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Reliability based design of interventions in deteriorated timber structures

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Abstract

When accommodating new uses or mitigating the consequences of deterioration, the strength increase of existing structures is significantly more onerous than a similar increase at the design stage of new structures. The safety methods prescribed in current standards were defined for the design of new structures and are frequently conservative for the assessment and repair of existing structures. This work will introduce the fundamental aspects of structural reliability and their application in the context of existing timber structures regarding the use of target reliability indices. Firstly, the fundamental methods of structural reliability are introduced. The use of reliability methods requires the use of more detailed information in respect with material properties, loads and model uncertainty. The main sources of such information are described. After overviewing the fundamental methods of structural reliability, methods to introduce additional information, namely results of non-destructive tests in the structural assessment are discussed. Finally, the intervention on a timber structure will be analyzed, within a case study, by considering different repair scenarios that lead to discussions on different suitable safety thresholds for existing and repaired structures.

Keywords:

timber structures; reliability; safety assessment; intervention; target reliability index

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Target reliability indices on timber structures

1. Introduction

When accommodating new uses or mitigating the consequences of deterioration, the strength increase of the existing structure is significantly more onerous than a similar increase at the design stage of the new structure. Repairing (process aiming at returning a damaged structure to its original or prior condition) or upgrading (process aiming at reaching a higher level of performance than the existing and/or original level) existing structures are usually complex operations, which results in higher direct costs. Besides, both require significant limitations to the use of the structure during the intervention procedures, by leading to significant users costs too. The safety methods prescribed in current standards were defined for the design of new structures and are frequently conservative for the assessment and repair of existing structures.

Consequently, it is fundamental to use a safety assessment methodology more flexible than the traditional semi-probabilistic approach present in codes. Semi-probabilistic methods, like the partial safety methods, take the uncertainty in structural variables into account in a simplified manner. Each variable is described by a single value (e.g., mean, nominal or characteristic value) while each load and strength variable is affected by a partial safety factor dependent on its variability, on the associated probabilistic distribution and the target safety level.

The uncertainty deriving from the safety margin can be taken into account by increasing the partial safety factors, by resulting in a small overdesign of structures which has negligible costs for new structures. Regarding their design, the partial safety factors are adequate, except in some very particular cases (e.g., very high consequences of failure structures, materials or loads not covered in codes). However, the safety assessment of the existing structure differs from the design of the new structure in three main aspects: *i*) costs, *ii*) lifetime period, *iii*) uncertainty quantification. Following, these aspects are further discussed. Firstly, increasing the safety of the existing structure is significantly more expensive than a similar increase at the design stage. In fact, the main cost will typically be the rising in material costs at the design stage, while for the existing structure it involves closure or limitation of use, more complex retrofitting or upgrading methods, and overall greater risks. Secondly, the remaining life of the existing structure is usually shorter than that of the new structure. This might lead to a reduction in the representative values of loads. As last point, the uncertainty present in the existing structure is significantly different from that expected in the new structure. On one hand, measurements and non-destructive tests can be used in the existing structure to reduce existing uncertainty. On the other hand, assessment of existing structures is required when doubts on the safety of the structure exist, in particular as a result of deterioration. Deterioration is extremely complex to predict and characterize, adding a new source of uncertainty in the safety assessment.

Structural reliability methods use a more general approach to quantify structural safety. They explicitly include the uncertainty in each parameter influencing the performance of the structure, and they define safety thresholds that take into account the importance of the structure and the cost of improving safety. Probabilistic methods are based on the computation of the probability of failure, by taking into account the uncertainty in all structural parameters which stand for material properties, loads, model errors, and geometry. Being compared to semi-probabilistic methods, probabilistic methods require more information and significantly higher computational power. However, the use of probabilistic methods overcomes the main limitations described in the assessment of existing structures. On one hand, a target probability of failure can be defined, based on the equilibrium between construction/repair cost and risk. Furthermore, the differences in uncertainty in structural parameters are explicitly taken into account. Finally, the results of new results and information on the structure including measurements or non-destructive test results can be consistently incorporated in the safety assessment.

Considering the potential advantages of using probabilistic methods in the assessment of existing structures, the present work will focus on the use of reliability analysis in the context of timber structures.

2. Framework

2.1. Reliability methods

Reliability analysis of timber structures requires three main steps: definition of resistance properties, definition of effects of actions, and computation of the probability of failure or reliability index (Köhler, 2007; Köhler et al. 2007). Basically, structural reliability aims at quantifying the safety of structures in terms of the probability of a limit state being violated, denoted probability of failure, p_f. The failure of a structural element is considered when the value of its resistance R is exceeded by the value of the load effect S resultant of a determined loading Q, on a specific element. Therefore, p_f may be described as the probability that the structural resistance R, modelled by a random variable with a known probability function $f_R(r)$, being inferior or equal to the load effects S, equally modelled by a random variable with a known probability function $f_R(s)$ such as defined in Eq. 1.

$$p_f = P(R - S \leq 0) = \int_{R-S\leq 0} f_{RS}(r,s)\,dr\,ds \tag{1}$$

This integral can only be computed analytically in very simple cases. In the last decades, a wide range of methods have been developed to compute the probability of failure of complex problems. When both R and S are given by normal random variables, with means μ_R and μ_S and variances σ_R^2 and σ_S^2 respectively, the probability of failure according to Cornell (1969) may be stated as in Eq. 2:

$$p_f = \Phi\left[\frac{-(\mu_R - \mu_S)}{\left(\sigma_S^2 + \sigma_R^2\right)^{1/2}}\right] = \Phi(-\beta) = 1 - \Phi(\beta) \tag{2}$$

where $\beta = \mu_M/\sigma_M$ is defined as reliability index, with μ_M and σ_M being the mean and standard deviation of the safety margin ($M = R - S$), and Φ ( ) represents the standard normal distribution function. In this case, it is visible that p_f increases when either one of the variances increase or when the difference between means of R and S decreases. Although these are the fundamental principles for a reliability assessment, when the limit state is non-linear or the random variables are non-normal, the procedure above cannot be applied directly. The random variables must be approximated by normal variables whereas the limit state function must be approximated by a linear function, within the interactive procedure known as First Order Reliability Method (FORM).

With concern to the probabilistic safety assessment of timber structures, the key resistance properties (or reference properties) are the bending strength (f_m), the bending modulus of elasticity (E_m) and density (ρ_m). Probabilistic models for these parameters are proposed in the Joint Committee for Structural Safety Model Code (JCSS, 2006). In this document, a lognormal distribution is proposed for the bending strength and modulus of elasticity, with coefficients of variation of 25% and 13%, respectively, whereas density is represented by a normal distribution with coefficient of variation equal to 10%. Considering the JCSS model code, the other resistance properties of timber can be obtained based on the key properties through empirical expressions.

As mentioned by Faber et al. (2014), the tail behaviour of the probability distributions for timber material characteristics plays an important role in the overall probabilistic modelling. In this case, the parameters that define the probability distributions must be estimated with care. To this end, Bayesian and Maximum Likelihood estimations have been applied with success (Faber et al. 2014; Sousa et al. 2015, 2016b). Correlation analyses and statistical tests have also been used while discussion about the applicability of interval estimation instead of point estimation methods to determine the distribution parameters has been provided in Jenkel et al. (2015).

2.2. Bayesian updating

Throughout their lifetime, structures change due to many aspects from natural causes to human decisions or even by extreme or accidental actions, only to point a few. The knowledge on the structure also changes from the design stage to the construction and the assessment phases. In the design stage, it is assumed that structure properties are those of similar structures whereas in the construction phase visual inspection provides further information. In addition to these, samples of material, non-destructive tests or load tests can be carried out when assessing the structure. Thus, the assessment of existing structures should be regarded as a successive process of model updating and consequent evaluation regarding new information. Bayesian methods are therefore suited to this aim because they allow the combination of new information with pre-existing data, in a consistent manner.

The assessment of structures usually starts by analysing the structure using conservative values (or probabilistic distributions) to all relevant parameters, based on data and experience on similar structures. An initial analysis will indicate which variables are critical to the safety of the structure and, consequently, for which a conservative estimate has greater impact. If this analysis indicates that the structure is unsafe, it might be interesting to gather further information and obtain values or probabilistic distributions of the critical parameters closer to reality and, typically, less conservative. It must be kept in mind that the structural safety is fundamentally dependent on the extreme values of structural parameters, which are strongly influence by the standard deviation or by the existing uncertainty. Reducing this uncertainty will lead to the improvement of safety estimates in most cases.

The information gathered is often limited and localized (e.g., a small timber sample only provides localized information that must be extrapolated to the entire beam carefully), and is associated with significant statistical uncertainty. As a result, its combination with prior information can be more informative that either source of data individually.

In Beconcini et al. (2016), a methodology for the probabilistic reliability assessment of heritage buildings is presented where Bayesian updating techniques were implemented for a rational use of the collected information. In Sousa et al. (2016a), a methodology for the holistic assessment of timber elements was also described where information gathered in different scales was combined by following a probabilistic framework that allows for the structural assessment of existing timber elements with possibility of inference and updating the mechanical properties of structural elements through Bayesian methods. Furthermore, Bayesian methods were used, according to Sousa et al. (2013), in order to update the information on the mechanical properties of timber through semi and non-destructive test results.

A review on the onsite assessment of structural timber members by means of hierarchical models and probabilistic methods is also provided in Sousa et al. (2015) where the applicability and limitations of statistic and probabilistic methods on the prediction and inference of timber's reference material properties are discussed and exemplified.

2.3. Reliability targets

The decision on the need to repair the structure or on the design of upgrading solutions requires the definition of a target safety level, by defining the acceptable minimum level of performance. For new structures, such targets are defined in codes, either implicitly through partial safety factors, or explicitly thought prescribed target reliability indices. In order to design both for ultimate and serviceability limit states, diverse target reliability indices are established for various structural situations by considering different consequences classes, reference periods of time and relative cost of safety measures. The European standard EN 1990 (CEN, 2002), also known as Eurocode 0, refers three reliability classes RC1, RC2 and RC3 associated with three consequences classes CC1, CC2 and CC3. The definition of the three reliability classes is given in Table 1, and the correspondent minimum target values for the reliability index β regarding ultimate limit states are stated in Table 2. RC is normally related directly to CC.

Table 1: Definition of consequence classes (adapted from CEN, 2002).

Consequences classes	Description	Examples of buildings and civil engineering works
CC1	Low consequence for loss of human life, and economic, social or environmental consequences small or negligible	Agricultural buildings where people do not normally enter, greenhouses
CC2	Medium consequence for loss of human life, economic, social or environmental consequences considerable	Residential and office buildings where consequences of failure are medium
CC3	High consequence for loss of human life, or economic, social or environmental consequences very great	Grandstands, public buildings where consequences of failure are high

Table 2: Recommended minimum values for reliability index β related to ultimate limit states (adapted from CEN, 2002).

Reliability Class	Minimum values for β	
	1 year reference period	50 year reference period
RC1	4.2	3.3
RC2	4.7	3.8
RC3	5.2	4.3

While it is valid for the design of new structures within the scope of Eurocode, this approach is not satisfactory for the assessment of existing structures and the design of upgrading solutions. In fact, it is uneconomical to require all existing structures to comply with safety target levels calibrated for new structures. This would result in the need to retrofit every structure denoting even mild signs of deterioration which is clearly impossible.

Due to the shorter design life of existing structures and the higher costs of increasing safety, it is reasonable to accept lower safety thresholds for existing structures as discussed above.

The assessment of existing structures may be considered through the procedure defined in ISO 13822 (ISO, 2010), which is based on the principles of structural reliability and consequences of failure. This standard is applicable to the assessment of any type of existing structure that was initially designed or analysed according to accepted engineering principles, as well as it is valid for structures built with good workmanship and experience principles. This standard is intended to serve as a basis for preparing national standards or codes of practice by taking into account both the current engineering practice and also the economic conditions. It is applicable to heritage structures if additional considerations are taken. Moreover, it may be applied to any sort of materials, however specific adaptations may be needed depending on the material. The assessment procedure may be considered when the anticipated change of use or extension of design life expectancy is required or entailed by the reliability check due to exterior load effects. ISO 13822 (ISO, 2010) is based on ISO 2394 (ISO, 2015) principles which propose a more detailed approach to the definition of the target reliability, considering both the consequences of failure and the cost of improving safety, as shown in Table 3.

Table 3: Target reliability index, β_{target} (lifetime reference period) according to ISO 2394 (2015).

Relative cost of safety measure	Consequences of failure			
	small	some	moderate	great
High	0	1.5	2.3	3.1
Moderate	1.3	2.3	3.1	3.8
Low	2.3	3.1	3.8	4.3

This proposal can be used for assessing existing structures, by simply considering the cost of safety measure as large. However, a more detailed analysis of the costs involved in the decision making process can yield more consistent and reasonable results. After the structural assessment, a decision must be made as to upgrade the structure or not, and how to achieve that improvement in case of upgrading.

On a first approach, the target reliability indices for structural design or assessment of structures should be defined based on cost optimization, by considering the consequences and nature of the failure, economical losses, environmental and social impacts, and the cost of measures to reduce the probability of failure. Since construction/repair only occur once, the equilibrium between costs and risk must consider the risk over the entire lifetime. This means that economic constraints influence the acceptable lifetime probability of failure and structures with different design lifetimes should have the same lifetime reliability, which results in different annual probabilities of failure. A purely economic analysis can be used in the cases where structural failure results in no human lives lost. Otherwise, measures of individual risk, like the Life Quality Index, can be used to guarantee that decisions which are optimal from an economic viewpoint do not result in unacceptable risk to users.

For existing structures, the safety cost is usually significantly higher than that for new structures. At the design stage of new structures, safety can be increased by changing the dimension of elements or the strength of timber or connectors, with marginal costs. For existing structures, the costs are significantly higher, not only due to the increased complexity of an upgrade solution, but also due to the loss of use during a period, with resulting business losses or relocation costs. This increase in safety cost shifts the optimal point, which results in lower optimal design parameters and lower safety levels, as long as human safety constraints are not violated. As a consequence, the target reliability index depends on the consequences of failure influencing the failure cost, on the cost of improving safety, and on the project time horizon influencing the human safety constraint.

For instance, ISO 2394 (ISO, 2015) indicates that the target level of reliability should depend on the balance between the consequences of failure and the cost of safety measures. From an economic point of view, the objective is to minimize the total working-life cost.

The difference in cost for the safety improvement between new and existing structures is mostly dependent on the loss of use of the structure, design, survey, mobilizing construction crew. On the other hand, once the decision to upgrade has been taken, the difference in cost between different alternatives only depends on the materials and application, which is higher for existing structures. Indeed, more expensive materials are usually employed in upgrading as a result of geometrical and weight restrictions, difficulty in access and need for faster construction methods. For this reason, Steenbergen et al. (2015) proposed two different reliability targets. The first, β_0, defines the minimum reliability index for which not upgrading is acceptable. The second index, β_{up}, defines the target reliability index for the design of the upgrade. If only economic optimization is considered, the target reliability is independent of the lifetime of the structure. Table 4 presents the proposal of Steenbergen et al. (2015) for the target reliability index considering only economic consequences.

Table 4: Target reliability index considering only economic consequences (Steenbergen et al., 2015).

Consequence class	β_{up}	β_0
CC1	2.8	1.8
CC2	3.3	2.3
CC3	3.8	2.8

When the potential loss of lives is considered, not only the importance of the structure must be considered, but also the time horizon of the structure and the expected number of fatalities. The number of potential fatalities can be estimated based on the importance of the structure and the expected collapsed area, A_{col}, by following the failure of the element.

Except for exceptional structures, the target reliability is dominated by the risk of loss of human lives, except for very short lifetimes. For higher consequence classes, the risk grows significantly with the collapsed area, which results in significant differences with the collapse area. For buildings of CC1 large groups of users are not expected and, consequently, the target reliability is independent of the collapsed area. The proposal of Steenbergen et al. (2015) for both targets is presented in Tables 5 and 6.

Table 5: Target reliability index for design of upgrade actions β_{up} (Steenbergen et al., 2015) for lifetime equal to reference period.

Lifetime	1 year		5 years		15 years		30 years	
A_{col} (m^2)	< 500	> 500	< 500	> 500	< 500	> 500	< 500	> 500
CC1	3.1	3.1	2.8	2.8	2.8	2.8	2.8	2.8
CC2	3.6	4.2	3.3	3.8	3.3	3.5	3.3	3.4
CC3	3.9	4.8	3.8	4.5	3.8	4.2	3.8	4.1

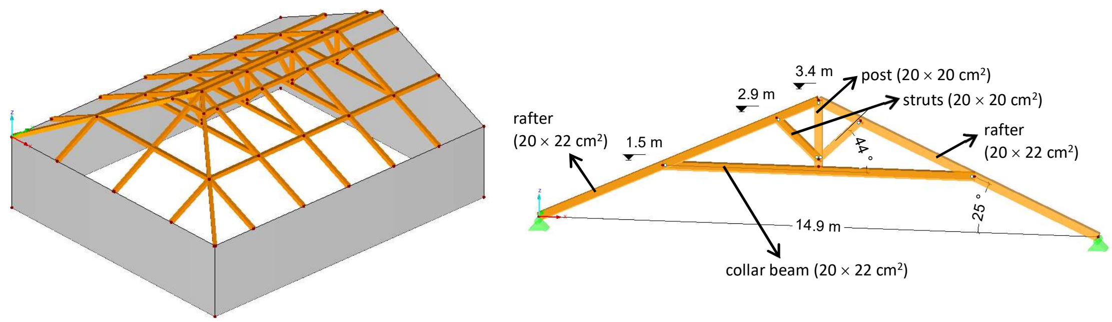
Table 6: Target reliability index for defining the need to upgrade β_0 (Steenbergen et al., 2015) for lifetime equal to reference period.

Lifetime	1 year		5 years		15 years		30 years	
A_{col} (m^2)	< 500	> 500	< 500	> 500	< 500	> 500	< 500	> 500
CC1	3.1	3.1	2.6	2.6	2.2	2.2	1.9	1.9
CC2	3.6	4.2	3.2	3.8	2.8	3.5	2.6	3.4
CC3	3.9	4.8	3.5	4.5	3.2	4.2	3.0	4.1

3. Application on a timber roof truss

3.1. Initial design

A timber truss pertaining to a three pitched timber roof will be considered as case study to demonstrate the influence on using target reliability indices for the intervention design in existing timber structures. The timber roof is composed by four collar beam trusses, spaced 3 m from each other. The disposition of elements was based on the structural configuration of the Chimico Laboratory, a Portuguese neoclassic building from the 18th century, located in Coimbra (Figure 1a). The collar beam trusses of this roof were studied in several previous works regarding its inspection, geometric survey and safety analysis (Lourenço et al., 2013), as well as full scale testing and repair interventions (Branco et al., 2017). Considering the same geometry, the initial design following Eurocode 5 (CEN, 2004) was considered for one of the central trusses (Figure 1b). The design was made as it was for new structures according to the currently applicable design codes.



a) b)

Figure 1: Configuration of the timber roof: a) three-dimensional perspective; b) planar collar beam truss.

For the initial design, load combinations with permanent (i.e. weight of the trusses, roof tiles and sheeting), imposed (i.e. live load) and wind loads (i.e. upwind and downwind) were considered. The expected values (mean values), coefficient of variation (CoV) and probabilistic distributions used for each load type are provided in Table 7. Strength and stiffness variables that were used in the probabilistic assessment are also provided in the same table. As to obtain design values according to the principles of Eurocode 5, partial safety factors were used together with the characteristic values for resistance and load variables. In terms of mechanical properties, the elements of the timber truss were considered to be made of pine with strength class C24 (CEN, 2016).

During the design stage, it was found that the most conditioning limit state corresponds to the verification of combined bending and axial compression on the rafters, while the critical load combination corresponds to the presence of permanent load together with wind load. From the design and accounting to a homogenization of dimensions along the truss, a cross-section of 20×22 cm^2 was used for the rafters and collar beam, whereas for the post and struts a cross-section of 20×20 cm^2 was used (Figure 1b).

Table 7: Variables used in the design of the timber roof truss.

Variable	Distribution	Expected value	CoV
Bending strength, f_m	Lognormal	36.2 N/mm^2	0.25
Bending stiffness, E_m	Lognormal	11000 N/mm^2	0.13
Compression parallel to the grain strength, $f_{c,0}$	Lognormal	29.2 N/mm^2	0.20
Permanent load, P	Normal	1.00 kN/m^2	0.10
Live load, Q	Gumbel	0.32 kN/m^2	0.40
Wind load - upwind, W_{up}	Gumbel	0.23 kN/m^2	0.35
Wind load - downwind, W_{down}	Gumbel	0.32 kN/m^2	0.35

By taking into consideration these cross-section dimensions, the reliability analysis was performed with the purpose of computing the reliability index of this truss. The analysis considered the probability of failure for the most critical element (i.e. rafter) by accounting the limit state equation and load combination that conditioned the initial design. Following the definition of probability of failure provided in Eq. 1, the limit state equation, G, for the present study case is presented in Eq. 3:

$$G = 1 - \left(\left(\frac{\sigma_{c,0}}{f_{c,0} \cdot k_{\text{mod}}} \right)^2 + \left(\frac{\sigma_m}{f_m \cdot k_{\text{mod}}} \right) \right) \quad (3)$$

where $\sigma_{c,0}$ and σ_m are the design compressive stress along the grain and the design bending stress respectively, which results from the load combination with permanent and wind loads. On the other and, $f_{c,0}$ and f_m are the compressive strength along the grain and the bending strength respectively. The parameter k_{mod} is the modification factor for duration of load and moisture content. The resistance parameters were defined according to the values given in EN

338 (CEN, 2016) for class C24, while probability distributions and variation were established according to the Probabilistic Model Code for timber (JCSS, 2006). Geometry of the cross-section (i.e. width and height) was considered deterministic and constant, as it is assumed that a new construction evidences low dimensional variation along the length of the elements.

Permanent loads were modelled as a normal distribution, by providing an expected value of 1.0 kN/m^2 with CoV of 10%. Wind loads were calculated through Eurocode 1 (CEN, 2005) by resulting in a mean exterior wind pressure of 0.32 kN/m^2 for downwind and 0.23 kN/m^2 for upwind. Since the location and configuration of the building were known, Gumbel distributions with CoV of 20% were considered for wind speed, by resulting in a final wind load with a CoV of 35%.

In this work, a FORM algorithm implemented in PRADSS (Sørensen, 1987) was used to compute the reliability index of the structure, by presupposing failure of the whole structure if a critical element failed. After performing the reliability analysis a reliability index, $\beta = 4.85$ ($p_f \approx 6.3\times10^{-7}$) was obtained. This value is consistent with the target values proposed in Eurocode 0 (CEN, 2002).

3.2. Damaged structure

Roof structures (comprising both covering and structural components) are critical elements on a building, as being subject to the action of the exterior environment they may be influenced by natural and physical agents that originate different pathologies. In the case of timber roofs, decay due to fungi or xylophages insects is often encountered, especially on the rafters featured by direct contact to the roof cover and on the supports due to rising damp. Probabilistic decay models for timber elements have been discussed (e.g. Leicester et al. 2009, Sousa et al., 2014) where it is assumed that decay process may be modelled by bi-parametrical models. The first parameter corresponds to the time before noticeable decay starts whereas the latter deals with the annual decay penetration rate which depends on climate, durability, and structural conditions of the timber element.

Considering that the truss is exposed to decay along its lifetime, a safety analysis to the rafter was made by taking into account the homogeneous decrease of size on all faces of its cross-section. The rafter was chosen since it was determined as the most critical element on the initial design. As previously mentioned, the rafter is the structural element which often presents decay on existing timber roofs. The reliability analysis considered the most critical combination of loads (i.e. permanent load combined with wind load). The results are presented in Figure 2, in terms of reliability index for each decrease of cross-section. This analysis allows to verify what would be the decrease on the element's cross-section needed to reach a target reliability index for defining the need for upgrade, β_0, as mentioned for example in Steenbergen et al. (2015). Therefore, two target reliability indices are shown in Figure 2, regarding a reference period of 1 year and 30 years. Moreover, two modelling approaches were considered in relation to redistribution of stresses due to the loss of cross-section of one of the rafters. In the first scenario, it is considered that no redistribution of stresses is made to the other elements of the truss while the rafter is losing resistant cross-section. On the other hand, the second scenario implies that stresses are redistributed along the truss during the decay process.

It is ascertained that, by not considering a redistribution of stresses during the decay process, lower decreases of cross-section correspond to a higher loss of structural reliability compared to the case where it is assumed that the truss is able to redistribute the stresses from one element to the other. In the more conservative approach (no redistribution), the value $\beta_0 = 3.6$ is reached for a cross-section decrease of 11 mm on each face whereas $\beta_0 = 2.6$ is reached for a cross-section decrease of 20 mm on each face. Meanwhile, the value $\beta_0 = 3.6$ is reached for a cross-section decrease of 22 mm on each face for the case with redistribution, whereas $\beta_0 =$

2.6 is reached for a cross-section decrease of 38 mm on each face. These results indicate that the stress distribution approximately doubles the limit depth for cross-section decrease before achieving a target reliability index to define the need of upgrading.

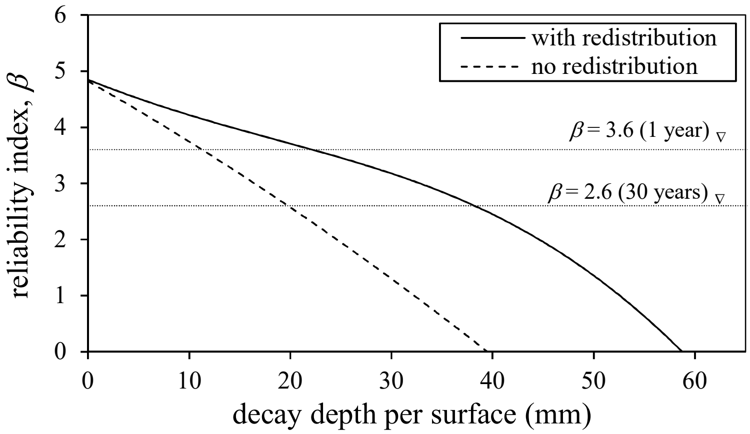


Figure 2: Reliability index of the timber truss according to different loss of cross-section dimensions.

The results presented in Figure 2 correspond to the specific analysis of the rafter with loss of cross-section within this specific collar beam truss but are indicative of the procedure for determining the reduction of cross-section that leads to the need to intervene on the structure. Therefore, it is important to obtain reliable information on the geometry of the residual cross-section when analyzing existing timber structures. To that aim, non-destructive tests are often used and its information may be implemented within a reliability safety analysis by consideration of new information. As previously mentioned, this information may be implemented through Bayesian updating approaches. The work of Sousa et al. (2013) may be consulted for further information and examples on the use of non-destructive test data for reliability-based assessment of existing timber structures.

3.3. Interventions and target reliability indices

In the presence of old timber roofs, there is a common propensity for a full replacement by a new structure and sometimes even to use another material. However, this is often not the most appropriate solution, when considering both the preservation of the building's identity, and the economic and structural optimizations. These points are critical in historical constructions, where execution issues usually arise due to in-situ constraints, which sometimes render the replacement option unfeasible. This situation is exemplified for the Portuguese construction context by Rodrigues (2013) who verified through the cost analysis that the replacement of the tie-beam and rafter from the timber truss would be approximately three times more costly than localized repairs. Within the Portuguese context, another example is given by Ilharco et al. (2010) where, through local repairs and reinforcement using steel screws and timber elements, it was possible to obtain a rehabilitation cost for a timber roof structure of approximately 20% of the value that would correspond to its full replacement.

Attending to these premises, thus taking into account that local repairs are substantially less costly than full replacement of the elements, two types of interventions will be considered as to exemplify the use of target reliability indices depending on relative cost of implementing a safety measure.

In the present case study, it will be considered that intervention is required on the rafter from the timber truss. The first intervention option is the localized repair of the rafter using timber pieces connected by screws on each side of the rafter. This is a traditional repair solution and details of this intervention are discussed in Branco et al. (2017) for a timber truss with the same configuration of this case study. The second type of intervention consists in the replacement of the rafter element by a new one. Considering the costs concerning each intervention, it can be considered that the local intervention produces low relative costs for implementing the safety measures, whereas the replacement of the element produces moderate relative costs.

For the safety analysis and design of the intervention, it was considered that the rafter would fail at the point of higher stress near the connection with the collar beam. Therefore, it was stated that the cross-section did not present any further contribution to the resistant cross-section of the element. In order to obtain an analysis standing for the influence of intervention on the new cross-section of timber elements, it was assumed that the connection between new and old elements would be sufficiently reliable as not to influence the safety level. As a result, the design would only be conditioned by the resistance of timber elements.

In both interventions, new timber elements were designed with height equal to the initially existing elements. However, the width of the repaired cross-section, b_{rep}, would be determined by the design needs.

Figure 3 shows the results of the design with concern to the reliability index obtained after intervention. For the localized intervention, the repaired section width represents the sum of the width of each timber piece connected on each side of the rafter. Conservatively, the intervention design was made without considering the stress redistribution between the rafter and the other elements from the truss, as loading was not removed during repair.

By considering the target reliability indices, β_{target}, following ISO 2394 (2015) and considering moderate consequences of failure, the value of $\beta_{target} = 3.8$ would be used for the local intervention, while $\beta_{target} = 3.1$ would be used for the intervention which takes into account the replacement by a new element. These target reliability indices result in a repaired section width of 153 mm for the local intervention and of 125 mm for the replacement by a new element. These values compared to the initial design ($b_{initial}$ = 200 mm corresponding to a $\beta = 4.85$) represent a decrease of 24% and 38% on the cross-section width, respectively for the localized repair and the replacement of the element. For exemplification, $\beta_{target} = 3.3$ is also represented in Figure 3, which corresponds to the value of β_{up} proposed by Steenbergen et al. (2015) for the lifetime reference period of 30 years and the consequence class CC2. The consideration of the value β_{up} = 3.3 would be related to a repaired cross-section width of 132 mm (i.e. 34% decrease compared to the initial design).

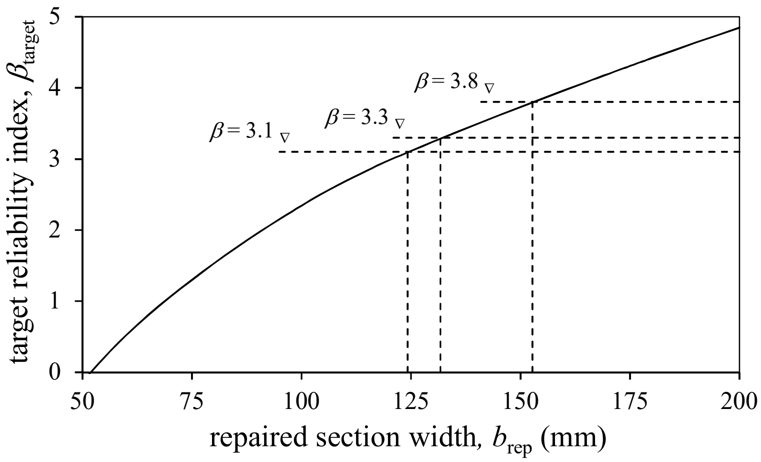


Figure 3: Reliability index according to the variation of the cross-section width from the repaired element.

4. Concluding remarks

The present work introduces key aspects of structural safety and their applicability to the assessment of existing timber structures and the design of retrofit solutions by taking into consideration the use of target reliability indices. The use of traditional methods, developed for the design of new structures, frequently results in over conservative decisions, with very high costs. Therefore the use of a more detailed methodology is clearly justified.

The definition of target reliability indices for existing timber structures, by considering both cost optimization and risk to human lives was presented in the context of existing codes and applied to a case study. The case study comprised of different possible interventions on a timber truss element and their influence on design with different target reliability indices. For this case study, it was evidenced that it is possible to lower the cross-section size by considering the differences between the design of the new structure and the design of the intervention in then existing structure. In this way, the costs of the intervention can be optimized.

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