Robustness Evaluation of Timber Structures – Results from EU COST Action E55:WG3

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ABSTRACT: The present paper outlines results from working group 3 (WG3) in the EU COST Action E55 – 'Modelling of the performance of timber structures'. The objectives of the project are related to the three main research activities: the identification and modelling of relevant load and environmental exposure scenarios, the improvement of knowledge concerning the behaviour of timber structural elements and the development of a generic framework for the assessment of the life-cycle vulnerability and robustness of timber structures.

1 INTRODUCTION

Timber is an efficient building material, not least in regard to its mechanical properties but also because it is a highly sustainable material considering all phases of the life cycle of timber structures: production, use and decommissioning. Timber is a widely available natural resource throughout Europe; with proper management, there is a potential for a continuous and sustainable supply of raw timber material in the future. Timber is a light material and compared to its weight the strength is high; the strength to weight ratio in grain direction is even higher than for steel. However, considering its beneficial properties, timber is still not used to its full potential in the building and construction sector. Many building developers, architects and structural engineers do not consider timber as a competitive building material compared with concrete, steel or masonry. Attributes such as high performance regarding reliability, serviceability and durability are generally not associated with timber as a building material. One of the main reasons for this is that timber is a highly complex material; it actually requires a significant amount of expertise to fully appreciate the potential of timber as a structural building material. There are also a number of issues which need to be further researched before timber can achieve the same recognition as a high quality building material such as steel and concrete. These issues are the focal point of the EU COST Action E55 – 'Modelling of the performance of timber structures' ending in 2011 (Koehler 2006). The objectives of the project are achieved according to three main research activities: the identification and modelling of relevant load and environmental exposure scenarios, the improvement of knowledge concerning the behaviour of timber structural elements and the development of a generic framework for the assessment of the life-cycle vulnerability and robustness of timber structures. The present paper outlines the latest results achieved by working group 3 (WG3) which are dealing with robustness of timber structures. WG3 considers the subtasks: definition of structural robustness of timber structures, quantification of robustness and methods of assessing robustness of timber structures as well as methods of designing for robustness of timber structures. Recently results from WG3 subtasks have been presented in the factsheets (Branco and Neves 2009; Dietsch 2009; Kirkegaard and Sørensen 2009; Sørensen et al. 2009). The following sections outline the main contributions.

2 ROBUSTNESS FRAMEWORK

During the last years, robustness of structural systems has obtained a renewed interest due to a much more frequent use of advanced types of structures with limited redundancy and serious consequences in case of failure. The interest has also been facilitated due to recently severe structural failures such as that at Ronan Point in 1968, the World Trade Centre towers in 2001, the Siemens Arena in 2003 and the Charles de Gaulle International Airport in 2004. In order to minimize the likelihood of such disproportionate structural failures many modern building codes (CEN 2002a; CEN 2002b) consider the need for robustness in structures and provide strategies and methods to obtain robustness. One of the main issues related to robustness of structures is the definition of robustness. The most general definitions are very similar to each other particularly those taken from structural codes despite the use of different terms (robustness, structural integrity, but also progressive collapse prevention). These definitions are focused on the prevention from an escalation of damage within the structure, given a certain initial (localized) failure/damage. During the last decades a variety of research efforts have attempted to quantify aspects of robustness such as redundancy and identify design principles that can improve robustness (Baker et al. 2007; Canisius et al. 2007). Due to many potential means by which a local collapse in a given structure can propagate from its initial extent to its final collapse state, there is no universal approach for evaluating the potential for disproportionate collapse, or for robustness (Ellingwood et al. 2007).

The requirement for robustness is specified in most buildings codes in a way like the general requirements in the two Eurocodes: *EN 1990 - Basis of Structural Design* (CEN 2002a) and *EN 1991-1-7 - Accidental Actions* (CEN 2006). *EN 1990 - Basis of Structural Design* (CEN 2002a) provides principles, e.g. it is stated that a structure shall be ‘designed in such a way that it will not be damaged by events like fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause’. It also states that potential damage shall be avoided by ‘avoiding, eliminating or reducing the hazards to which the structure can be subjected; selecting a structural form which has low sensitivity to the hazards considered; selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localized damage; avoiding as far as possible structural systems that can collapse without warning; tying the structural members together’. *EN 1991-1-7 - Accidental Actions* (CEN 2006) provides strategies and methods to obtain robustness. Actions that should be considered in different design situations are: 1) designing against identified accidental actions, and 2) designing against unidentified actions (where designing against disproportionate collapse, or for robustness, is important).

The basic concepts in robustness are presented in Figure 1 and the following issues:

a) Exposures which could be unforeseen unintended effects and defects (incl. design errors, execution errors and unforeseen degradation) such as
   - unforeseen action effects, incl. unexpected accidental actions
   - unintended discrepancies between the structure's actual behaviour and the design models used
   - unintended discrepancies between the implemented project and the project material
   - unforeseen geometrical imperfections
   - unforeseen degeneration
b) Local damage due to exposure (direct consequence of exposure)
c) Total (or extensive) collapse of the structure following the local damage (indirect consequence of exposure)

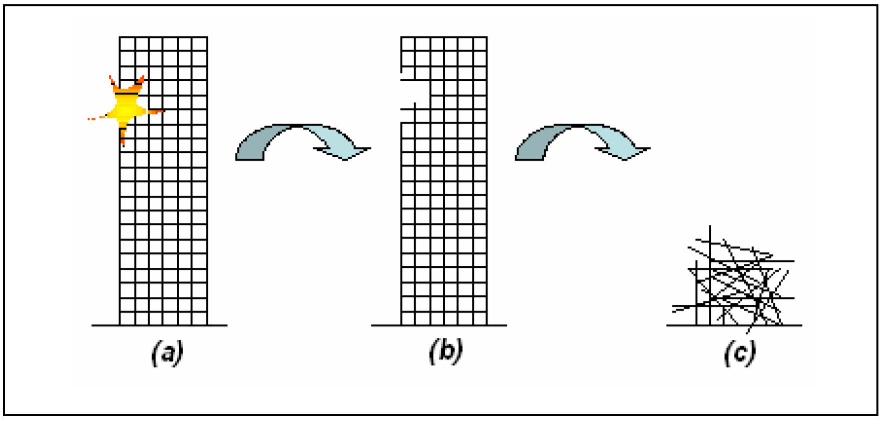

Figure 1: Illustration of the basic concepts in robustness (CEN 2006) .

Robustness requirements are especially related to the step from b) to c), i.e. how to avoid that a local damage develop to total collapse, i.e. robustness is meant to avoid failures caused by errors in the design and construction, lack of maintenance and unforeseeable events. During the last decades there has been a significant effort to develop methods to assess robustness and to quantify aspects of robustness. An overview of these methods is given in (Baker et al. 2007). The basic and most general approach is to use a risk analysis where both probabilities and consequences are taken into account. Approaches to define a robustness index can be divided in the following levels with decreasing complexity (Vrouwenvelder and Sørensen 2009) :

- A risk-based robustness index based on a complete risk analysis where the consequences are divided in direct and indirect risks (Baker et al. 2007)
- A probabilistic robustness index based on probabilities of failure of the structural system for an undamaged structure and a damaged structure (Frangopol D.M. and J.P. 1987; Fu G. and D.M. 1990)
- A deterministic robustness index based on structural measures, e.g. pushover load bearing capacity of an undamaged structure and a damaged structure (ISO 2007).

Other simple measures of robustness have been proposed based on e.g. the determinant of the stiffness matrix of a structure with and without removal of elements. Due to many potential means by which a local collapse in a given structure can propagate from its initial extent to its final state, there is no universal approach for evaluating the potential for disproportionate collapse, or for robustness (Ellingwood et al. 2007). However, for reduction of the risk of collapse in the event of loss of structural element(s), a structural engineer may take necessary steps to design a collapse-resistant structure that is insensitive to accidental circumstances. This means that the following structural traits should be incorporated in the design (Ellingwood et al. 2007):

- *Redundancy*: incorporation of redundant load paths in the vertical load carrying system.
- *Ties*: using an integrated system of ties in three directions along the principal lines of structural framing.
- *Ductility*: structural members and member connections have to maintain their strength through large deformations (deflections and rotations) so the load redistribution(s) may take place.
- *Adequate shear strength*: as shear is considered as a brittle failure, structural elements in vulnerable locations should be designed to withstand shear load in excess of that associated with the ultimate bending moment in the event of loss of an element.
- *Capacity for resisting load reversals*: the primary structural elements (columns, girders, roof beams, and lateral load resisting system) and secondary structural elements (floor

beams and slabs) should be designed to resist reversals in load direction at vulnerable locations.

- *Connections (connection strength):* connections should be designed in such way that it will allow uniform and smooth load redistribution during local collapse
- *Key elements:* exterior columns and walls should be capable of spanning two or more stories without buckling, columns should be designed to withstand blast pressure etc.
- *Alternate load path(s):* after the basic design of structure is done, a review of the strength and ductility of key structural elements is required to determine whether the structure is able to "bridge" over the initial damage.

3 EVALUATION OF ROBUSTNESS

To reach a better understanding of aspects which influence on the robustness of timber structures several benchmarks examples (Cizmar et al. 2010; Kirkegaard and Sørensen 2008; Kirkegaard et al. 2009) have been considered where the purpose and aim have been:

- to investigate system reliability (spatial distribution of strength and stiffness) and robustness of timber structures using probabilistic methods.
- to model failure modes (different types incl. connections and behaviour after failure: ductile / brittle).
- to discuss how to model the effect of human errors (unintentional errors and defects).
- to model local failures – due to local extreme snow load, design/execution/maintenance errors in connections.
- to identify key elements, and how to design key elements.

Especially probabilistic modelling of failure modes has been considered where characteristics like redundancy and ductility have been evaluated. In Eurocodes ductility is only awarded for concrete and steel structures but not for timber structures. It is well-know that structural systems can redistribute internal forces due to ductility of a connection, i.e. some additional loads can be carried by the structure. The same effect is also possible for reinforced concrete structures and structures of steel. However, for timber structures codes do not award that ductility will result in a semi-rigid behavior plus higher level of safety due to a lower probability that premature brittle failures occur as well as possible redistribution of forces for statically undetermined structures either internally in the joint or to other structural elements. A redistribution of forces, a so-called statical system effect, will usually increase the reliability of the whole structural system and give an extra safety margin compared to the deterministic code results. In general when a structural system collapses one or more structural elements have failed. Such a failure mode can for any mechanical system be assigned to one of the following three categories: series systems, parallel systems or combination of series and parallel system (also referred as hybrid systems). In series systems failure of any element leads to the failure of the system. Parallel systems are those systems in which the combined failure of each and every element of the system results in the failure of the system (Madsen et al. 1986). Since a redistribution of the load effects takes place in a redundant structural system after failure of one or more of the structural elements it becomes very important in parallel systems to describe the behaviour of the failed structural elements after failure has taken place. If the structural element has no strength after failure the element is said to be *perfectly brittle*. If the element after failure has a load-bearing capacity equal to the load at failure, the element is said to be *perfectly ductile.* Clearly all kinds of structural elements and material behaviours cannot be described as perfectly brittle or perfectly ductile. All kinds of combinations in between exist, i.e. some, but not all, of the failure strength capacity is retained after initial failure.

It is very important for calculation of a parallel system reliability to describe the behaviour of the failed element after the failure has taken place. For the series system this is not needed because when one element fails the failure of system is inevitable, i.e. a non-redundant system. However, before the reliability modelling in a parallel system of failure elements can be performed the structural behaviour of the considered failure mode must be clarified. More specifically the failure of the structural elements and consequences with determination of residual load-carrying capacity and load redistribution in each step in the structural element failure sequence must be described. Then the failure functions of the failure elements in the parallel system can be formulated. Failure function no. 1 models failure in parallel system element no. 1 without failure in any other elements. Failure function no. 2 models failure in parallel system element no. 2 with failure in the structural element corresponding to failure element no. 1 (i.e. after redistribution of loads). Failure function no. 3 then models failure of parallel system element no. 3 with failure in the structural elements corresponding to failure element nos. 2 and 1, etc.

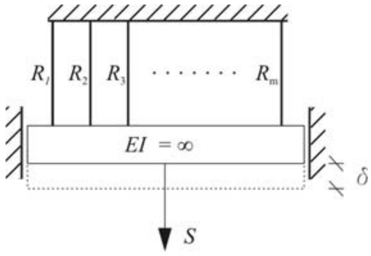

Figure 1: Mechanical model for parallel system.

A stochastic load S is assumed and a parallel system consisting of m independent elements with identically distributed stochastic strengths R_i, see Figure 1, Then the system failure occurs if the maximum system strength is exceeded by the load for a given imposed deformation δ, i.e. the component failure of the parallel system is given as the intersection of the individual failure events, i.e. the probability of system failure can be given as

$$P_{f,sys} = P\left(\max_{\delta} \sum_{i=1}^{m} \left\{ R_i(\delta) - S \leq 0 \right\} \right) \tag{1}$$

By using (1) (Daniels 1945; Gollwitzer and Rackwitz 1990; Hendawi and Frangopol 1994) have presented results for probabilities of failure for the system in figure 1 under different post-failure member behaviours (ductility), correlations, strength and load variabilities and number of members. In general it is shown that for a small number of elements the brittle system behaves much like the series system. As the number of elements is increased the reliability of the parallel system is increased significantly (and vice-versa for the series system). Further as the ductility increases linearly the reliability of the system increases much steeper (exponentially), so a relatively little ductility accounts for a considerable extra reliability. At last increases in correlation between elements imply a system reliability decrease. In summary, if there is a moderate degree of ductility, ductile systems will provide significant extra reliability only if elements are low correlated or with no correlation at all and if the load variability is not too high. On the other hand, if there is a brittle behaviour, there is a relatively little effect of the system (especially for the small systems). There is even a small negative effect for medium coefficients of strength variation.

As mentioned in section 2 (Frangopol & Curley 1987) and (Fu & Frangopol 1990) proposed some probabilistic measures related to structural redundancy – which also indicates the level of robustness. A redundancy index (RI) and a related redundancy factor β_R are defined by:

$$RI = \frac{P_{f(\text{damaged})} - P_{f(\text{intact})}}{P_{f(\text{intact})}} \qquad \beta_R = \frac{\beta_{\text{intact}}}{\beta_{\text{intact}} - \beta_{\text{damaged}}} \tag{2}$$

where $P_{f(\text{damaged})}$ is the probability of failure for a damaged structural system and $P_{f(\text{intact})}$ is the probability of failure of an intact structural system. The redundancy index provides a measure on the robustness / redundancy of the structural system. The index takes values between zero and infinity, with smaller values indicating larger robustness. β_{intact} is the reliability index of the intact structural system and β_{damaged} is the reliability index of the damaged structural system. The robustness index takes values between zero and infinity, with larger values indicating larger robustness.

By using the redundancy index RI and the structural reliability framework in (2) the effect of ductility in timber structures has been evaluated, see (Kirkegaard et al. 2009). The ductile behavior of joints as well as timber material in compression could have a positive influence on the robustness of timber structures (Brunner 2000; Piazza M. et al. 2004; Stehn and Björnfot 2002; Stehn and Borjes 2004). Timber has no or a very little ductility in the tensile area, while in compression linear elastic-plastic behaviour can be assumed (Glos 1981). Another very important issue is that the joint ductility, elastic displacements, displacements at maximum load and ultimate displacements depend significantly upon the type of the connections used (dowel type fasteners, tooth plates and punched metal plates). There are also significant differences between different dowel type fasteners (bolts, dowels, nails, etc.). Based on these observations levels of ductility D_f = 1, 2, 3, 4 have been studied. The ductility level is given as $D_f = \delta_f / \delta_y$ where δ_y is the yielding dicplacement and δ_f the ultimate displacement. The redundancy index RI versus number of elements for different levels of ductility D_f is estimated based on Monte Carlo simulations where correlation between the strength of structural elements and load models for permanent and live load are introduced according to (JCSS 2001; JCSS 2006). Based on the tentative results it can be concluded that the robustness of a structural timber system can be increased significantly due to ductile behaviour (Kirkegaard et al. 2009).

4 ROBUSTNESS DESIGN OF TIMBER STRUCTURES

Design rules for robustness require insensitivity to local failure and the prevention of progressive collapse. This is often verified by applying the load case "removal of a limited part of the structure". The fact sheet (Dietsch 2009) has evaluated typical secondary systems for timber roof structures against these requirements, including exemplary comparative calculations for typical purlin systems. The results were compared against typical reasons for damages and failure. Applying the finding that most failures of timber structures are not caused by random occurrences or local defects, but by global (repetitive) defects (e.g. from systematic human errors mistakes), it was shown that the objective of load transfer - often mentioned as preferable - should be critically analysed for such structures.

Evaluating purlin systems from a structural perspective will highlight continuous systems due to their lowered maximum bending moments, enabling the realisation of larger spacing at given span and cross-section. Due to this and due to the acceleration of the construction process, the majority of purlin systems today are realized by continuous systems like lap-jointed beams.

The evaluation from a robustness perspective reveals more debatable results. Continuous systems (due to their redundancy and higher stiffness) will result in an increased load transfer in the case of failure of one structural member. Many publications on robustness mention this as preferable. Nevertheless, as recent studies have revealed, are most failures of structures not caused by local defects or random occurrences but by global defects from systematic mistakes or global deterioration, meaning the damaging effects are highly correlated. Such structures are not able to withstand a large load transfer and will therefore be more prone to progressive collapse. This idea is supported in (Starossek 2006), stating that the "alternate load path" approach (realized by e.g. parallel systems) may "in certain circumstances not prevent but rather promote collapse progression". Hence, the idea of compartmentalization is introduced which is realized by a deliberate reduction of continuity at chosen compartment borders. For the systems discussed, this approach might be preferable, if the strength and/or stiffness required for the formation of an alternate load path cannot be guaranteed in case of failure of one element. Two failure examples, both featuring systematic mistakes in design and construction, emphasize this. The Siemens-Arena, having statically determinate secondary members, sustained a partial collapse after the failure of two main beams while the Bad Reichenhall Ice-Arena suffered a progressive collapse triggered by its very stiff and redundant secondary system. These two structures and their particular failure mechanisms with respect to robustness are therefore presented in more detail in (Munch-Andersen and Dietsch 2009).

In summary this means, that there is no strategy for the structural designer, which ensures robustness in all cases. When deciding on a robustness strategy one has to consider different scenarios. The major difference is weather the cause of failure is likely to be a systematic (mostly human) error or an unforeseeable (mostly local) incident. Experience tells that human errors are by far the most common cause. In order to reduce the risk of collapse and in particular progressive collapse, it is crucial to reduce the number of human errors by e.g. enhanced quality control. Only then it would be possible to choose an unambiguously beneficial robustness strategy.

It is the belief that the given statements are valid for the majority of timber structures. However, to put this comparison on a broader foundation, further comparative calculations on other systems should be carried out and the evaluation should also be extended to a probabilistic approach (Munch-Andersen and Dietsch 2009).

5 CONCLUSIONS

The present paper has given an outline of the latest results achieved by working group 3 (WG3) which are dealing with robustness of timber structures. WG3 considers the subtasks: definition of structural robustness of timber structures**,** quantification of robustness and methods of assessing robustness of timber structures as well as methods of designing for robustness of timber structures.

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