterioration models with data from NDT, a hypothetical trial of tests with a Resistograph® device are assumed. This kind of tests allows determining areas with different resistance to the drilling process, thus making it possible to see decayed areas and its depth. With these measurements, the residual cross-section may be derived and therefore the model of deterioration may also be updated regarding the r parameter.

The hypothetical tests performed to the structure were conducted in year 19. For updating purposes the most hazardous model was considered and that 8 tests were performed obtaining different values of residual cross-section (n = number of tests). From those values a sample of penetration ratios were found as presented in Table 3.

The r parameter is often described by a lognormal distribution with coefficient of variation equal to 0.5 (Brites et al. 2008) and two approaches were considered in order to update the deterioration model. These approaches regard vague *prior* information on the mean and on both mean and standard deviation for the updating scheme.

Table 3 – Sample of penetration ratios derived from hypothetical Resistograph® tests.

	***r* (mm/year)**								μ	σ
X_i	0.45	0.52	0.65	0.47	0.40	0.42	0.55	0.54	0.50	0.082
$Y_i = \ln(X_i)$	-0.799	-0.654	-0.431	-0.755	-0.916	-0.868	-0.598	-0.616	-0.70	0.160

Considering vague information on both mean and standard deviation, a 5% quantile value for X was obtained with respect the following equation (JCSS, 1996):

$$X_d = \exp\big(\mu(Y)\big) \cdot \exp\left(-t_{vd} \cdot \sigma(Y) \cdot \sqrt{1+\frac{1}{n}}\right) \tag{10}$$

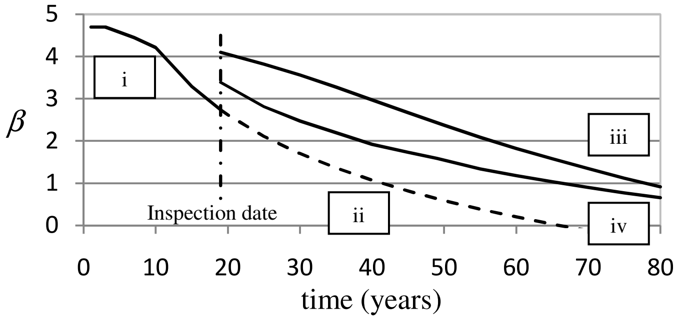
where t_{vd} has a central t-distribution and for this case a value of 1.89 (ν = n – 1 = 7 degrees of freedom and 5% quantile), $\mu(Y)$ and $\sigma(Y)$ are the mean and standard deviation for Y = ln (X). That value and the standard deviation observed for the test results were them used to obtain the mean value of r, from the following equation:

$$x_{0,05} \approx \mu \cdot \exp\left(-1.645\ \frac{\sigma}{\mu}\right) \tag{11}$$

where -1.645 is obtained from the standard Normal distribution function such that $\Phi(-1.645) = 0.05$, and $x_{0.05}$ is the 5% quantile of the Lognormal variable.

A mean value for r of 0.57 mm/year and standard deviation of 0.16 mm/year was obtained and implemented in the deterioration model. In respect to this, the updated model of deterioration of the timber element was considered from the date of the inspection and tests.

For the next approach, it was considered that the information from the coefficient of variation of r is known and equal to the model used previously (COV_r = 0.5). A mean value of 0.58 mm/year and standard deviation of 0.35 mm/year was obtained and used in the updating of the deterioration model. In Figure 7, the updated models may be compared with the first prevision made with a model assumed without any NDT data.



Label: Decay models:

i. – assumed before NDT information
ii. – after NDT information, but not updated
iii. – updated model with NDT data, with no *prior* information
iv. – updated model with NDT data with σ known

Figure 7 – Deterioration model with updating.

By updating the model with information from the NDT's an increase in reliability is well noticed. The differences, although significantly large, may be explained by different reasons in the real life, such as the element may have not been exposed to extreme conditions of humidity or temperature or it might be from a higher durability class than first assumed.

From Figure 7, it is shown that with the model updated with σ known, lower values of reliability are obtained. This is a consequence from the consideration of a coefficient of variation in the *prior* information that is higher than the one observed in the NDT's. From this, it is possible to point out two conclusions: (i) the *prior* information is not adequate to this specific structure and climate and, thus should be disregarded; (ii) or, the number of tests is insufficient and therefore the observation sample is not adequate and should be improved.

It must be paid attention to the fact that other hypothetical results given by the NDT's may had also indicated a decrease in reliability.

For the purpose of this example let us consider, once more, the deterioration model with r = 1mm/year and $COV_r = 0.5$. Since the deterioration phenomenon has a constant activity, also the reliability decreases in time and therefore robustness may also be considered to be decreased. Redundancy index and the redundancy factor are not suitable to be considered for this example since this is not a redundant structure.

For instance, let us consider that this example structure was designed to sustain a period of time of 50 years. For this case, if a limit reliability level of $P_f = 10^{-3}$ for lifetime reference period is considered, then a $TI_{50} = -0.62$ is obtained and, therefore, the damage due to the deterioration process limits the performance and the life expectance of the structure.

Reliability-based robustness indices may be obtained in a similar way regarding the updating of deterioration models with NDT data. If the previously hypothetical data would be considered than, for the case of the model updated with no *prior* information, a $TI_{50} = -0.22$ would be obtained. Although this index presents an indication of a higher robustness, it is erroneous to conclude that updating a model leads to a better or worse level of robustness. Actually, updating a model will only provide a more accurate and precise definition of the structural behavior of a specific element or system of elements, both in terms of reliability and robustness.

5.1.2. Column

The limit state equation was considered to be related to the maximum compression stress along the height of the column. The following limit state equation was used:

$$g = k_{mod}\, f_{c,0} - \big((1-\alpha)G + \alpha Q\big)/A \qquad (12)$$

where, the compression strength parallel to grain is given by $f_{c,0}$. Considering the area, A, of the cross-section as the design parameter:

$$k_{mod}\,\frac{f_{c,0,k}}{\gamma_m} - \Big((1-\alpha)G_k\,\gamma_G + \alpha\, Q_k\,\gamma_Q\Big)/A \geq 0 \qquad (13)$$

Based in a semi-deterministic approach through safety partial factors, the load variables were defined through their characteristic values, and the remaining parameters deterministically. The reliability obtained with consideration to design of timber elements subjected to simple compression, as given in Eq. (6.2) of EC5, is presented in Figure 8. For this analysis it was considered that the height, h, of the column would be such that second order effects on structures (e.g. buckling of slender elements) could be disregarded (in EC5 this respects to $\lambda_{rel} \leq 0.3$).

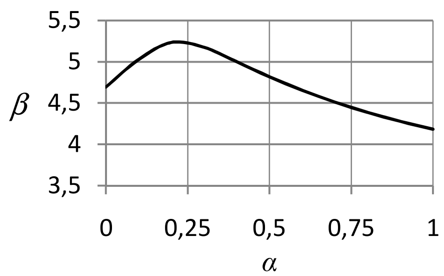


Figure 8 – Reliability index with reference time one year with respect to Eq. (13)

The mean value found for reliability was approximately $\beta = 4.7$ which is higher than the suggested in the Probabilistic Model Code – PMC – (JCSS 2000) for 1 year reference period and reliability class 1 ($\beta = 4.2$) but equal to the requirement in EC 0 annex B (EN 1990:2002) for reliability class RC2. Also, it is noticeable that this limit function equation for simple compression produces similar design reliabilities compared to the simple bending limit equation in terms of evolution of α (see Figures 5a) and 8). However, the reliability values obtained according to the limit states equations as suggested by EC5 for design in simple compression are higher than for simple bending. Another difference is that, in this case, β for $\alpha = 1$ is inferior than β for $\alpha = 0$.

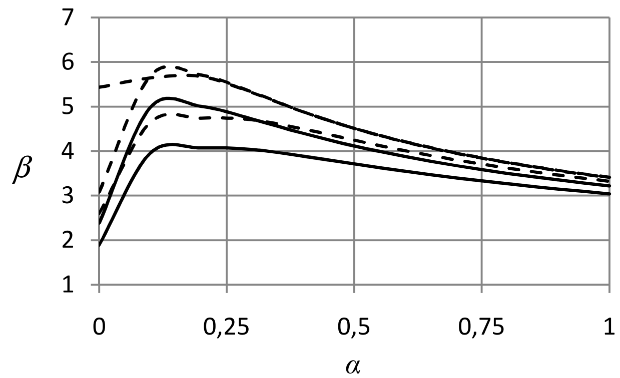
In order to have a practical updating example, a cross-section of 60 × 60 mm^2 was considered for analysis. The objective of this updating analysis is to have a suitable method to update the value of compressive strength parallel to grain of a timber element when NDT results are available and also to consider the uncertainty involved in this process. The data used in the updating scheme is given in topic 2.2 and the uncertainty was modeled as in topic 2.3.

In order to evaluate the validity of the considered correlations a reliability analysis was conducted. Also with this procedure it was intended to analyze the influence of the uncertainty introduced by each separate NDT. Firstly, the resistant parameters of the column were implemented in reference stochastic models considering the values for compression strength parallel to the grain given by the destructive tests (DT). Then, for an updating scheme, the compression strength parallel to the grain was modeled with respect to the linear regression obtained by the Maximum Likelihood method for each NDT. The parameters of the models are given in Table 4. The two references models for $f_{c,0}$ pretend to establish a benchmark for comparison. The first one is modeled by the mean value of the destructive tests and with a coefficient of variation as proposed by the JCSS Probabilistic Model Code (JCSS 2006). The second is modeled by the mean value and COV as given by the destructive tests. For both models, a lognormal distribution was considered.

Table 4 – Variables used in the stochastic model for a column example.

Variable [X]	*Distribution*	*E [X]*	*COV [X]*	*Description*
$f_{c,o}$	Lognormal	μ_{DT}	0.2	Compression strength parallel to grain – mean value of destructive tests (reference model)
$f_{c,o}$	Lognormal	μ_{DT}	$\sigma_{DT} = 0.15$	Compression strength parallel to grain – mean value and COV of destructive tests (reference model)
$f_{c,o}$	-	$a - b \cdot \mu_{RM} + \varepsilon$	-	Compression strength parallel to grain – mean value of Resistograph® tests
$f_{c,o}$	-	$a - b \cdot \mu_{depth} + \varepsilon$	-	Compression strength parallel to grain – mean value of Pilodyn® tests
$f_{c,o}$	-	$a - b \cdot \mu_{Edin} + \varepsilon$	-	Compression strength parallel to grain – mean value of Ultrasound tests
$\boldsymbol{\varepsilon}$	Normal	0	σ_{ε}	Uncertainty parameter of each NDT
G	Normal	60000 N	0.10	Permanent load
Q	Gumbel	40000 N	0.40	Annual maximum live load
A	Deterministic	3600 mm^2	-	Area of the cross-section

According to the NDT data, three models were made with respect to each type of test. The results for the reference models and for the models updated by the correlations between compression strength parallel to grain and results from NDT are given in Figure 9.



Label (from top to bottom, in $\alpha = 0$):
Continuous line: *reference models*
$f_{c,0} = \mu_{DT}$; $COV_{DT} = 0.15$
$f_{c,0} = \mu_{DT}$; $COV = 0.20$

Dashed line: *models with NDT information*
$f_{c,0} = a - b \cdot \mu_{RM} + \varepsilon$
$f_{c,0} = a - b \cdot \mu_{Edin} + \varepsilon$
$f_{c,0} = a - b \cdot \mu_{depth} + \varepsilon$

Figure 9 – Reliability of the reference models and models obtained by NDT information.

The results shown in Figure 9 denote higher values of reliability for the models updated with NDT data. This is mainly due to the consideration of $f_{c,0}$ as a function of the correlation given between the destructive and non-destructive tests. Although uncertainty is implemented through the consideration of the parameter ε, some variability of $f_{c,0}$ is lost since it was already considered in the reference models as a stochastic variable.
The Resistograph® and ultrasound updating scheme must be used with attention since they led to higher values of reliability than the references values. The main differences are found for the maximum value of the reliability curves around $\alpha = 0.12$. However, the data with respect to the Pilodyn® tests presented very similar values to one of the reference models.

5.2. Truss structure

As a practical example, a planar timber truss is considered (Figure 10), submitted to both permanent, *G*, and live load, *Q*. Considering that the elements of this kind of structures are mainly submitted to axial stresses, three different limit state conditions were initially assumed. The limits state conditions are related to tension and compression parallel to grain, and to instability due to buckling of compressed elements.

In this example, before conducting any kind of reliability assessment, the different elements were designed in terms of cross-section dimensions. The structure was assumed to be constructed with chestnut timber. Two different strength classes for timber were considered in the design procedure. The classes considered were D30 and D50 as given by EN 338:1995. For both cases, the element design respected the following premises:

- tension elements (1, 2, 3, 4, 8, 10) with a 75% of cross-section used;
- compression elements (5, 6, 9, 12, 13, 15, 16) with a 95% of cross-section used, with respect to buckling verification;
- cross-section uniformed with respect to an easier construction process;
- vertical elements (5, 7, 9, 11, 13) with the same dimension that the most stressed strut (9).
- both chords are composed by a single 10 m long element.

With these premises it was intended to have a common example of this kind of timber structural system and also to permit that the most conditioning limit state would be related to the compression parallel to grain. Therefore, it would be possible to use the previous mentioned NDT data in an updating methodology.

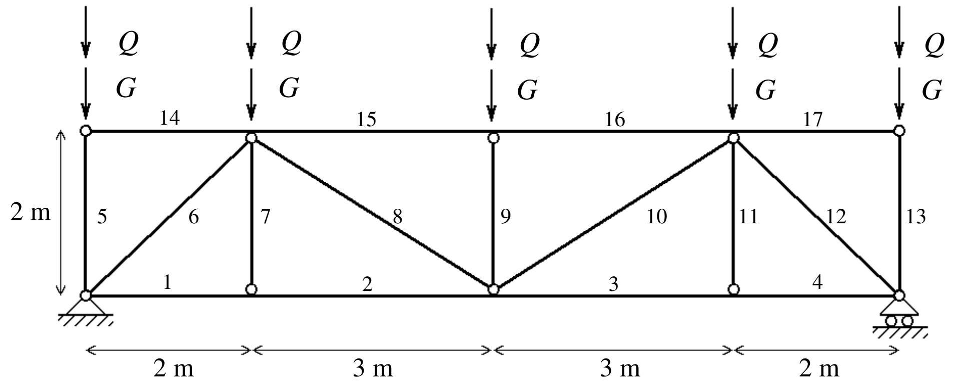


Figure 10 – Structural model of a planar truss.

When evaluating the safety level of a structure, both in terms of reliability or robustness, it is important to assess which is the most likely failure mode as well as to detect the key elements of the structure. In this case, key elements should be understood as the most vulnerable elements which its weakening or local failure would correspond to the worst structural behavior, such as a global failure. Determining the key elements is also important in an updating methodology. Effort in finding new information will be better employed regarding those key elements.

In this example, the failure of the lower or upper chords would correspond to the structural failure of the system, denoting a series system behavior in a reliability analysis. Therefore, they represent key elements of the structure in case of sudden failure of one of these elements. For D30 design the reliability index of the structural system was found to be $\beta = 5.18$ (failure of the upper chord by instability), whereas for D50 the reliability index of the structural system was found to be $\beta = 4.64$ (failure of the upper chord by instability). This is a consequence of the assumed design considerations and also due to the estimation point considered in the reliability analysis.

However, when considering that deterioration of the timber elements might be a relevant parameter in the reliability evaluation of a specific structure, the key elements must be found accordingly. Therefore, in order to define which were the key elements and the most conditioning limit state, regarding deterioration of the timber elements, a reliability assessment was conducted considering a perimetral loss of cross-section, λ, for each element separately. The elements considered as key elements were those which the influence of perimetral loss of cross-section would be more pronounced and lower reliability index would be found. The key elements regarding each limit state are shown in Figure 11.

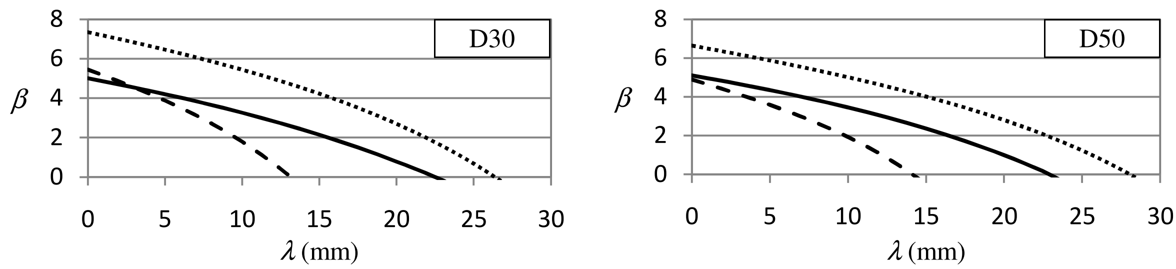


Figure 11 – Reliability index with reference time one year with respect to perimetral loss of cross-section for the limit state condition: buckling (element 9) – dashed line; tension (elements 8 and 10) – continuous line; compression (element 9) – dotted line.

The initial values (λ = 0), for the most conditioning limit state, are in accordance to usual design values. For D30, the reliability index of the key elements is β = 5.01, whereas for D50 the reliability index is β = 4.88. For the structure with D30 timber, tension parallel to grain is the most conditioning limit state at the beginning of loss of cross-section in elements 8 and 10. However, when the reliability index values start to be inferior to 4.5 the most conditioning limit state condition is given by buckling influence in element 9. For D50 timber, the buckling limit state is always the most conditioning, being element 9 the key element. Since timber tension behavior is more influenced by the presence of defects than compression, tension strength parallel to grain has a larger difference from one strength class to another when compared to compression strength parallel to grain. Regarding these results, element 9 is considered to be a key element and the buckling limit state is the most conditioning. Element 9 is also considered as a key element because when weakened, or in case of failure, the stresses are redistributed to the other elements of the truss producing shear and bending stresses in the upper chord. Since that element was not initially designed for that kind of stress, its reliability highly decreases and structural failure is a likely scenario. The increase of shear and bending stresses in the upper chord is mainly noticeable when elements 8, 9 or 10 are weakened.

Regarding the influence of a deterioration process in element 9, the structural reliability index would be β = 6.28 and β = 5.90 for D30 and D50 design, respectively. In D30 design, the failure of element 9 by instability would be followed by the failure of the upper chord by shear. In D50 design, the failure of element 9 by instability would be followed by failure of the upper chord by lateral torsional instability. Regarding the removal of element 9, the redundancy factors β_R = 1.92 and β_R = 2.38 were found for D30 and D50 design, respectively. Although the structural reliability index is higher in the case of D30 design, the redundancy factor is higher in the case of D50 design. This situation is due to the fact that the removal of element 9 leads to a higher difference between the intact and damaged reliability indexes in the case of D30 design.

After assuming the information taken from EN 338:1995, an updating methodology was implemented with consideration to the data given in topic 2.2. With respect to the new information and removal of element 9, updated redundancy factors were calculated with respect to the D50 design. The updated redundancy factors and relative difference with the previous redundant factor are given in Table 5, for each different source of information.

Table 5 – Updated reliability and redundant factors by NDT data and relative difference with the redundant factor assuming EN 338 (1995) information.

	β_{intact}	$\beta_{damaged}$	β_R	*Relative difference (%)*
EN 338:1995	4.640	2.696	2.387	---
Resistograph®	5.288	2.953	2.264	5.11
Pilodyn®	5.159	2.927	2.311	3.18
Ultrasound	5.286	2.950	2.263	5.21

The updated redundancy factors are similar for the updating with Resistograph® and ultrasound techniques since the uncertainty parameter given by the Maximum Likelihood method is similar for both NDT. In each case, the updated redundancy factor is similar to the redundancy factor obtained in the D50 design with consideration to EN 338:1995. Although the initial and damaged reliability indices from the EN 338 (EN 338:1995) design are significantly different from the updated values, the redundancy factor is similar.
After updating the resistance parameters, a deterioration process was considered for elements 9 and 15 separately and the reliability of the structural system was then calculated (Figure 12). When considering only the deterioration in element 9, a parallel system was considered and the necessary failures of the elements for a global collapse were considered as independent events. However, load redistribution on other elements was considered during the process of deterioration of an isolated element. The updating information consists in the information given by the Pilodyn® tests. Since no information is known for the environmental conditions, it was considered a loss of cross-section independent of time. Therefore the structural reliability is directly related to a cross-section loss parameter, λ. When new information is obtained, regarding the rate of deterioration, the loss of cross-section may be then related to the penetration ratio, r. After gathering the information about the penetration ratio a model of deterioration may be implemented and correlated with the corresponding reliability. This finally leads to a time evolution curve for reliability assessment of the structure.

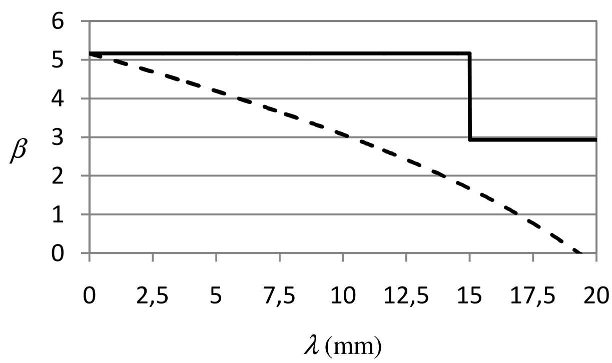


Figure 12 – System reliability index with reference time one year with respect to perimetral loss of cross-section for element: 9 – continuous line; element 15 – dashed line.

From the analysis of Figure 12, it is visible that the system reliability is more conditioned by the loss of cross-section of element 15. However, when considering the loss of cross-section of element 9, a sudden decrease of the system reliability is visible due to the loss of resistant capacity of that element (failure due to compression instability). Although the system reliability is always lower for the loss of cross-section in element 15, in terms of robustness, the decay process in element 9 may be considered more relevant since the same action leads to effects not proportionate to the causes.
Since no time relation is present in Figure 12, it is not possible to obtain a time index, *TI*. However, when the model is updated with information about the deterioration rate, it becomes an intuitive procedure. This information may be gathered through NDT and then updated as in the previous examples.

6. CONCLUSIONS

This work presented a framework for reliability assessment of timber structures using data gathered from NDT results. For that purpose, three example cases were presented: (i) a simple supported beam; (ii) a column; and (iii) a truss structure. In those examples, the structural reliability was calculated in both design and assessment phases. After updating the resistant properties, deterioration models were implemented and robustness indicators were considered. From the analysis of the examples several conclusions may be taken.
As found in the truss structure example, different strength classes of timber, even for the same design considerations, led to different reliability and robustness levels. Also, not always a higher reliability

index produces higher indicators of robustness, depending in the nature of the structural failure that is associated.
Determining the key elements was found to be a fundamental step in order to understand the level of robustness of a structure. By changing the design of key elements, different load paths and consequently different structural systems may be found. In that case, a parallel system may be turned into a series system with great interest specially to increase the reliability of the structure. The structural system may, also, influence both the reliability of the system as well as its robustness level.
Although model updating is a useful method for safety assessment of existing structures, NDT data updating does not increase or decrease robustness. The objective of NDT data updating is to allow for a better definition the characteristics of the structural elements, with special regard to the key elements of a structure. From the deterioration model analysis, it is concluded that different indicators for robustness assessment should be used depending in the type of possible actions / loads that might influence the structure.
Further investigation should address a better definition of climatic conditions and parameters for different geographical locations, as well as its influence in timber elements. Effort should also be taken into obtaining a larger database for NDT results and its correlations with timber resistant properties in order to have more reliable information for data updating. Future work should address the development of robustness indicators.

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REFERENCES

Biondini, F. (2008). “Robustness, Redundancy and Indeterminacy”, *HE2008*, Rome, November.

Brites, R.D., Neves, L.C., Machado, J.S. and Lourenço, P.B. (2008). “Avaliação da Segurança de Cobertura Tradicional de Madeira Sujeita a Degradação Biológica”. *CINPAR 08*, Aveiro, June.

EN 1990:2002 (2002). “Eurocode 0 – Basis of Structural Design”. European Committee for Standardization (CEN), Brussels.

EN 1995-1-1:2004 (2004). “Eurocode 5 – Design of Timber Structures – Part 1-1: General - Common rules and rules for buildings”, European Committee for Standardization (CEN), Brussels.

EN 338 (1995). “Structural timber – Strength classes.” European Committee for Standardization (CEN), Brussels.

Feio, A. (2005). *Inspection and Diagnosis of Historical Timber Structures: NDT Correlations and Structural Behaviour*. Ph.D. thesis, University of Minho, Portugal.

Frangopol D.M., and Curley, J.P. (1987). “Effects of Damage and Redundancy on Structural Reliability”. *Journal of Structural Engineering*, 113, 1533-1549.

Fu, G. and Frangopol, D. (1990). “Balancing weight, system reliability and redundancy in multiobjective optimization framework”. *Journal of Structural Safety*, 7, 165-175.

Joint Committee on Structural Safety (1996). *Background documentation Eurocode , Part 1: Basis of Design*.

Joint Committee on Structural Safety (2000). *JCSS Probabilistic Model Code, Part 1: Basis of Design.* Probabilistic Model Code.

Joint Committee on Structural Safety (2001). *Probabilistic Assessment of Existing Structures*. Edition by D. Diamantidis, RILEM Publications.

Joint Committee on Structural Safety (2006). *JCSS Probabilistic Model Code, Part 3: Resistance Models – 3.5 Properties of Timber*. Probabilistic Model Code.

Leicester, R.H. (2001). “Engineered durability for timber construction”. *John Wiley & Sons, Ltd. CSIRO*. Highett, Australia, 216-227.

Wang, C, Leicester, R. and Nguyen, M. (2008). “Probabilistic procedure for design of untreated timber poles in-ground under attack of decay fungi”. *Reliability Engineering and System Safety*, 476-481.