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Overstrength in timber engineering: General discussion and proposal for a more reliable and broader application

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ABSTRACT

This review paper explores the concepts of overstrength and ductility as fundamental components of capacity-based design with a focus on timber engineering. It addresses the necessity of implementing overstrength and capacity-based design principles for effectively managing both accidental and characteristic load scenarios. Current approaches define overstrength either within the Load and Resistance Factor Design (LRFD) framework or by means of structural reliability analyses at component level, both with clear limitations in controlling the overall structural reliability and the target failure hierarchy, which is exemplified and demonstrated by a parameter study. In response, a new thematic approach for overstrength factors is proposed, formulated within the structural reliability framework, which enhances the understanding and application of ductility in design. This new methodology is benchmarked against existing approaches, therefore demonstrating its effectiveness. A clear and concise definition of ductility is emphasised as essential for a meaningful application of capacity-based design in clear conjunction with the concept of overstrength. The aim is to create a clear classification of component failure modes either as brittle (non-dissipative, non-ductile) or ductile (and dissipative, i.e. connections which provide also ductility in reverse loadings without severe pinching), which is crucial for reliably achieving a failure hierarchy where ductile failures precede brittle failures. The findings underline the importance of integrating overstrength and ductility into timber engineering practice to improve structural safety and at the same time to achieve resilient designs.

1. Introduction

The premise is to avoid unforeseen and hardly predictable sudden collapses of structures, structural components and joints. The general aim is to identify advance signs of approaching local or global overloading in the form of excessive deformation. The ability of structures, components and joints to form such non-linear deformations offers, on the one hand, the possibility of the above-mentioned warning of an approaching collapse and, on the other hand, the potential to activate less stressed components and structural areas through local stress redistributions or global load redistributions in statically indeterminate structures, which can also increase the robustness of the structure. With corresponding plastic deformation, acting energy can also be dissipated and thus the total load acting on the structure reduced (cf. [60]).

In order to selectively activate plastic deformations before brittle failures occur, it is essential to follow a clear failure hierarchy, i.e. yielding of the ductile regions, by simultaneously preventing the load-bearing capacity of the brittle regions from being reached. Such a procedure is well-known from the capacity-based design developed by Paulay and Priestley [78], among others. Such a failure hierarchy is, however, not only of great importance for accidental load scenarios, e.g. in conjunction with earthquakes, as a typical area of application, but also in general, therefore also for characteristic load scenarios.

In this context, two concepts are inseparably linked: on the one hand, the concept of overstrength design, to ensure that the intended failure hierarchy is reliably achieved, and, on the other hand, the concept of ductility, whose definition is the prerequisite for a clear distinction between brittle and ductile failure modes and, moreover, the basis for a

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classification of the extent of ductility in absolute and relative terms (see also [72,60,77]). While the basic concept of capacity-based design and the associated overstrength can be considered clear and generally accepted, looking at the diversity of discussions on its components, overstrength and ductility, there still seems to be some uncertainty, at least in timber engineering, and a need to further develop the concept of overstrength and to standardise and regulate ductility. Comprehensive discussions on ductility, on its definition, determination, classification as well as fundamental discussions on the principal performance requirements and the need for ductility are given in Jorissen & Fragiacomo [60], Malo et al. [69], Brühl [16], Ottenhaus et al. [77] and Quenneville et al. [80]. With regard to the concept of overstrength, discussed in more detail in Chapter 2, from a general perspective currently discussed approaches are clearly different and the attributable parts contributing to overstrength are very diverse and material-specific.

In this article, the concept of overstrength is discussed with a clear focus on its application in timber engineering, but in a general, broader framework than is currently the case: in addition to accidental load cases in conjunction with dynamic loadings, such as in case of earthquakes, explosions or impacts, the motivation to apply it to characteristic (permanent) load cases, corresponding to the typical use of a structure, is emphasised. After a general introduction to the topics of ductility and overstrength, the current state-of-knowledge on overstrength is deeply and comprehensively presented and discussed, with a clear focus on available definitions and factors contributing to the overstrength of connections, structural components and timber structures. Furthermore, existing regulations on overstrength are analysed comprehensively to provide a fundamental basis for the formulation and discussion of a new, alternative approach. The aims of the paper are to (i) briefly outline the inseparable concepts of ductility and overstrength in timber engineering from a general perspective, (ii) present and discuss deeply and comprehensively existing approaches to overstrength in timber engineering by highlighting their possible limitations, and (iii) motivate and present a new approach to overstrength for a consistent, reliable and economical application of the concept in timber engineering. This new approach is formulated within the structural reliability framework and in its current stage not directly applicable in daily design procedures. On the basis of comprehensive studies on various design scenarios it is intended to provide a hands-on formulation for the design within the LRFD framework in a follow-up study.

Please note: although failure modes in conjunction with timber, as a hierarchically structured natural material, should be rather classified as quasi-brittle, to increase the clarity of the language in the following "quasi-brittle failure modes" are described as "brittle failure modes" throughout. Furthermore, components displaying brittle failure are classified as non-ductile, non-dissipative, and components displaying ductile failure as dissipative, although this only applies to the specific scenario and related failure modes. To facilitate reading, variables and notations used in the cited literature are harmonised.

2. The concept of overstrength in timber engineering: Existing approaches and the impact of ductility

2.1. General remarks

The concept of overstrength is an inherent part of the concept of capacity-based design. Capacity-based design aims to provide sufficient well predictable and controllable inelastic deformations in pre-defined ductile components (zones), while limiting inelastic deformations in brittle components (zones). The aim is to obtain a structural system with a clear hierarchy of failure modes, to realise potential warning signals, maximise energy dissipation, and overall to ensure better and more reliably predictable structural behaviour [24,73]. To achieve these aims, brittle failure modes need to be prevented by explicitly considering potentially higher load-bearing capacities (overstrength or reserve strength) of ductile failure modes.

Within the Load and Resistance Factor Design (LRFD) framework, the following condition can be formulated:

$$R_{\mathrm{b,d}} \geq \gamma_{\mathrm{Rd}} R_{\mathrm{d,d}} \Leftrightarrow R_{\mathrm{b,k}} \frac{k_{\mathrm{mod}}}{\gamma_{\mathrm{M}}} \geq \gamma_{\mathrm{Rd}} R_{\mathrm{d,k}} \frac{k_{\mathrm{mod}}}{\gamma_{\mathrm{M}}} \tag{1}$$

with $R_{\mathrm{b,d}}$ and $R_{\mathrm{d,d}}$, respectively, as the design values of the brittle and ductile load-bearing capacities, $R_{\mathrm{b,k}}$ and $R_{\mathrm{d,k}}$ as the corresponding characteristic values, usually determined from experiments as 5 %-quantile values with one-sided 75 % confidence (cf. [27,52]), k_{mod} as modification factor which takes into account influences from load duration and climatic conditions on timber components, γ_{M} as material partial safety factor and γ_{Rd} as the overstrength factor; see e.g. EN 1995–1–1 [30] and EN 1998–1 [31].

2.2. Factors contributing to overstrength

The overstrength factor γ_{Rd} is usually defined as the product of a number of different contributors which are, to some extent, also material-specific; cf. Mitchell and Paultre [72] and Fajfar and Paulay [38]. Mitchell et al. [73], who provide the background for NBCC [74] on the seismic design of steel, concrete, timber and masonry structures, and Schick [86] identified the following main contributors to overstrength: (i) restricted choices in dimensions of components, in particular due to standardised cross-sections and cross-sectional shapes and geometries, in conjunction with the common rounding of dimensions in practice (geometry aspect), (ii) the difference between characteristic (nominal) and design values (safety aspect), (iii) the discrepancy between experimental and characteristic (nominal) capacities calculated from engineering models (model bias and model uncertainty aspect), (iv) additional, material-related capacities, e.g. in steel due to strain hardening (material aspect), and (v) reserve capacities in structural systems before collapse, e.g. due to load sharing and redistribution within the structural system, which are not identifiable in component analysis (system aspect; comparable to the structural system effect, cf. EN 1995–1–1 [30]).

In timber structures, ductility and energy dissipation usually need to be realised in adequately designed and executed connections which provide yielding of metal fasteners (e.g. dowels, bolts, screws, rods) and fittings (e.g. slotted-in or outer steel plates, specially profiled/shaped metal plates, angles, anchors), whereas timber components usually tend to be subject to brittle failure. There is a lot of research available on the potential for ductility and energy dissipation in different types of connections between single components (for example between beams, cf. [9,16,82,92]), between shear walls made of cross-laminated timber (CLT) (cf. [41,50,89,70]), and multiple timber components such as light-frame timber shear walls, including the interaction with floors and foundation (cf. [6,87]).

For steel, the characteristic strength properties usually represent nominal (minimum) values (cf. EN 1993–1–1 2014 [29]) and not statistical properties, as is the case for timber and concrete. Delivered steel qualities are frequently higher than ordered, a circumstance which needs to be considered in capacity-based design, especially for the lower steel grades (cf. [8,57]). According to EN 1998–1 [31], the yield strength of steel, $f_{\mathrm{y,max}}$, shall not exceed 1.1 $\gamma_{\mathrm{ov}}\, f_{\mathrm{y}}$, with γ_{ov} as "overstrength factor" recommended at $\gamma_{\mathrm{ov}} = 1.25$ (contribution (iii)), f_{y} as the nominal yield strength for a specific steel strength class, and factor 1.10 to take into account additional effects such as strain hardening and/or possible load redistribution within the component with the beginning of yielding (contribution (iv)). This simple regulation of a fixed and constant reserve capacity has some drawbacks. Gündel [49] reports on insufficient yield strength for commonly available low-grade steel, such as S235, and vice versa for higher steel grades, such as S355 and S460, when applying this regulation. He proposes a model for better alignment of steel quality and yield strengths. Another approach for steel dowels is given in Blaß and Colling [8].

In contrast to steel, for timber construction products the characteristic strength properties are usually represented by one-sided 5 %-quantile limit values determined with 75 % confidence (see [27,52]). To calculate these values, different statistical evaluation methods are allowed, e.g. parametric vs. non-parametric, frequentist vs. Bayesian approach. This allows for freedom in analysis but also raises the range in statistics for equal properties. Other contributors to overstrength in timber engineering are briefly discussed in the following. Some of them, such as the material partial safety factor, the under-utilized structural components and the conservativeness of design models, have already been listed above in a more general sense. Most of them are the result of simplifications in design procedures, production and engineering models, justified by ease-of-use and the need for conservativity. However, conservativity in predicted ductile capacities might result in reversed failure hierarchies, i.e. in brittle instead of ductile failures (cf. [77]).

- **material partial safety factor (γ_M)**: in EN 1995–1–1 [30] γ_M is regulated as dependent on the timber construction product; it is 1.30 for all strength properties in structural timber, 1.25 for glulam and CLT and 1.20 for laminated veneer lumber (LVL), whereas it is 1.30 for all failure modes in connections. Current regulations disregard individual variabilities and uncertainties of strength properties which result in additional safety in design values for strength properties subject to less variation and uncertainty, and vice versa (c.f. [62]).
- **modification factor (k_{mod})**: the modification factor k_{mod} in EN 1995–1–1 [30] considers timber's interaction with the ambient climate and its reliance on the duration of load exposure over the service life. For ease-of-use, for specific timber construction products k_{mod} is equal in service class (SC) SC1 and SC2, and lower for SC3. Similar to γ_M, k_{mod} does not distinguish between strength properties. Consequently, properties more affected by moisture and long-term loading need to be conservatively regulated. For instance, in European product and design standards, the characteristic strength value for tension perpendicular to the grain is usually much lower than calculated from experiments. Some European Technical Assessments (ETAs), such as ETA-14/0354 [35], ETA-18/1018 [36] and ETA-19/0031 [37], consider differences between SC1 and SC2 by means of service and/or load duration class-specific regulations.
- **under-utilised structural components**: in typical design processes, the exposed structural components are often overdesigned, i.e. they are not fully utilised and have a reserve capacity. In designing, engineers usually focus on the most exposed structural components, while within the same structure many other similar and equally sized components are not optimised due to simplicity in design, production, construction, maintenance and inspection. Additionally, similar to steel profiles, timber products come in discrete preferential dimensions and strength classes, again providing an additional source of safety. The potential of load sharing and redistribution among components, i.e. the structural system effect, is also often not considered.
- **size and volume effects**: size and volume effects, which describe the dependency of strength properties on geometrical dimensions and stressed volumes, are only marginally accounted for in the design and often even only in a positive sense, i.e. only when the components have smaller than reference dimensions (cf. EN 1995–1–1 [30]). Consequently, the capacity of large-dimensional components might be overestimated, whereas it might be conservative for the net cross-section capacity at connections where usually a smaller than reference volume is involved.
- **definition of characteristic values**: if determined from experiments, characteristic values of timber strength properties are usually based on the lower 75 % confidence limit of 5 %-quantile values to consider the statistical uncertainty associated with the sample. The smaller the sample and/or the larger the variability of the investigated property, the larger the additional safety added to the point estimate of the 5 %-quantile value. This fact can contribute to severely underestimated capacities.
- **timber grading methodologies**: currently it is permissible to assign structural timber in a specific strength class by means of different grading methodologies. For example, grading of structural timber to strength class C24 according to EN 338 [33] can be done visually or by machine grading, the latter even by applying different numbers and combinations of strength indicating properties. Although the characteristic properties for a specific strength class should then be the same, structural timber graded only in one strength class and/or visually usually features a much higher variability in its properties than timber graded in more than one strength class and/or by machine (cf. [14]).
- **conservative design models**: design models codified in standards or elsewhere are usually regulated on a conservative basis but without any information on model bias or model uncertainty, usually not even in the underlying documents. As an example, large discrepancies between experimental and predicted capacities are reported for groups of laterally loaded dowel type fasteners; cf. Köhler [63] and Hübner [51].
- **derivation of overstrength from experiments**: overstrength values calculated from experiments might have a very limited basis as usually not all possible failure modes can be observed, and the observed failure modes might be censored by others, which is a consequence of either different boundary conditions at testing than in real structures or due to small samples. The information on testing is limited to the experiments performed, while statistical uncertainty and uncertainty from sampling (missing or at least often limited representativeness) can become very large.
- **statistical distribution models**: the decision on a statistical distribution model for representing a random variable must be made with great care and with an awareness of the fact that it might have a significant influence on the calculated probability of failure. Evaluation standards such as EN 14358 [27] and ISO 12491 [52] provide rules for common statistical distribution models but leave the choice to the user. In contrast, PMC of JCSS [54] gives clear recommendations for statistical distribution models for a large number of action and resistance random variables.

Although contributors to overstrength such as the regulation of the modification factor, those for size and volume effects as well as uncertainties associated with the freedom in timber grading methodologies might appear irrelevant for the discussion of the overstrength factor at first sight, as this factor is commonly linked only to the ductile capacity, it will be demonstrated later in Chapter 3 that the brittle capacity and in particular its distribution characteristics and variability can also have a significant influence.

2.3. Review on current definitions for the overstrength factor

In the following, an overview of available concepts of overstrength is given, with a clear focus on timber structures. Deam [24], for example, analysed the applicability of the capacity-based design rule $R_{b,k} \geq R_{d,0.95}$ in seismic design according to NZS 4203 [76]; meanwhile replaced by NZS 1170.5 [75] which, in respect to the following discussions, can be also written as $R_{b,k} \geq \gamma_{Rd}\, R_{d,k}$, with $\gamma_{Rd} = R_{d,0.95} / R_{d,k}$. In this equation, $R_{b,k}$ represents the lower limit of the 5 %-quantile (one-sided confidence value) of the load-bearing capacity for the brittle component and $R_{d,0.95}$ the 95 %-quantile (point estimate) of the ductile component. By applying this design rule, a target failure probability of brittle before ductile components of $p_{f,Rd} = 0.04$ (permissible range 0.023–0.067) should be achieved, which aligns with a target reliability factor of $\beta_{Rd} = 1.75$ (permissible range 2.0–1.5). To analyse this, Deam [24] conducted a structural reliability analysis by modelling R_b and R_d as lognormal and/or Weibull distributed random variables with coefficients of variation {CV[R_b]; CV[R_d]} in the range of 5–50 % and by treating R_d as action and R_b as resistance. He largely confirmed the rule for likely combinations of analysed variabilities and distribution models but outlined its conservatism. He also highlighted the impact of distinctly different variabilities of R_d and R_b on $p_{f,Rd}$, which becomes largest in cases of CV[R_d] < < CV[R_b] and Weibull distributed R_d and R_b. With R_d as lognormal and R_b as Weibull distributed, CV[R_d] = 5 % and CV[R_b] ≥ 45 %, $p_{f,Rd}$ was even exceeded.

By following the principles of capacity-based design according to Pauley and Priestley [78] and with a focus on joints of laterally loaded dowel-type fasteners as possible ductile components in timber structures, Jorissen and Fragiacomo [60] defined the overstrength factor γ_{Rd} as the ratio between the 95 %-quantile and the design value of the connection's (ductile) load-bearing capacity, as

$$\gamma_{Rd} = \frac{R_{d,0.95}}{R_{d,d}} = \frac{R_{d,0.95}}{R_{d,0.05}} \frac{R_{d,0.05}}{R_{d,k}} \frac{R_{d,k}}{R_{d,d}} = \gamma_{sc}\gamma_{an}\gamma_M \tag{2}$$

with $R_{d,0.95}$, $R_{d,0.05}$, $R_{d,k}$ and $R_{d,d}$ as 95 %- and 5 %-quantile values, the characteristic value and the design value of the ductile load-bearing capacity, respectively. Here, the characteristic value $R_{d,k}$ is understood as the lower limit of the 5 %-quantile value (one-sided with 75 % confidence) predicted from theoretical, empirical or numerical models, such as the European yield model (EYM), mechanical models for beams, plates and diaphragms in EN 1995–1–1 [30], which are summarised further as engineering models, whereas the 5 %- and 95 %-quantile values are determined directly from experiments but again defined as limit values, one-sided with 75 % confidence, e.g. according to EN 14358 [27]. Inserted into Eq. (1), the design requirement according to the LRFD framework gives

$$R_{b,d} \geq \gamma_{Rd} R_{d,d} \Leftrightarrow R_{b,d} \geq R_{d,0.95} \tag{3}$$

i.e., the design value of the load-bearing capacity of the brittle component shall at least be equal to or larger than the 95 %-quantile value of the load-bearing capacity of the ductile component from experiments and determined with 75 % confidence. If no experiments are available, the common procedure is to estimate characteristic values $R_{d,k}$ or design values $R_{d,d}$ from engineering models. For the calculation of $R_{d,0.95}$ based on $R_{d,d}$, Jorissen and Fragiacomo [60] propose γ_{Rd} as the product of three partial factors γ_{sc}, γ_{an} and γ_M; see Fig. 1(a). Thereby, γ_{sc} accounts for the variability within the experimentally determined ductile capacity (statistical aspect), whereas γ_{an} expresses the model bias and model uncertainty at the level of 5 %-quantile values ($\gamma_{an} = \theta_{0.05} = x_{exp,0.05} / x_{mod,0.05}$, with $\theta = x_{exp} / x_{mod}$; model bias and model uncertainty aspect). The material partial safety factor of the ductile capacity γ_M prevents ductile failures with sufficient reliability and is equal to one in the case of ductile design according to EN 1998–1 [31] as well as in the case of accidental design situations according to EN 1995–1–1 [30] (design value aspect). If precise engineering models or specific experiments are available which can be used directly for the design, γ_{an} is equal to one, otherwise greater than or less than one.

Looking at Fig. 1(a), which covers also the concept of overstrength according to Jorissen and Fragiacomo [60], it appears that all partial factors of γ_{Rd} can be derived from statistics of one single density function f(R_d) which is only true for $\gamma_{an} \equiv 1.00$. Otherwise ($\gamma_{an} \neq 1.00$), $R_{d,k}$ is the characteristic value predicted from (stochastic-mechanical) engineering models with density function f($R_{d,mod}$), whereas the experimentally based statistics $R_{d,0.05}$ and $R_{d,0.95}$ are represented by the density function f($R_{d,exp}$). To conclude, the ductile capacity R_d might be better described by two different density functions, one related to the predicted values $R_{d,mod}$ and one to the tested values $R_{d,exp}$; see e.g. Schick et al. [88] and Fig. 1(b). Factor γ_{sc} accounts for the variability in the experimentally determined ductile capacity. Overall, apart from the variability within samples, variabilities between samples, batches, production lines, producers, etc. might have to be considered as well, cf. Fink et al. [40].

Sustersic et al. [91] define the overstrength factor as the product of two partial factors, the overdesign factor $\gamma_{od} = R_{d,d} / E_d$ and the principle overstrength factor $\gamma_{Rd} = R_{d,0.95} / R_{d,d}$. γ_{od} simply gives the inverse of the degree of utilisation between the action E and the ductile load-bearing capacity R_d at design level. γ_{Rd} is calculated directly from experiments and defined as the ratio between the 95 %-quantile value (upper, one-sided limit value with 75 % confidence) and the design value of the ductile capacity, based on the 5 %-quantile value (lower, one-sided limit value with 75 % confidence). In the case of seismic design, it is simply the ratio $R_{d,0.95} / R_{d,0.05}$ (both one-sided with 75 % confidence). In conclusion, following their definition, the overstrength factor is based solely on the ductile load-bearing capacity as determined from experiments, which corresponds to γ_{sc} according to Jorissen and

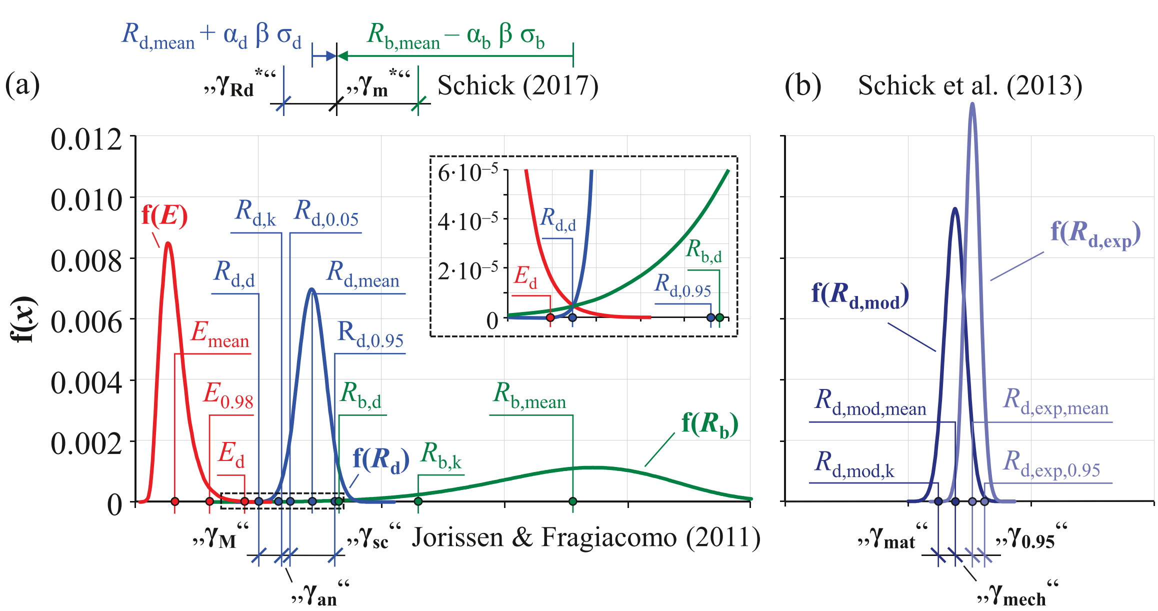

Fig. 1. Definitions of the overstrength factor and its partial factors in the context of structural reliability analysis with action E (Gumbel distribution; CoV = 35 %), ductile capacity R_d (normal distribution; CoV = 8 %) and brittle capacity R_b (Weibull distribution; CoV = 20 %): (a) definitions of Schick [86] and Jorissen & Fragiacomo [60] together with a detailed section at the design level; (b) definition of Schick et al. [88]; please note: "…" represent measures of the partial overstrength factors, originally defined as ratios and combined in a multiplicative model for calculating the overstrength factor.

Fragiacomo [60]. In the case of seismic design with $\gamma_M = 1.00$ and with $\gamma_{an} = 1.00$, both definitions are equal. The additional partial factor γ_{od} in Sustersic et al. [91] allows explicitly to account for higher than demanded ductile capacity, a circumstance which is frequently also assigned to γ_{Rd}; see e.g. Mitchell et al. [73] and Schick [86].

Schick et al. [88] analysed the overstrength factor in conjunction with timber light-frame walls and seismic design. They defined the overstrength factor as the ratio between the 95 %-quantile value of the wall capacity, as ductile component, and its characteristic capacity estimated from engineering models, i.e. $\gamma_{Rd} = R_{d,exp,0.95} / R_{d,mod,k}$. With $\gamma_M = 1.00$ in seismic design, this is essentially the same definition as in Jorissen and Fragiacomo [60]. However, although γ_{Rd} is again defined as the product of three partial factors, their individual definitions are clearly different from those in Jorissen and Fragiacomo [60]; see Fig. 1 (b) and

$$\gamma_{Rd} = \frac{R_{d,exp,0.95}}{R_{d,mod,k}} = \frac{R_{d,mod,mean}}{R_{d,mod,k}} \frac{R_{d,exp,mean}}{R_{d,mod,mean}} \frac{R_{d,exp,0.95}}{R_{d,exp,mean}} = \gamma_{mat}\gamma_{mech}\gamma_{0.95} \tag{4}$$

The main difference is the explicit separation into two distribution functions which together represent the load-bearing capacity of the ductile component: one for the engineering model with density function $f(R_{d,mod})$ and one for the experiments with density function $f(R_{d,exp})$. As in Jorissen and Fragiacomo [60], γ_{Rd} depends only on the uncertainty and variability of the ductile load-bearing capacity; there is no interaction with the brittle capacity nor between the brittle capacity and the action. Schick et al. [88] define γ_{mat} as the ratio between the average and the characteristic value of the predicted ductile capacity, where the average value is calculated by means of the same model by substituting the characteristic values with average values. The partial factor γ_{mech} is associated with mechanical effects, such as the influence of friction, as well as with general conservatism in design rules and differences between nominal (ordered) and delivered qualities, as is often the case for steel. Thus, this partial factor is simply the ratio between the average values from experiments and the engineering model, i.e. the model bias. Finally, the partial factor $\gamma_{0.95}$ is the ratio between the upper limit of the 95 %-quantile value from experiments (determined one-sided with 75 % confidence) and the average value. The explicit division of the ductile capacity into two distributions, one for the predictions and one for the experiments, allows a clear definition of all measures contributing to the overstrength factor γ_{Rd}, the model bias, model uncertainties and additional variabilities coming from, for example, differences between samples, batches and producers.

Brühl et al. [18], and later Brühl and Kuhlmann [17] and Brühl [16], deduced overstrength factors from structural reliability analyses with limit state function $g(\boldsymbol{X}) = R_b - \gamma_{Rd} R_d$, similar to Dean [24] with the capacity of the ductile component as action and the capacity of the brittle component as resistance. They underline the necessity to consider the variabilities in both capacities R_d and R_b as well as the influence of possible interactions (correlations) between material properties contributing to them. The interaction of the brittle capacity with loads is, however, not considered. According to Brühl [16], the safety margin between R_d and R_b should be linked with the consequences of failure and possibly regulated in conjunction with the consequence classes in EN 1990 [28].

Another definition of overstrength is given in Schick [86], which is also discussed in Seim and Schick [90]. This definition was later applied also in Aloisio et al. [2,1], who focused on separating bias and uncertainty in estimating the ductile resistance in aleatoric and epistemic, demonstrated on CLT-to-CLT screwed joints. The definition of Schick [86] is again based on a structural reliability formulation with the ductile capacity as action and the brittle capacity as resistance, both treated as random variables and arranged at sufficient distance to secure a predefined safety level. In reference to NZS 4203 [76], the probability of brittle before ductile failure is set at $p_{f,Rd} = 0.04$ ($\beta_{Rd} = 1.75$). With this information and the corresponding distribution models and parameters for R_d and R_b, the design point is calculated as

$$\begin{aligned} &R_{b,d} = R_{d,d} \Leftrightarrow R_{b,mean} - \alpha_b\beta_{Rd}\sigma_b = R_{d,mean} + \alpha_b\beta_{Rd}\sigma_d, \\ &\text{with} \quad g(\boldsymbol{X}) = R_b - R_d = 0 \quad \text{and} \quad p_{f,Rd} = P(g(\boldsymbol{X}) \leq 0) \end{aligned} \tag{5}$$

with $g(\boldsymbol{X})$ as limit state function, α_b and α_d as sensitivity factors and σ_b and σ_d as standard deviations, respectively, for the brittle and ductile load-bearing capacities. The statistics $R_{b,mean}$, $R_{d,mean}$, σ_b and σ_d are computed from MC simulations and the sensitivity factors α_b and α_d from FORM analyses according to the PMC from JCSS [55]. Based on the design point and the characteristic values $R_{b,k}$ and $R_{d,k}$, which are predicted from engineering models, the overstrength factor γ_{Rd} is defined as the product of two partial factors γ^*_m and γ^*_{Rd} and defined as

$$\gamma_{Rd} = \gamma^*_{Rd}\gamma^*_m = \frac{R_{d,mean} + \alpha_d\beta_{Rd}\sigma_d}{R_{d,k}} \frac{R_{b,k}}{R_{b,mean} - \alpha_b\beta_{Rd}\sigma_b} \tag{6}$$

As the focus is again on seismic design, $\gamma_M = 1.00$ applies.

To summarise the review on presented overstrength definitions, so far Mitchell et al. [73], Jorissen & Fragiacomo [60], Sustersic et al. [91] and Schick et al. [88] refer to the LRFD framework and only to the ductile load-bearing capacity. Any interactions of R_d and R_b or R_b and E remain unconsidered. The so defined overstrength factor merely needs to provoke an overdesign of the brittle components (brittle failure modes) by accounting for the variability as well as the model bias and model uncertainty in predicting the load-bearing capacity of the ductile components (ductile failure modes), by defining it as a ratio between a meaningful upper boundary of possible experimental values, here the 95 %-quantile value, and the characteristic value of the ductile capacity as used in the design, so that a brittle failure prior to a ductile failure becomes unlikely. As exemplarily highlighted in the detailed section in Fig. 1(a) (see also Fig. 3(a)), simply ranking quantile values might not always be sufficient, neither from an economical nor from a reliability point of view. It is not only the probability mass which accumulates below or above limit values, such as characteristic and design values, but rather the distribution of the probability mass, characterisable by skewness and kurtosis as measures of the shape of statistical distribution models selected to represent the random variables.

Detached from the LRFD framework, Deam [24], Brühl et al. [18] and Schick [86] deduced the overstrength factor from structural reliability analyses by treating the ductile load-bearing capacity as action and that of the brittle component as resistance. However, the interaction of R_b and E again remains unconsidered.

Overall, the analysed overstrength concepts are usually formulated for specific design cases and limit states. What is still missing is an operational definition of overstrength which allows to apply the concept on a more general basis. The new approach, formulated in Section 3.1, has the potential to fill this gap.

2.4. Current status of the definition of ductility

The successful application of the concept of capacity-based design is conditioned on a clear classification of brittle and ductile failure modes and thus a clear definition and classification of ductility, something which is still under debate in timber engineering; cf. Jorissen & Fragiacomo [60], Malo et al. [69], Brühl [16], Ottenhaus et al. [77] and Quenneville et al. [80]. Jorissen & Fragiacomo [60] outline the advantages of ductility in timber constructions and present a comprehensive summary of absolute and relative definitions of ductility. Malo et al. [69] quantify ductility by means of analytical expressions with the advantages of less uncertainty in ductility measures compared with measures directly determined from experiments and of a better representation of the load-displacement curve with the potential to implement such information directly in numerical tools. They also propose ductility measures, with an explicit differentiation between measures for static and dynamic loading. In this context, it might also be of interest to look at recent analyses addressing overstrength and ductility in timber engineering. Jockwer et al. [57] analyse the failure behaviour and reliability of joints with laterally loaded dowel-type fasteners and report that ductility determined from experiments on single fasteners might be different to ductility determined from experiments on joints. Based on structural reliability analyses, they clearly illustrate that the joint's load-bearing capacity is always a mixture of capacities from different failure modes whose proportions vary depending on the material properties as well as the geometric and other boundary conditions. In this context, a distinction between ductile and brittle components is not expedient as components and especially joints and connections can fail in different ways. Instead, classification is required in ductile and brittle failure modes, which again necessitates meaningful measures for ductility and energy dissipation. Cabrero et al. [19] also conducted structural reliability analyses for joints with laterally loaded dowel-type fasteners. They found that even in cases where the characteristic or design value of the ductile load-bearing capacity is lower than that of the brittle failure mode, as commonly applied in LRFD, due to the often higher variability of brittle capacity, a brittle failure might be even more likely. Because of this, they call for a brittle overstrength factor to minimise the risk of brittle instead of the desired ductile failure. Brühl [16] summarises ductility measures and findings for a large number of different classes of timber joints and adds his own experiments on tension and moment joints. For a better classification of joints, he outlines the need to combine absolute and relative ductility measures. Ottenhaus et al. [77] focus on connections as potential ductile elements (PDEs) in timber structures by treating ductility as the ability to (largely) resist a certain load even at increasing deformations. They propose a performance-based definition and treatment of ductility without the need for a yield point but instead explicit knowledge of post-peak behaviour. Quenneville et al. [80] give recommendations on how to increase ductility in connections and outline the need to differentiate between ductility as observed in monotonic and cyclic tests. This is because inelastic deformation in steel components can be reversed by reversing the loading direction, whereas such deformation in timber, resulting from crushing of fibres surrounding the fastener, is irreversible and leads to severe pinching in reversed loading (see also [89] and [77]).

At the European level, the definition and determination of ductility and energy dissipation are regulated in EN 12512 [26]. The standard for cyclic tests on joints defines ductility $D = v_u / v_y$ as a relative measure, with v_u as ultimate displacement (either at the maximum load F_{max}, below 30 mm displacement as post-peak displacement at 80 % of F_{max}, or as 30 mm displacement, whichever occurs first) and v_y as yield displacement (displacement at the intersection between the secant stiffness, between 10 % and 40 % of F_{max}, and a tangent stiffness in the yielding part of the load-displacement curve). For monotonic testing of joints, EN 26891 [32] defines F_{max} as the maximum load until 15 mm displacement, whereas for monotonic testing of single, laterally loaded fasteners EN 383 [34] defines F_{max} as the maximum load until 5 mm displacement. Nevertheless, very stiff joints, such as axially loaded groups of screws, glued-in rods or glued-in perforated steel plates, can yield medium to high relative ductility measures although their absolute displacements are only a few millimetres (cf. [41]).

To conclude, current procedures for determining ductility are in general quite different; they are scale-dependent, i.e. different for single fasteners and joints, and overall fail to achieve harmonisation. Fixed limits in experimentally determined load-displacement curves prevent complete information on post-peak or even peak behaviour, and the generally monotonic testing of fasteners and joints itself should be reconsidered; characterising fasteners and joints based on monotonic and cyclic experiments is recommended and seen as a necessity for the classification of ductility and energy dissipation. Evidently, there is an urgent demand for harmonising fasteners and joints for timber structures and for a performance-based definition and treatment of ductility. A clear classification in ductile and brittle failure modes is a prerequisite for a reliable application of capacity-based design and for a successful realisation of the intended failure hierarchy.

3. Definition of a new concept of overstrength and its benchmark with current approaches

3.1. Formulation of a new concept of overstrength

According to Chapter 2, there are currently in principle two approaches in literature defining overstrength in timber engineering: Approach I based on the LRFD framework (essentially following [73,60,91,88]) and Approach II on a structural reliability analyses at component level with ductile capacity as action and brittle capacity as resistance (essentially following Dean 1996; [86,16]). Both approaches are conditioned on ductile capacity R_d vs. action E fulfilling the target failure probability $p_{f,0}$, which is defined by the limit state function $g(\boldsymbol{X}) = (R_d / E) = 1.00$, with $P(g(\boldsymbol{X}) \leq 1.00) = P(R_d / E \leq 1.00) = p_{f,d} \leq p_{f,0}$, with $p_{f,d}$ as failure probability of the ductile component and with $p_{f,0} = 10^{-6}$ as nominal failure probability for characteristic load cases in the ultimate limit state (ULS) design, consequence class CC2 and one-year reference period according to EN 1990 (2010) [28].

Approach I defines the overstrength factor as $\gamma_{Rd} = R_{d,0.95} / R_{d,d}$, with $R_{d,0.95}$ as 95 %-quantile value of the ductile capacity calculated from experiments and $R_{d,d}$ as design value of the ductile capacity usually predicted from engineering models. Hereby it is conditioned that $R_{d,d} \geq E_d$, with E_d as design value of action, and with $R_{b,d} \geq \gamma_{Rd} R_{d,d}$, with $R_{b,d}$ as design value of brittle capacity.

Conditioning that $p_{f,d} \leq p_{f,0}$, in Approach II a second sub-limit state function is defined with R_d as action and R_b as resistance, ensuring that $P(R_b \leq R_d) \leq p_{f,Rd}$, with $p_{f,Rd}$ as probability to observe brittle before ductile failure. The additional safety requested for the brittle capacity, expressed via $p_{f,Rd}$, is not yet defined at European level and subject to further analysis and left to public authorities and standardisation bodies; as an example, NZS 4203 [76] recommends $p_{f,Rd} = 0.04$ and Brühl [16] proposes its regulation in conjunction with consequence classes; cf. EN 1990 (2010) [28]. Although the entire distribution characteristics of R_b and R_d are taken into account, by applying this approach it is not ensured that the brittle capacity is sufficient to overcome $p_{f,0}$. Within the parameter study presented in Section 3.2, $p_{f,Rd} = \{10^{-1}; 10^{-2}\}$ apply.

To summarise, whereas Approach I considers only the variability as well as the model bias and uncertainty associated with the ductile load-bearing capacity, Approach II considers the entire distribution characteristics of R_d and R_b. However, both definitions do not consider the interaction of brittle capacity with action, as for the resistance part only the ductile capacity is aligned with the acting loads and moments.

The identified gap is covered by the new Approach III for defining overstrength which presents a logical consequence of system reliability analysis. This approach, although deduced from structural reliability analyses at system level, is still formulated at component level. Conditioning that $p_{f,d} \leq p_{f,0}$ is ensured by $g(\boldsymbol{X_1}) = (R_d / E) = 1.00$, with $P(g(\boldsymbol{X_1}) \leq 1.00) = P(R_d / E \leq 1.00) = p_{f,d} \leq p_{f,0}$, a second limit state function $g(\boldsymbol{X_2}) = (R_b / E) = 1.00$ is defined, with $P(g(\boldsymbol{X_2}) \leq 1.00) = P(R_b / E \leq 1.00) \leq p_{f,d}\, p_{f,Rd}$. In doing so, the failure probability for the brittle component $p_{f,b}$ has to fulfil $p_{f,b} \leq p_{f,d}\, p_{f,Rd}$, or in the limit case with $p_{f,d} = p_{f,0}$, the additional safety for brittle capacity is directly multiplied by the target probability required for the ductile capacity. With $p_{f,0} = 10^{-6}$ and $p_{f,Rd} = \{10^{-1}; 10^{-2}\}$, for example, the probability of brittle failures would become $p_{f,b} \leq \{10^{-7}; 10^{-8}\}$, as it is applied in the parameter study in Section 3.2 hereafter.

This new approach is a logical consequence of system reliability analyses on the serial chain of ductile and brittle failure modes. Piluso et al. [79] and Maglio et al. [66], for example, conducted parameter studies analysing the reliability of moment-resisting steel frames following the principles of the theory of plastic mechanism control in a probabilistic setting. By defining the global failure as the design target which has to be reached, e.g. in 95 % of the cases before other local failures, overstrength factors were calculated for different variabilities of yield strength as measures of aleatoric uncertainty. However, such analyses are scarce and the authors are not aware of comparable studies in timber engineering which, in contrast to steel structures, have to cope with much larger epistemic and aleatoric uncertainties.

3.2. Parameter study and benchmark of the new Approach III with current approaches

The applicability and adequacy of the three approaches is analysed by a parameter study. The influences from statistical distribution models and variabilities, expressed via the coefficient of variation (CoV[X]), are analysed, with the corresponding expected value of action E, E[E] being fixed and those of resistances, E[R_b] and E[R_d] being set according to the boundary conditions, which are given as: in Case (i), action E and resistances R_b and R_d are assumed as lognormal distributed; in case (ii), action E, as usual, is represented by a Gumbel distribution (extreme value type-I; maxima), R_b by a Weibull and R_d by a normal distribution (see e.g. [54]; EN 1990 2010 [28,53]). The exemplary distributions were determined analytically for Case (i), whereas for case (ii) the distributions were found via an iterative numerical approach. In case (ii) the distributions for ductile resistance R_d were preselected, while the distributions for action E and brittle resistance R_b were aligned to R_d to meet the predefined target reliability criteria. The reliability indexes were calculated by First Order Reliability Method (FORM) and normal tail approximation.

The influence of parameters is examined by means of structural reliability analyses at component level, which means that there is only one action (one exposure scenario) and one resistance, either ductile or brittle (as discussed later, typical ULS scenarios usually feature a number of different actions and resistances which then require a system reliability analysis). This applies not only for Approaches I and II but also for Approach III, where the structural reliability analysis is performed separately for the ductile and brittle capacities as resistances. In doing so, it is also assumed that action and resistances are independent random variables. Whereas this is common but not always given for action and resistance, the ductile and brittle capacities might be correlated, for example in steel-timber connections where both capacities might depend on the properties of timber as anchoring material. With a clear focus on the essence of observations and conclusions from the parameter study, $\gamma_M = 1.0$ applies, as is also common in seismic design. It is assumed that either the models predicting the capacities are perfect or that the model uncertainty is already part of the variabilities of R_b and R_d and that any model bias is also adequately accounted for. Based on these assumptions, the overstrength factor as currently applied within the LRFD framework reduces to $\gamma_{Rd} \equiv \gamma_{sc} \equiv R_{d,0.95}$ / $R_{d,0.05}$ which, by shifting the distribution of R_b as close as possible to that of R_d, further simplifies to $\gamma_{Rd} = R_{b,0.05}$ / $R_{d,0.05}$. Furthermore, the coefficient of variation of action is set with CoV[E] = 30 %, the expected value E[E], in this case as an arbitrary measure, is fixed in Case (i), and in case (ii) it is adjusted accordingly to an arbitrary but fixed E[R_d]. It is analysed how the variation of CoV[R_b] and CoV[R_d] influences the probability of failure of both R_b and R_d, their reliability indices β_b and β_d as well as the overstrength factor γ_{Rd}.

The main outcomes are shown in Fig. 2, with some important conclusions summarised below:

3.2.1. Case (i) – {E; R_d; R_b} ~ Lognormal (LN) distributed

- Looking at Fig. 2(a), of the three approaches investigated Approach III is the only one which maintains a constant ratio between the reliability of brittle and ductile capacity (expressed here as a ratio between the corresponding reliability indices β_b and β_d).
- For similar values of CoV[R_b] and CoV[R_d], Approach I or II lead to higher than demanded reliabilities, synonymous with conservative and uneconomical designs, while combinations with CoV[R_b] > > CoV[R_d] can result in reliabilities below the target value and structural behaviour that does not meet the target specifications.
- Overstrength factors for CoV[R_b] = 30 % and CoV[R_d] = 5 % according to Approach I lead to comparable reliability indices β_b and β_d, which means equally likely brittle and ductile failures.
- The overstrength factor γ_{Rd}, which converts the results from the structural reliability analysis to the LRFD framework, is summarised in Fig. 2(b). According to Approach I, γ_{Rd} only depends on CoV[R_d], i.e. it remains constant and independent of CoV[R_b]. According to

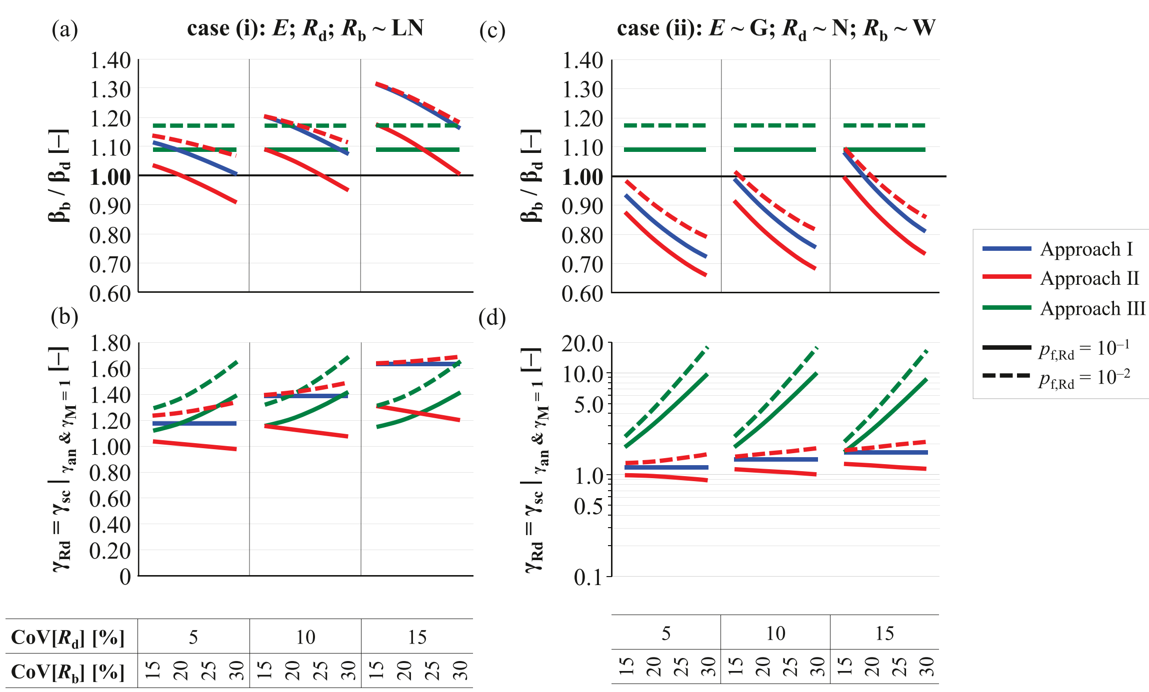

Fig. 2. Structural reliability analyses with $p_{f,0} = p_{f,d} = 10^{-6}$ and $p_{f,Rd} = \{10^{-1}; 10^{-2}\}$, following Approach I, II & III of overstrength definitions, for various combinations of CoV[R_b] and CoV[R_d] and statistical distribution models: (a; b) Case (i); (c; d) case (ii); (a; c) ratio between the reliability indices for brittle and ductile failures; (b; d) overstrength factor $\gamma_{Rd} = R_{b,0.05}$ / $R_{d,0.05}$ (please note: ordinates are scaled differently, (d) in logarithmic).

Approach II, at $p_{f,Rd} = 10^{-1}$ γ_{Rd} slightly decreases with increasing CoV[R_b], whereas it slightly increases at $p_{f,Rd} = 10^{-2}$. For Approach III, the course of γ_{Rd} is partly even inverse to Approaches I and II and clearly dependent on both CoV[R_b] and CoV[R_d].
- Overall, the differences between Approach I and II are small compared with Approach III. In fact, Approach III is the only one which gives consistent overstrength factors by maintaining the target reliability levels. In comparison with case (ii), in Case (i) the overstrength factors for $p_{f,Rd} = 10^{-2}$ are also only slightly higher than for $p_{f,Rd} = 10^{-1}$.

3.2.2. Case (ii) – E ~ Gumbel (G); R_d ~ Normal (N); R_b ~ Weibull (W) distributed

- According to Fig. 2(c), Approach III is again the only one which maintains a constant ratio between β_b and β_d regardless of CoV[R_b] and CoV[R_d].
- Following Approach I and II, with the exception of CoV[R_b] = CoV [R_d] all other combinations result in $\beta_b \leq \beta_d$, mostly in $\beta_b << \beta_d$; consequently, the intended failure hierarchy, ductile before brittle, cannot be achieved. The reliability index β_b decreases relative to β_d as the ratio CoV[R_b] / CoV[R_d] increases.
- For Approach III, the calculation of γ_{Rd} reveals remarkable differences in cases when CoV[R_b] is much larger than CoV[R_d], which is reflected both in absolute values of γ_{Rd} as well as relative to the outcomes from Approach I or II. For example, the combination {CoV [R_d] = 5 %; CoV[R_b] = 30 %} results in $\gamma_{Rd} > 10$ for $p_{f,Rd} = 10^{-1}$ and values close to 20 for $p_{f,Rd} = 10^{-2}$, while a combination with a lower CoV[R_b] = 15 % yields more than five times lower $\gamma_{Rd} < 3$. At the same time, according to Approach I and II, γ_{Rd} is always less than two even at $p_{f,Rd} = 10^{-2}$ and regardless of the CoV[R_b] and CoV[R_d] values.
- It is concluded that the differences between Approach I and II compared with Approach III are much bigger than in Case (i), proving that the results are clearly dependent on the statistical distribution models selected to represent E, R_d and R_b in addition to the dependency on the variabilities. The effect of the statistical distribution model becomes higher for higher values of CoV[R_b] and CoV [R_d].

The theoretical differences between the Approaches I, II and III must first be fitted into the discussion between the differences in results obtained for Cases (i) and (ii). The implementation of Approach III appears beneficial on the reliability scale both in Cases (i) and (ii). The inconsistencies between the Approaches I and II compared with Approach III are larger in case (ii), meaning that the selection of the statistical distribution model could have a significant impact on the overstrength factor which dictates the additional resistance demanded from brittle capacity or, in other terms, the under-utilisation of the brittle component. A discussion on statistical distribution models which best cover the behaviour of timber construction products, joints and connections in timber structures in realistic scenarios is beyond the scope of this paper, however, some additional information on the topic is provided in Chapter 4. Nevertheless, it can be argued that case (ii) reflects a possible scenario in timber structures, as evidenced by the frequent selection of Weibull distribution for brittle failures (cf. [5,42,43,21]; PMC of [54]; EN 1990 2010 [28,53]) and normal distribution for components displaying ductile failure whose capacities are characterised by internal stress and/or external load redistribution in the sense of averaging effects; for thorough discussions on the pros and cons of statistical distribution models typically used for representing properties of timber as quasi-brittle material, cf. Brandner [11].

Furthermore, on the reliability scale the inconsistency in results in both analysed cases between Approach I and II compared with Approach III become greatest when the difference between CoV[R_b] and CoV[R_d] is greatest. Despite this theoretically sound outcome it needs to be investigated which values and combinations of CoV[R_b] and CoV[R_d] are practically possible and expectable, which is done by a literature review on experimental investigations of brittle and ductile failure modes with a focus on different types of timber joints and connections in Chapter 4.

An additional aspect which needs to be briefly discussed is the circumstance that real limit states can be seldom formulated at component level but rather need to be dealt with in the context of system reliability analyses. This is because common load and resistance scenarios usually involve more than one action and one resistance (failure mode) random variable. The type of interaction and correlation between the limit state functions of the components also need to be considered. The interaction can be categorised as either serial, parallel or as serial, sub-parallel. For example, if a timber-steel-timber connection (e.g. a connection with a slotted-in steel plate in the centre of the timber component anchored by laterally loaded dowels) is loaded, a number of different failure modes can occur, each with a certain probability. Possible failure modes in the timber component may include embedment failure, net cross-section failure, as well as block, row and plug shear and splitting. In the steel components, the dowels can shear off due to MNV interaction or yield in bending, and apart from other factors, the slotted-in steel plate can also fail in tension. These failure modes need to be classified either as brittle or ductile. In terms of structural reliability, brittle failure modes are usually associated with a chain-like behaviour (serial system), e.g. according to the “weakest link principle” (decreasing capacity with increasing number of brittle failure modes; usually represented by the Weibull distribution model), while ductile failure modes are usually associated with an averaging effect in conjunction with yielding, which enables redistribution of stresses (internal; local) and loads (external; global) by maintaining (largely) the capacity of single components times the number of components, which is associated with a parallel system behaviour, usually represented by the normal distribution model. For additional and more detailed information, cf. Weibull [93], Daniels [23], Gollwitzer [48], Melchers [71] and Brandner [11]. To conclude, also from this point of view, the distinction between brittle and ductile failure modes is of crucial importance, especially when a certain failure hierarchy needs to be ensured.

4. Probabilistic representation and review of variabilities of brittle and ductile capacities

4.1. General remarks and follow-up on the influence of statistical distribution models

The properties of structural timber are influenced by natural growth conditions and by the overall production process, i.e. they are subject to remarkable material- and process-related variabilities. Particularly, the variabilities of density as well as of elastic and strength properties, have a great influence on the response to externally applied loads and moments (actions), not only within a certain section (e.g. connection, supports, etc.) but also within a complete structural system. Therefore, the variability in the properties of timber has a direct influence on the specific load-bearing and deformation capacity of components and connections within a structure. The building process additionally contributes to this variability. These circumstances need to be considered to effectively achieve the target failure hierarchy at each structural level, from the level of single components and joints via sub-structures, such as frameworks, up to the level of the entire load-bearing system. It needs to be ensured that in specific ULS design scenarios, sufficient ductility and energy dissipation within timber structures is achievable with anticipated reliability, for example by applying elastic-plastic design. The effects of variability can be explicitly considered within a probabilistic framework based on statistics gained from experiments at all hierarchical levels of timber structures.

As previously described, according to Approach I the effect of variability on the load-bearing capacity of the ductile component, expressed by the overstrength partial factor γ_{sc}, is covered by the ratio $R_{d,0.95}$ / $R_{d,0.05}$. The ratio itself is influenced by CoV[R_d] and by the statistical distribution model representing R_d. Fig. 3(b) analyses the ratio for CoV[X] up to 40 % and for the three statistical distribution models: normal, lognormal and Weibull. The results show that γ_{sc} can already reach very high values at moderate CoV[X] and that for higher CoV[X] the effect of the statistical distribution model becomes more pronounced. For CoV[X] < 20 %, γ_{sc} is below 2.0 and the influence of the statistical distribution model becomes negligible. It is worth mentioning that load-bearing capacities of ductile components will rarely exceed a CoV[R_d] of 20 % (see the review in Table 2). Therefore, for the capacity of ductile components, the selection of the statistical distribution model would not be crucial for the accuracy of Approach I and II, as also confirmed by looking at Fig. 2(b; d). High variability in ductile capacity would rather indicate the need for replacement or improvement in order to avoid high overstrength factors and uneconomical and inefficient design.

As demonstrated in Chapter 3, apart from the distribution and variability of R_d, the distribution and variability of R_b is also decisive for a reliable representation of the overstrength factor, as specific pairs of CoV[R_b] and CoV[R_d] can lead to either uneconomical design (Case (i): CoV[R_d] > 10 %) or design on the unsafe side (case (ii): CoV[R_b] $>$ CoV[R_d]) when following Approach I or II. Furthermore, looking at case (ii) in Chapter 3 the shape of the lower tail of the Weibull distribution affects the structural reliability more than the lognormal distribution in Case (i), and together with the right-skewed Gumbel distribution results in obviously unsafe structures in the case of Approach I and II. Because of the dependency of the overstrength factor on the statistical distribution model, and also from a general point of view, it is necessary and highly advisable to standardise the statistical distribution models to be used for the probabilistic characterisation of random variables such as actions and resistances in order to increase transparency and comparability of analyses. Such standardisation needs to be done with great care, based on a sound and representative database, by considering physical boundary conditions and underlying phenomena as well as comprehensive studies analysing the impact of the choice from different perspectives, therefore also for calculation of the overstrength factor.

4.2. Review on variabilities in ductile and brittle capacities

Apart from the statistical distribution model, it is the variability of the properties which also needs to be set in a representative range for each individual property. In order to demonstrate the possible and expectable values for CoV[R_d] and CoV[R_b] in timber structures, a literature review was conducted (see Table 1 and Table 2). It summarises only some of the research in the last twenty-five years with a focus on the range of CoV[X] possible to obtain for timber connections and joints. Although there was much more research on timber connections and joints in the past, the focus is on the up-to-date research due to comparable and contemporary test methods used in these experimental campaigns. It is also clear that timber construction products such as glulam, CLT and LVL as well as steel connectors have evolved in recent years due to improved structural adhesives, better factory production control and other factors. It is therefore assumed that the CoV[X] values shown in Table 1 and Table 2 could be higher when applied to timber structures, timber construction products and connections and joints used in the past. The review was based on timber joints and connections with (primary) laterally and axially loaded dowel-type fasteners. Moment-resistant connections (e.g. rotationally loaded connections) were not included due to the complexity of load transfer on each fastener and difficulties in distinguishing between ductile and brittle failure modes.

From the variabilities for ductile and brittle capacities as summarised in Table 1 and Table 2 the following conclusions are made: uncertainties caused by sampling, testing, data processing (decision on governing failure mode, allocation of specimen to failure classes, decision on valid/invalid specimens, etc.) and statistical evaluation (limited sample, parametric/non-parametric evaluation, etc.) are part of the reported variabilities, but their share is usually unknown. This might explain, at least to some extent, the heterogeneity in reported outcomes. Nevertheless, some trends can be synthesised, such as the circumstance that capacities of fasteners penetrating several layers/laminations are less variable. Furthermore, CoV[R_b] is within 5 and 30 % and it can be expected that the value is usually dominated by the variability in the timber construction product properties, whereas CoV[R_d] is within 1 and 20 %, in most cases between 3 and 8 %, and usually dominated by the variability of the properties of the component displaying ductile failure, commonly steel. Furthermore, by comparing the outcome of series which display either ductile or brittle failures, the ratio CoV[R_b] / CoV[R_d] is at least two or even greater.

Before discussing variabilities related to joints and connections, variabilities for strength properties of structural timber are briefly summarised as these are usually of relevance for the brittle failure modes in joints and the basic values of timber construction products manufactured using them, such as glulam and CLT, whose properties are less variable because of stress-sharing within the rigid composites of bonded structural timber, cf. Colling [22], JCSS [54], Brandner [11], Brandner et al. [13] and Fink [39]. According to the PMC of JCSS [54] for the strength properties of structural coniferous timber, the following coefficients of variation are given: 30 % for tension parallel to the grain, 25 % for bending, tension perpendicular to the grain and shear, 20 % for compression strength parallel to the grain and 10 % for compression strength perpendicular to the grain. These values can be considered as expected on average at that time. Depending on the classification of the base material in the course of strength grading, the expectable variabilities can vary considerably, as shown for tensile strength parallel to

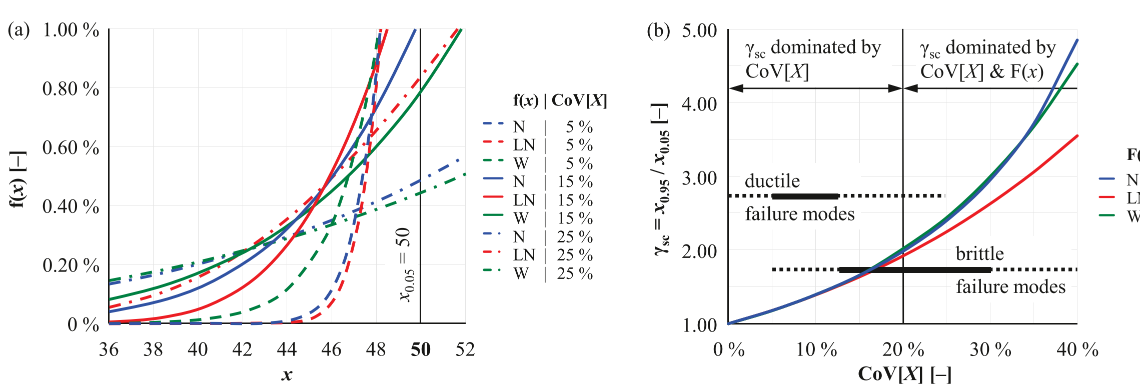

Fig. 3. (a) influence of the statistical distribution model and CoV[X] on the distribution of the probability mass in the lower tail area preconditioning equal 5 %-quantile values $x_{0.05} = 50$; (b) quantile ratios $x_{0.95}$ / $x_{0.05}$ for normal (N), lognormal (LN) and Weibull (W) distribution models depending on CoV[X], together with typical (expectable) ranges of CoV[X] in the respective failure mode classes.

Table 1
Review of experimental investigations on connections with dowel-type fasteners displaying (primarily) brittle failure, with a focus on the observed range of CoV[R_b].

main fastener exposition	connection type	base material	reference	failure modes	N_s	N_f	CoV [R_b] [%]
laterally loaded fasteners	bolted steel-timber-steel connections	structural timber	Quenneville and Mohammad [81]	block shear; plug shear; row shear; splitting	10	1–8	6–24
		glulam			10	1–8	5–24
	nailed timber connections	glulam	Johnsson [58]	plug shear	3–20	10–276	7–25
	bolted steel-timber-steel connections	glulam	Dodson [25]	block shear; row shear; splitting	5	4–6	11–21
	doweled steel-to-timber connections	glulam	Chun and Niederwesterberg [20]	row shear	3–4	6–16	3–8
	connections with rivets	glulam	Zarnani and Quenneville [96]	plug shear	4	24–64	11–17*
		LVL			3	16–64	4–9
	nailed & screwed timber connections	glulam	Yurrita and Cabrero [94]	plug shear	3	15	2–15
		LVL			3	15	2–11
	doweled steel-to-timber connections	glulam	Yurrita et al. [95]	block shear; row shear; net-tension	4	9	2–28
		LVL			4	9	2–12
	screwed connections	CLT	Azinović et al. [3]	plug shear; row shear; net-tension	3–6	60	4–29
	screwed connections	CLT	Chun and Niederwesterberg [20]	plug shear (step shear)	3	9–15	4–16
axially loaded fasteners	timber-to-timber connections with inclined / cross-wise arranged screws	structural timber	Kevarinmäki [61]	withdrawal	5	2–8	5–11
			Blaß et al. [7]		5	2–11	2–8
	steel-to-timber connections with inclined / cross-wise arranged screws	glulam	Brandl [10]	screw-tension; head tear-off; net-section; splitting; withdrawal	22	8	4–10
		glulam	Krenn [65]		5–30	2–8	3–11**
	glued-in rods connections	CLT	Azinović et al. [4]	withdrawal; rolling shear; block shear	5	1	7–25
	connections with screws inserted perp. / at an angle to grain	structural timber; glulam; LVL	Mahlknecht and Brandner [68]	screw-tension; block shear; withdrawal	2–11	4–25	3–15
			Mahlknecht et al. [67]		2–21	3–19	3–30**

N_s … number of specimens per sample; N_f … number of fasteners per connection
* for some samples, only ductile failures reported – variability of ductile failures lower than listed in the table and below 8 %.
** screw-tensile & head tear-off appear ductile but feature only a few millimetres deformation at F_{max}, so they are rather rated as brittle but with some potential to (re-)distribute loads within groups of axially loaded screws.

Table 2
Review of experimental investigations on connections with dowel-type fasteners displaying (primarily) ductile failures, with a focus on the observed range of CoV[R_d].

main fastener exposition	connection type	base material	reference	description of tests	N_s	N_f	CoV [R_d] [%]
laterally loaded fasteners	dowelled timber-to-timber connections	structural timber	Jorissen & Fragiacomo [60]	monotonic shear, parallel to grain	10–25	1–9	3–22
	nailed & stapled connections	light-frame timber walls (structural timber and OSB + GFB)	Schick et al. [88]	monotonic shear cyclic shear	4–7 1–4	14–17	4–18
	screwed timber-to-timber connections	CLT	Fragiacomo et al. [44]	cyclic shear	3–5	10	4–13
	screwed connections	CLT	Gavric et al. [45]	cyclic shear	6	2–8	5–17
	nailed connections with hold-downs	CLT	Gavric et al. [47]	cyclic uplift (tension)	6	9–12	4–8*
	nailed connections with angle brackets	CLT	Gavric et al. [47]	cyclic shear	6	8–11	3–7*
	dowelled timber-steel-timber connections	glulam	Brühl [16]	monotonic tension	3–4	6–10	3–8
	screwed (inclined / cross-wise) timber-to-timber connections	CLT	Brown et al. [15]	cyclic shear	3–5	8–16	2–8*
axially loaded fasteners	structural steel**	/	JCSS [56]	tensile testing of structural steel	/	/	4–7***
	single screws in tension**	/	Krenn [65] Koppauer [64]	tensile testing of screws	65 29	1	< 2 < 5 (within batches < 3; between batches < 3)
			Ringhofer & Schickhofer [84]		3–15		< 3
			Sandhaas & Blaß [85]		~ 10		< 5****

* some samples featured brittle failure modes; variabilities of those capacities are higher than listed in the Table and up to 30 %.
** values included as lower boundary of expectable variability in ductile capacities of timber joints dominated by steel failure.
*** CoV[f_u] = 4 % and CoV[f_y] = 7 %, respectively, for the ultimate strength and yield limit of structural steel.
**** CoV between test series can be much larger, e.g. screws with a nominal diameter of 8 mm: observed variability between samples approx. 12 % for carbon steel screws and approx. 28 % for stainless steel screws.

the grain of boards in Brandner and Schickhofer [14].

According to the review of experiments featuring brittle connection failures, summarised in Table 1, CoV[R_b] can normally reach up to 30 %, while some isolated studies state even higher values up to 50 % [94]. However, such high values are excluded from Table 1 as they might be caused by other circumstances, e.g. the testing itself as well as heterogeneity within small samples and limited representativeness. CoV [R_b] = 30 % was therefore intentionally taken as a limit in Chapter 3, where the effect of extreme pairs of CoV[R_b] > > CoV[R_d] on the overstrength factor was demonstrated.

From a comparison of Table 1 and Table 2, the first obvious conclusion is that the upper limit of CoV[R_d] reaches approximately 20 %, which is clearly below the upper limit of CoV[R_b]. This finding was also reported in the papers reviewed. The experimental campaigns of Brown et al. [15] and Gavric et al. [47,46] were designed to test primarily ductile joints and connections, however, for some series unexpected brittle failures occurred, for which variability was significantly higher (CoV[R_b] up to 30 %) than for those test series featuring pure ductile failures (CoV[R_d] < 10 %). Similarly, many of the test campaigns in Table 1 (e.g. [3,20,94,96]) reported connection failures which were not purely brittle and thus classified as mixed mode failures, i.e. a partial ductile failure of fasteners prior to the global brittle failure of the connection. These experimental campaigns also report that capacities of connections displaying purely ductile failures are less variable than capacities of connections featuring brittle failure behaviour. Zarnani and Quenneville [96] reported that the coefficient of variation of those test series with ductile failures was less than 8 %, while similar connections with a rather brittle response reached 11–17 %.

The observations and conclusions deducible from reviewed experimental campaigns are also confirmed in theoretical studies; cf. Jockwer et al. [57] and Cabrero et al. [19]. Jockwer et al. [57] point out that irrespective of the defined geometric and material boundary conditions there will be always a mixture of failure modes (see also Chapter 2). Nevertheless, depending on settings, each failure mode will assume a different proportion, which means that the likelihood of observing them in experimental investigations might be very small. In their structural reliability studies they found clearly larger variabilities for capacities dominated by brittle failure modes (CoV[R_b] approx. between 14 % and 33 %) than for those dominated by ductile failures (CoV[R_d] approx. 6 %). Similar values are also reported by Cabrero et al. [19], with CoV [R_b] between approx. 19 and 25 % and CoV[R_d] between approx. 5 and 6 %. In addition, Ottenhaus et al. [77] clearly point out that the characterisation of ductility also requires knowledge of post-peak behaviour. Considering this, joints and connections which show some ductility before the maximum load but then globally fail in a brittle manner should rather not be rated as ductile joints or connections.

One reason for clearly lower variabilities in ductile load-bearing capacities is that ductile failures are usually governed by yielding of metal fasteners and/or fittings. The values of CoV[R_d] for ductile connection failures are therefore more similar to CoV[R_d] of individual ductile components (e.g. fasteners or fittings) and usually not affected by the clearly higher varying properties of timber components. For the ductile failure of axially loaded fasteners, CoV[R_d] reported in Table 2 originates from tension tests on single screws and on structural steel, which can be used as a base material from which the fittings (e.g. angle brackets, hangers, etc.) are assembled. Connections with axially loaded fasteners would typically reach CoV of less than 3 % and in no case greater than 5 %. From Table 2 it can also be concluded that the mechanism of ductile failure of connections with laterally loaded fasteners is more complex when compared with connections with axially loaded fasteners, as they usually contain several damaged components, fasteners, connectors and timber. Furthermore, the ductile failure of fasteners in shear depends on the formation of plastic hinges, which depends on the properties of both the fastener and the timber. The higher complexity and interaction of fasteners with timber are seen as reasons for the higher CoV[R_d] for connections with laterally loaded fasteners compared with axially loaded fasteners in Table 2.

Another finding from Table 1 is the relationship between CoV[R_b] from joints and the degree of homogenisation in properties of used timber construction products: Yurrita et al. [95], Yurrita and Cabrero [94] and Zarnani and Quenneville [96] tested the same joint types in LVL and glulam. The CoV[R_b] of joints in glulam is up to two times higher than in LVL, which is a much more homogeneous product. An additional source of uncertainty in brittle failures could be related also to the connection layup: fasteners in timber construction products not penetrating several layers/laminations behave similarly to fasteners inserted in structural timber (cf. [83,12,4,3]). Lastly, the number of fasteners per joint (N_f) is provided in Table 1 and Table 2. In general, a higher variability was obtained for joints with a smaller number of fasteners (see also [12]); however, the variability observed in experiments on single fasteners might be even lower.

In general, the lower the variability the higher the reliability and usually also the likelihood in predicting and controlling the corresponding property. As outlined in Jockwer et al. [57], low variabilities are clearly associated with ductile failure modes, i.e. the yielding of metal parts. To explicitly account for such an advantage, Jorissen [59] and Jockwer et al. [57] point out the need for different partial safety factors to be able to quantify differences in reliability. To conclude, brittle failure modes show a lower reliability associated with a larger variability and usually with a lower load-bearing capacity, which may finally also necessitate larger safety margins, i.e. larger partial safety factors.

5. Summary and conclusions

Capacity-based design means that ductile capacities dominate the overall failure hierarchy, i.e. the occurrence of brittle failures prior to ductile failures is reduced to a small (defined) probability. This is achieved by overdesigning the components displaying brittle failure by setting a corresponding overstrength factor. In timber engineering, ductility is hard to achieve by the timber itself; it has to be realised at the joints between the timber components, usually by means of metal fasteners and metal fittings. As is well known, metal parts are usually well predictable because of the very low variability in their mechanical properties, whereas mechanical properties of timber construction products and in particular their strengths might suffer from large variabilities. Brittle failure modes can occur in metal fittings or, more commonly, in timber components. Considering this, it becomes obvious that variabilities in brittle failure modes are expected to be (much) larger than in ductile failure modes.

This paper summarises and discusses currently available definitions of overstrength and the different approaches taken to decide on the values, based either on the LRFD framework or on structural reliability analyses. Current definitions may fail to achieve the design target of preventing brittle failures, in particular in cases of clearly larger variabilities in brittle resistances compared with ductile resistances and in cases of skewed distributions and/or high tails. The existing structural reliability problem demands a system analysis involving the interaction of actions with ductile and brittle resistances. Without this, as demonstrated, the calculated and designed overstrength concept might fail, i.e. brittle prior to ductile failure might be even more likely, although current approaches might conclude the contrary.

In this context it also becomes clear that a successful overstrength design is conditioned on a clear definition of ductility to enable a division into brittle and ductile failure modes. Ottenhaus et al. [77] clearly point out the need for a performance-based definition instead of a force- or displacement-based definition and also confirm, as previously outlined also by Brühl [16] and Flatscher [41], that a good combination of available ductility measures might be promising instead of balancing absolute and relative measures. They also outline the need to consider post-peak behaviour as part of a ductility classification, something which according to current test standards for fasteners and joints in timber engineering receives insufficient attention or is even prevented by regulating capacity per notation at fixed displacement limits.

Based on this summary, the main conclusions of this paper are:

- Semi-probabilistic approaches according to the LRFD framework (e.g. Approach I) are unable to account for uncertainties other than those intrinsically considered in partial safety and other adjustment factors applied in calculating design values based on characteristic properties; the proposed probabilistic approach (Approach III) allows to directly adjust the design requirements to a more realistic situation, making the approach more explicit and flexible and the outcome more reliable and predictable.

- Approach II appears intuitively correct as brittle resistance R_b is shifted to the right to accommodate sufficient safety in respect to ductile resistance R_d. However, it fails in case of CoV[R_b] $>>$ CoV[R_d] and/or unfavourable combinations of statistical distributions of R_b and R_d.
- Approach III directly translates the stated demand in capacity-based design that R_b needs to be overdesigned to achieve a ductile failure, which means that $P(R_b \leq E) < P(R_d \leq E)$. It appears a trivial approach, but in benchmarking with Approach I and II it is the only consistent way to achieve the overall design target. In contrast, Approach I and II are unreliable, both in terms of safety and from an economical point of view.

- Although some benefits were shown on the theoretical scale, implementing Approach III in the current design practice embedded in the LRFD framework would not be straightforward since designers usually do not have sufficient knowledge of the probabilistic characterisation of both brittle and ductile failure modes, or of the actions and the corresponding statistical distributions and parameters. Therefore, overstrength factors usable within the LRFD framework need to be calibrated by means of structural reliability analyses. This necessitates harmonising/updating statistical distribution models for representing physical/mechanical properties, for example based on the PMC of JCSS [54], as well as of variabilities and uncertainties associated with these properties, for which the summaries in Table 1 and Table 2 may serve as basis. The harmonisation process should be carried out with great caution as the choice of statistical distribution models and the associated variabilities have a considerable influence on the results of the reliability analyses, as demonstrated exemplarily in Kohler and Fink [62] and in the parameter study in Fig. 2.
- Designers operating within the LRFD framework mostly rely on engineering models, usually formulated on the basis of characteristic values, i.e. quantile values associated with a certain degree of confidence. The engineering models are commonly subject to a certain model bias and model uncertainty, which might lead to even higher overall uncertainties for ductile than for brittle failure modes if no other information is available (at least theoretically). A typical source of bias arises from the difference between designed/ordered and delivered quality, a circumstance that is quite common in the steel industry (see e.g. [49]) but problematic for timber structures. However, in such cases the material capacity should not be more realistically weighted by an overstrength factor, rather it is suggested to implement adequate quality control to assure that the delivered material quality confirms the ordered one and/or to adjust the design and/or execution accordingly.

- The review of experiments featuring timber connections with brittle and ductile failures in Table 1 and Table 2 revealed the possible ranges of coefficients of variation (CoV), which confirmed the theoretical assumptions: CoV[R_b] is within 5 and 30 % and usually dominated by the variability in the timber construction product properties, whereas CoV[R_d] is within 1 and 20 %, in most cases between 3 % and 8 %, and usually dominated by the variability of the properties of the component displaying ductile failure, commonly steel. Furthermore, by comparing the outcome of series which display either ductile or brittle failure, the ratio CoV[R_b] / CoV[R_d] is at least two or even greater.

Overall, this underlines the necessity of implementing the newly formulated Approach III for the overstrength factor in order to agree on the extra safety margin for brittle failures and the need for comprehensive structural reliability analyses so as to calibrate the overstrength factors accordingly to make them available for design processes within the LRFD framework. The motivation for these next steps comes from the partly extreme sensitivity of the overstrength factor in the combination of different statistical distribution models and the extra safety margin, as demonstrated in Fig. 2(b; d). For the analysed combinations of statistical distribution models, with $\{E; R_d; R_b\} \sim$ LN in Case (i) and $E \sim$ G, $R_d \sim$ N and $R_b \sim$ W in case (ii), for the latter partly extremely high compensations were necessary, with overstrength factors up to ten times higher than for Case (i). Concerning the extra safety margin, for $p_{f,Rd} = 10^{-2}$ the overstrength factors are nearly doubled compared with $p_{f,Rd} = 10^{-1}$. Looking at the literature and current regulations, $p_{f,Rd} = 4$ and 5 % are applied. As the variabilities in timber strength properties are usually higher than for steel, $p_{f,Rd}$ in the range of 1–5 % appears a meaningful basis for discussion.

Research Ethics

The authors declare that there is no issue concerning ethical standards.

Informed consent was obtained from all individual participants included in the study.

Intellectual Property

We confirm that we have given due consideration to the protection of intellectual property associated with this work and that there are no impediments to publication, including the timing of publication, with respect to intellectual property. In so doing we confirm that we have followed the regulations of our institutions concerning intellectual property.

Authorship

All listed authors meet the ICMJE criteria. We attest that all authors contributed significantly to the creation of this manuscript, each having fulfilled criteria as established by the ICMJE.

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Declaration of Competing Interest

No conflict of interest exists.

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Data availability

Data will be made available on request.

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