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# UPDATING EUROCODE 5 – DESIGN GUIDANCE FOR INCREASING THE ROBUSTNESS OF TIMBER STRUCTURES

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**ABSTRACT:** The European structural timber design standard EN 1995-1-1 *Eurocode 5* is currently under an extensive revision and the latest draft includes a new Informative Annex on *Additional guidance for increasing the robustness of timber structures*. The design rules in this new Annex are focused on direct design methods that provide explicit verifications for specific scenarios of assumed local failure: i) design for resistance to removal of load-carrying elements and; ii) design for segmentation using fuse elements. Dynamic effects related to a sudden loss of a load-carrying element are considered through dynamic amplification factors, which are provisionally prescribed as $\gamma_{dyn}$ = 2.0. The design of fuse elements is based on limiting their load-carrying capacity, ensuring that they fail under certain damage scenarios. The upper values of the load-carrying capacity of connections are very dependent on the specific type of connection, but a over-strength factor 4 = $R_{k,0.95}/R_{k,0.05}$ is provisionally prescribed.

**KEYWORDS:** robustness, progressive collapse, EN 1995-1-1, Eurocode 5, element-removal analyses, fuse elements

## 1 INTRODUCTION

### 1.1 BACKGROUND

Twenty years after their initial publication, the structural Eurocodes are currently under revision. A new topic that is now explicitly considered in the latest draft version of FprEN 1990:2022 *Eurocode 0 Basis of structural design* [1] is resistance to disproportionate collapse. This draft includes an Informative Annex E with additional guidance for enhancing the robustness of buildings and bridges. This Annex E, however, provides only very general guidance and does not provide a clear hierarchy of design strategies to prevent disproportionate collapses. The European standardisation committee CEN/TC 250/SC 5, responsible for EN 1995-1-1 *Eurocode 5 Design of timber structures* [2] took on the task of developing specific design guidance for increasing the robustness of timber structures. This work has been carried out by members of its Working Group 10 and the outcome is a new Informative Annex to the latest draft of prEN 1995-1-1 Eurocode 5.

### 1.2 SCOPE AND OBJECTIVES

This paper presents the new Annex A *Additional guidance for increasing the robustness of timber structures* of prEN 1995-1-1 Eurocode 5. It gives an overview of design strategies to increase resistance to disproportionate collapse, focusing on timber structures [3–5] and including lessons learned from past accidents [6,7]. Advantages and shortcomings of the different strategies are discussed. The design rules in the new Annex A are presented, alongside with their background. Aspects that need to be further investigated are also highlighted

### 1.3 DISCLAIMER

The opinions expressed in this paper are those of the authors and do not necessarily represent the official position of the Technical Committee CEN/TC 250/SC 5 or the opinion of its other members.

## 2 DEFINITIONS

FprEN 1990:2022 Eurocode 0 [1] defines *robustness* as the “ability of a structure to withstand adverse and unforeseen events without being damaged to an extent disproportionate to the original cause”. The Swiss standard SIA 260:2013 *Basis of Structural Design* [8] uses a somewhat different formulation and defines robustness as the “ability of a structure and its members to keep the amount of deterioration or failure within reasonable limits in relation to the cause”. These definitions of robustness coincide with what has been more broadly defined as *resistance to disproportionate collapse* or *collapse resistance* [9,10,3]. The definition of robustness adopted in this paper is instead *insensitivity to initial damage*, which is one of the aspects of *collapse resistance* (Figure 1), alongside with *vulnerability* (susceptibility of a structural component to be damaged

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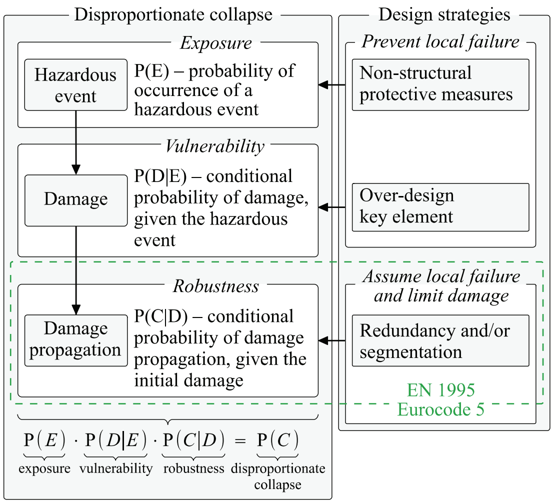


*Figure 1: Disproportionate collapse prevention strategies and robustness-focused scope of prEN 1995-1-1 Eurocode 5 (based on Starossek and Haberland [10] and Palma et al. [4]).*

by an abnormal event) and *exposure* (abnormal events, not explicitly considered in ordinary design) [11]. A robust structure is, therefore, less prone to disproportionate collapses. Finally, a disproportionate collapse might not be *progressive*, as statically determinate structures will most likely collapse after the failure of a single component.

The main difference between design for robustness and design for accidental situations [12] is that the latter assumes identified and quantified abnormal actions (e.g. far-field blasts or vehicle impacts), against which the structure can be explicitly designed, so that a specified reliability level is reached. Design for robustness, on the other hand, deals with exposures that cannot be identified and/or quantified. These can be errors in the design, construction, or use of the structure that lead to structural deficiencies, or external man-made (e.g. accidental or deliberate blast, impact, or fire) or natural events (e.g. extreme snow or wind loads, or degradation of the structure)Design for robustness deals mostly with threat-independent scenarios that assume an initial notional damage (e.g. sudden removal of a load-carrying member). The corresponding robustness-related design strategies are focused on limiting damage propagation through redundancy and/or segmentation.

## 3 DESIGN FRAMEWORK FOR RESISTANCE TO DISPROPORTIONATE COLLAPSE

Starossek's [11] design framework for resistance to disproportionate collapse (Figure 2) can be divided into the following main parts [13]:

i) risk assessment/classification of the structure;
ii) specification of hazard scenarios;
iii) specification of performance objectives;
iv) selection of design strategies
v) carrying out of verification procedures.

The level of *design requirements* should be based on the risk assessment of building. This can be achieved by undertaking the risk assessment and building classification frameworks described, e.g. in Section 4.1. The specification of *hazardous scenarios*, such as threat-specific (e.g. impact of a car in a ground-floor column) or non-threat-specific events (e.g. notional damage such as a sudden removal of a structural component), and *performance objectives* (i.e. the acceptable level of damage/consequences) should involve other stakeholders besides the owner and the design team, namely the relevant civil and building authorities and even insurance companies. For major projects, the specification of hazardous scenarios requires some experience, since the creation of general rules is difficult due to the many possible scenarios and the project-specific nature of many of them. Once the hazard scenarios are considered and the performance objectives are set, the structural design team then selects the *design strategies* (e.g. protection or overdesign measures to prevent local damage, robustness measures to limit damage propagation) and the *design verification procedures* (e.g. based on structural analysis models or even testing). A schematic overview of this design framework for resistance to disproportionate collapse is given in Figure 2.

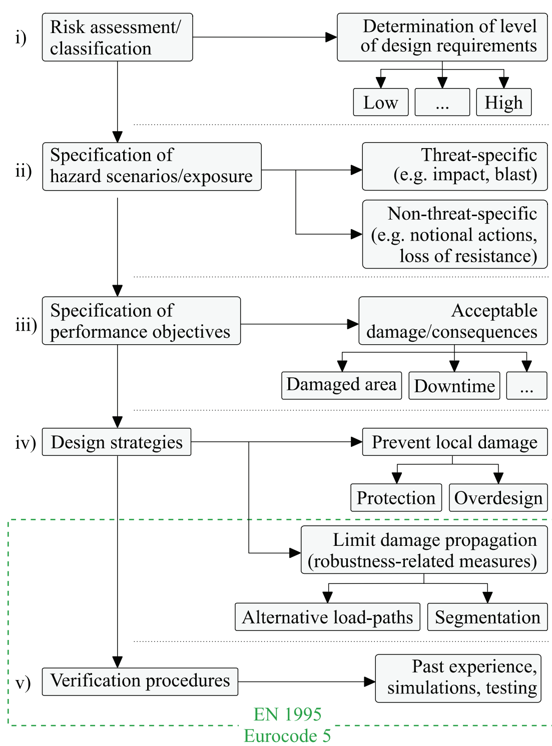


*Figure 2: Design framework for resistance to disproportionate collapse, based on Starossek [11], and the scope of prEN 1995-1-1 Eurocode 5.*


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For buildings with low importance and exposure, it should be possible to achieve an adequate level of resistance to disproportionate collapse without any explicit design verifications. However increasingly complex verifications are often required for buildings of high importance and/or exposure.

## 4 DESIGN IN ACCORDANCE WITH THE STRUCTURAL EUROCODES

### 4.1 APPROACH

The structural Eurocodes in their current and upcoming versions, as well as most other current structural design codes, adopt a limit-state design approach, based on a probabilistic representation of the design parameters so that predefined appropriate reliability levels are reached. EN 1990 *Eurocode 0* establishes the design principles, based mostly on the partial factor method, EN 1991 *Eurocode 1* specifies the actions and their values, and timber-specific aspects are addressed in EN 1995 *Eurocode 5*. (Figure 3).

FprEN 1990:2022 *Eurocode 0* [1] recommends that structures "be designed to have an adequate level of robustness" and states that "for most structures, design in accordance with the Eurocodes is assumed to provide an adequate level of robustness without the need for any additional design measures". This statement might be based on the fact that not so many disproportionate collapses have been observed [14] and that it has been traditionally accepted that "structural codes may consider these [progressive collapse] requirements satisfied if all other requirements can be fulfilled" [15,16]. However, assuming that designing in accordance with the Eurocodes automatically provides an adequate level of robustness is a rather "optimistic" view and has been shown not to be true in some cases, as some studies on multi-storey timber buildings have shown [17].

As other design frameworks for resistance to disproportionate collapse [18,19], FprEN 1990:2022 *Eurocode 0* [1] starts by assessing the *importance* of the building, i.e. the *direct* and *indirect* risks or consequences of a collapse, and its *exposure*, i.e. the probability of occurrence of a hazardous event (e.g. accident, malicious or unintentional actions). FprEN 1990:2022 Eurocode 0 [1] establishes five consequence classes (Table 1), based on an indicative qualification of consequences (loss of human life or personal injury and economic, social or environmental consequences). The exposure of the structure, i.e. the level of threat, is not explicitly accounted for (or it is assumed to be proportional to the importance of the building) [13].

FprEN 1990:2022 Eurocode 0 [1] then allows for the design method for "providing enhanced robustness" to be selected based on the consequence class (CC) of the structure (Table 2).

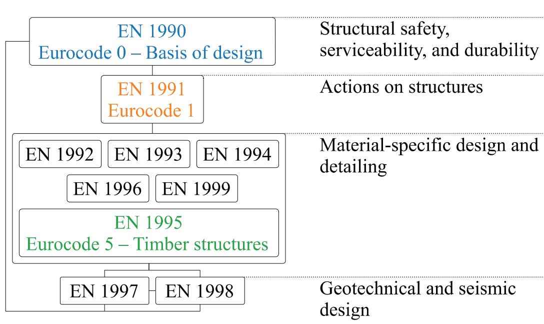


***Figure 3:*** *Links between the Eurocodes (https://eurocodes.jrc.ec.europa.eu/showpage.php?id=13)*

### 4.2 SHORTCOMINGS

One of the shortcoming regarding resistance to disproportionate collapse is that design verifications are made only on the local (component) level, following an element-by-element approach.It is hereby assumed that the reliability of the structure is not much smaller than the reliability of each member or connection [20]. However, the response of a structure to an initial local damage is dependent not only on the behaviour of its components in

***Table 1:*** *Qualification of consequence classes (FprEN 1990 2022* ***[1]****). Assignment of structures to consequences classes is a Nationally Determined Parameter (NDP), i.e. it can be defined at a national level by regulatory authorities in each Member State.*

| Consequence class (CC) | Indicative qualification of consequences | |
|---|---|---|
| | Loss of human life or personal injury [a] | Economic, social or environmental consequences [a] |
| CC0 – Lowest | Very low | Insignificant |
| CC1 – Lower | Low | Small |
| CC2 – Normal | Medium | Considerable |
| CC3 – Higher | High | Very great |
| CC4 – Highest | Extreme | Huge |

[a] The consequence class is chosen based on the more severe of these two columns.

***Table 2:*** *Indicative design methods for "enhancing robustness" taken from FprEN 1990 2022 [1].*

| Consequence class | Design methods |
|---|---|
| CC1 – Lower | No design methods to provide enhanced robustness need be applied. |
| CC2 – Normal | When specified by the relevant authority or as agreed for a specific project by the relevant parties, either:<br>a) For buildings: use of prescriptive design rules for ties to provide integrity, ductility and alternative load paths; or<br>b) Design of particular structural members as 'Key members'; and/or<br>c) Segmentation. |
| CC3 – Higher | Satisfy the requirements for CC2 appropriately adapted and in addition consider, where appropriate:<br>a) potential initial failure events;<br>b) propagation of failure;<br>c) resulting consequences;<br>d) risks. |

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isolation, but on their arrangement and connectivity to the structure and the requirements for the reliability of the components should depend on the characteristics of the structure [20].

Another issue of the current approach regarding design for resistance to disproportionate collapse is the difficulty in dealing with extreme risks, i.e. low probability/high consequence events, such as a disproportionate collapse, due to the lack of statistical data [11]. Strategies to enhance resistance to disproportionate collapse are "not generally associated with a target level of reliability as in structural member design against identified actions and could involve consideration of a conditional reliability" [1]. Therefore, these events cannot easily be handled within the current reliability-based framework, even though target reliability levels have been set in the past for ultimate limit states corresponding to progressive collapse [15,16].

As mentioned above, it is difficult to judge the resistance of a structure to disproportionate collapse simply based on the individual behaviour of its elements and neglecting the structure's sensitivity to initial damage. Tests on a medium-rise multi-storey platform-frame building, with structural walls and floors constructed from small section timber studs and cladded with wood-based panels, showed that such buildings had "significant inherent robustness and capacity to span over removed panels" [21–23]. On the other hand advanced mechanical simulations, which also took into account the variability of mechanical properties, of a medium-rise cross-laminated timber building showed that simple compliance with building codes [4] (both the Eurocodes and the *National Building Code of Canada* and CSA O86-09:2010) led to a probability of disproportionate collapse as high as 32% after removal of a load-carrying element [17].

Finally, the indicative design methods for "enhancing robustness" prescribed by FprEN 1990:2022 *Eurocode 0* [1] (Table 2) are not actually focused on increasing robustness, i.e. insensitivity to initial damage, and can hence be misleading. Strategies such as the overdesign of "key members" reduce the local *vulnerability* of the structure, but do not necessarily lead to increased *robustness* (Figure 1). Prescriptive design rules (e.g. providing ties or imposing failure modes with sufficient ductility in the connections) might, in some cases, promote rather than prevent collapse progression [7,11].

## 5 DESIGN STRATEGIES AGAINST DISPROPORTIONATE COLLAPSE

Given that different structures are vulnerable to disproportionate collapse in different degrees, design for resistance to disproportionate collapse cannot be completely independent of the specifics of each project. Hence, it should be considered in the early (i.e. conceptual) stages of the design process [4,5]. For structures in higher consequence classes, resistance to disproportionate collapse should be based on more detailed analyses that include explicit *direct* design verifications for specific scenarios (e.g. element-removal analyses). This *direct* approach is more compatible with architectural complexity and design procedures that allow evaluating the global structural behaviour as a function of the behaviour of single elements. It can give valuable insights regarding resistance to disproportionate collapse of the structure. However, the required time, skill, and computational effort is greater than with the *indirect* approach. In any case, design for resistance to disproportionate collapse should not be interpreted as "simply applying rules" as this could lead to these aspects being addressed too late in the design process, hence limiting the applicable strategies or involving considerable costs. Rather should design for resistance to disproportionate collapse be addressed in the conceptual design of the structures.

Resistance to disproportionate collapse can be achieved at different levels (Figures 1 and 2):

- preventing local failures by
  - *adopting protective measures* to reduce the probability, extent or mitigate the exposure of the structure to abnormal events, or by
  - *overdesigning key elements* to reduce the probability of damage in case of a hazardous event, i.e. reduce the vulnerability of key elements and increase safety against initial failure;
- assuming local failure and limiting damage propagation (robustness-related measures to increase insensitivity to initial damage), through
  - *redundancy*, e.g. through alternative load paths and/or
  - *segmentation*, e.g. ensuring that collapsing parts are isolated from the rest of the structure.

Design strategies based on adopting *protective measures* often fall outside the scope of structural design (e.g. vehicle barriers, access control, active fire protection). Strategies based on *overdesigning key elements* should be a last resort [3,5,25,26], used only in cases where other alternatives are not viable or too costly. Design of key elements follows the common design procedure, even if the considered hazards and assumed actions are anything but common, like blast and impact, and the corresponding structural design can often be done in accordance with available guidance [4,12].

The *redundancy* strategy is based on providing an alternative load path (ALP) for the forces previously transmitted through load-carrying components that are assumed to have failed. It is based on assessing the behaviour of the remaining structure after an initial notional damage. A commonly assumed initial damage is the notional removal of one or several components of the structure. Then a so-called *element-removal analysis* is performed, with the objective of evaluating if the

[4] The building was not designed for earthquake resistance, only for a 1.0 kPa horizontal wind action. Explicit design for earthquake resistance could have indirectly increased the resistance to disproportionate collapse.

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remaining structure is able to accommodate the damage. Redundancy on its own might not be suitable to avoid disproportionate collapse. In the case of repetitive structures, systematic design or execution errors can compromise the ability of a structure to redistribute loads and lead to progressive collapse [6,7], as the alternative load paths are all affected by a common-cause failure.

In these cases, *segmentation* can be an adequate design strategy. The objective of this strategy is to compartmentalise the structure in a way that collapse progression after an initial damage is halted at predefined locations, i.e. at the segment's borders, either through fuse-type elements or by having control joints at which the segments are physically separated. Most common solutions for *vertical segmentation* rely on providing shock-absorbing zones with high energy dissipation capacity. Examples of vertical segmentation are scarce, however, the 14-storey timber building "Treet", in Norway, includes a paradigmatic example [27]: this building has two "power storeys" that carry a prefabricated concrete slab on top of which four levels of residential modules are stacked; these "power storeys" should be able to halt a progressive collapse of the stacked residential modules, limiting the extent of collapse.

# 6 DESIGN FOR ROBUSTNESS IN prEN 1995-1-1 EUROCODE 5

## 6.1 SCOPE

As already discussed in Section 2, mistaking robustness with resistance to disproportionate collapse is a common misconception. Robustness-related strategies assume that damage has occurred and focus on limiting the extent of the damage, whereas increasing resistance to disproportionate collapse can be achieved through other strategies that are not related to robustness. Two other common misconceptions on the scope of prEN 1995-1-1 Eurocode 5 are that it should specify: i) *hazardous scenarios*, e.g. impact of a car in a ground-floor column, or a notional removal of a load-carrying component; and ii) *performance objectives*, e.g. allowed collapsed/ damaged area. However, as discussed in Section 3, these aspects are not to be addressed by the structural design team alone and must involve other stakeholders, such as the owner, the authorities, and other relevant parties.

The simple application of prescriptive design rules does not imply that performance requirements are met [11], hence they are therefore not addressed in prEN 1995-1-1 Eurocode 5. Preventing local failures by adopting protective measures also falls outside of the scope of structural design codes. Overdesigning key elements can be done within the normal design framework of the Eurocodes [4,12], even if the design actions are arbitrarily determined instead of statistically assessed (Section 5). In addition, even if overdesigning identified key elements contributes to increasing the resistance to disproportionate collapse, it is mostly unrelated to increasing robustness.

Ensuring robustness cannot simply be based on "applying rules". Choosing an adequate structural concept is as important for robustness as it is for seismic design. The new Annex A of prEN 1995-1-1 Eurocode 5 is focused on direct design methods, in which robustness can be explicitly demonstrated for specified initial damage scenarios (e.g. loss of one or more columns) and performance requirements (e.g. acceptable extent of collapse as a percentage of floor area, volume, or costs). This can only be achieved for design strategies based on *assuming local failure* and limiting the damage that follows, namely *redundancy* through alternative load paths and/or segmentation through fuse elements.

## 6.2 DRAFT CLAUSES IN prEN 1995-1-1

### 6.2.1 Overview

The members of Working Group (WG) 10 *Basis of design and materials*, of the European standardisation Technical Committee CEN/TC 250/SC 5 *Eurocode 5* have drafted robustness-related clauses for the upcoming revised version of Eurocode 5. A note in the main part of the standard highlights that designing for robustness is more related to the structural concept, redundancy, adequate choice of structural materials, and structural detailing than to complex analyses. More detailed provisions are given in informative Annex A *Additional guidance for increasing the robustness of timber structures*.

Annex A contains provisions for design strategies to increase robustness based on limiting the total damage following assumed scenarios of initial local failure (Figures 2 and 4), namely:

i) creation of alternative load paths (*redundancy* strategy); and
ii) segmentation of the structure into distinct parts that are able to collapse without inducing structural failures in other parts, by means of fuse elements.

Annex A is based on the assumption that the design will be based on linear-elastic structural analyses, as the other parts of Eurocode 5. Therefore, non-linear and dynamic effects have to be considered through adequate factors imposed on actions, resistances, and stiffnesses. It is not yet clear if enough data is available to derive all these factors, or if its application will be limited specific types of elements and connections.

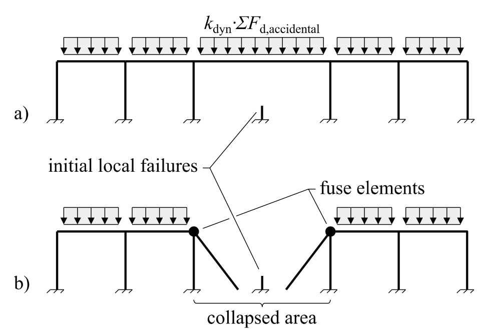


***Figure 4:*** *Design strategies in the latest draft of Annex A of the prEN 1995-1-1 Eurocode 5: a) design for removal of load-carrying elements; b) design for segmentation using fuse elements.*

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### 6.2.2 Design for removal of structural elements

The design verifications for *resistance to removal of load-carrying elements* includes two scenarios:

i) failure of a structural element including dynamic effects and impact of falling debris; and
ii) remaining structure, without the failed structural element.

In the first scenario, the dynamic effects corresponding to the sudden failure of a load-carrying element are considered by applying a partial factor $\gamma_{dyn}$ on the actions. Different values have been proposed for such dynamic amplification factors. Grantham and Enjily [23] propose a modification factor of 2.0 (for timber-frame walls). Mpidi Bita et al. [17] state that for a 12-storey CLT building the forces from the dynamic simulation were about 1.5 times higher than the outcome from static analysis. Dietsch and Kreuzinger [28] determine dynamic amplification factors due to brittle failures in timber elements with shear reinforcement in form of self-tapping screws around 1.2. Cheng et al. [29] performed dynamic element-removal experiments on small-scale beam-column frames and report dynamic amplification factors between 1.3 and 1.7 for frames with ductile connections and above 2.0 for frames with brittle connections. Stevens et al. [30,31] present results that show a range of dynamic amplification factors from 1.20 to 1.85 for steel buildings and from 1.00 to 1.40 for concrete buildings. In the current draft of Annex A, a single provisional value $\gamma_{dyn} = 2.0$ is recommended, but it is still under discussion whether different values should be given depending on the type of structure and connections.

In this first scenario, the prescribed mechanical properties are those corresponding to an *instantaneous* load-duration class in EN 1995-1-1:2004 Eurocode 5 [2]. The prescribed combination of actions is that for accidental design situations [1].

Some element-removal scenarios might not impose dynamic effects on the structure (e.g. gradual loss of an element during a fire, excessive settlement of a foundation). In such cases, only the second design scenario is relevant. It is also important to note that whenever an alternative load path is mobilised, the failure might go unnoticed if the structure does not exhibit visible deflections or extensive cracking. Such a situation can eventually lead to a progressive collapse. This is particularly important in the case of very stiff secondary systems [6,7].

### 6.2.3 Design for segmentation using fuse elements

Robustness can be increased by designing for segmentation, e.g by isolating collapsing parts of the structure. Segmentation by interrupting the structural continuity is particularly important to limit the extent of damage in the case of systematic flaws that reduce the resistance of the structure at many locations, namely in repetitive structures.

Design for segmentation may be achieved by limiting the upper value of the load-carrying capacity of fuse elements, which are structural elements that are presumed to fail under certain damage scenarios in order to halt a progressive collapse. Given the large variability exhibited by the mechanical properties of timber, the fuse elements should be connections and the upper design value of their load-carrying capacity $R_{d,sup}$ should be smaller than the design effect of the action that it will carry in the case of a collapse $E_{d,fuse}$

$$R_{d,sup} \leq E_{d,fuse}. \tag{1}$$

In the current draft of Annex A, the upper design value of the load-carrying capacity of the fuse element $R_{d,sup}$ is calculated using the 95$^{th}$-percentile value $R_{k,0.95}$ of the load-carrying capacity. Partial safety factors $\gamma_M$ or $\gamma_R$ are those for accidental design situations. The strength modification factor $k_{mod}$ to be used is that for actions assigned to the *instantaneous* load duration class.

The values of $R_{k,0.95}$ are very dependent on the specific type of connection. In the absence of more ac-curate information, it is proposed that $R_{k,0.95} = 4 \cdot R_{k,0.05}$ is used. In a more recent experimental study on nailed and screwed steel-to-timber connections, Munch-Andersen [32] found an over-strength factor (ratio between the 95$^{th}$-and the 5$^{th}$-percentile values) of about 2.2. Connections with laterally-loaded dowel-type fasteners are prone to have considerable over-strength due to a possible pronounced rope-effect, which might complicate their use as fuse elements.

The design effect of the action $E_{d,fuse}$ is typically based on analyses of specific scenarios of initial local failure and includes relevant dynamic effects. In many cases, $E_{d,fuse}$ is mainly dependent on the self-weight of the part of the structure that is assumed to collapse. The fuse elements can also be conceptualised as in the example given in Figure 5 (not applicable in seismic regions), in which the rotations during failure lead to the detachment of failing and attached member.

It is in any case important to ensure that the remaining structure is able to carry relevant horizontal loads and that its members are adequately braced.

Design for segmentation and design for removal of structural elements can be used in combination, e.g. the structure can be segmented and alternative load paths can be provided within the segments.

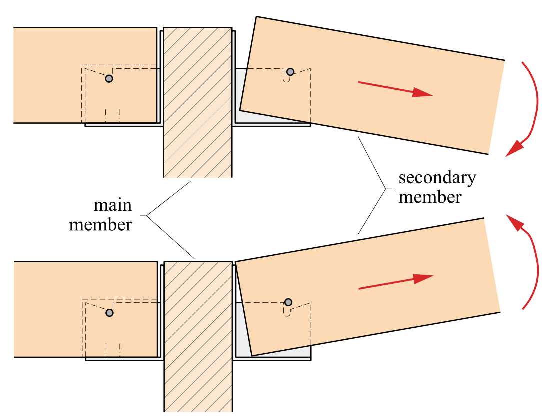


***Figure 5:*** *Example of fuse element by Waidelich [33].*


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## 7 CONCLUSIONS

Codification of resistance to disproportionate collapse of timber structures within the framework of the Eurocodes is not straightforward, in particular due to the lack of well-established design procedures. Designing for robustness (which is a part of the resistance to disproportionate collapse) shall primarily be considered in the early stages of design, e.g. by choosing an adequate structural concept, materials, and detailing.

The new Annex A of prEN 1995-1-1 Eurocode 5 is focused on direct design methods, in which *robustness* can be explicitly demonstrated for specified initial damage scenarios and performance requirements. Annex A contains provisions for design strategies to increase robustness based on limiting the total damage following assumed scenarios of initial local failure, namely the creation of alternative load paths and segmentation using fuse elements. Dynamic effects related to a sudden loss of a load-carrying element are considered through dynamic amplification factors, which are provisionally prescribed as $\gamma_{dyn} = 2.0$. The design of fuse elements is based on limiting their load-carrying capacity, and an over-strength factor $4 = R_{k,0.95}/R_{k,0.05}$ is provisionally recommended.

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