Guideline for Design for Robustness of Timber Structures

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2	**Definition of structural robustness** - Hazards: unforeseen loads and defects (incl. material defects); systematic/random, types of actions (permanent, snow, wind, accidental,..), human errors (design, execution) - Consequences - Definition of robustness - Factors affecting robustness: ductility / brittle - Use basic definition of robustness in EN1990	JDS, PHK, Dean
3	**Quantification of robustness and methods of assessing robustness of timber structures** - modeling of ductility/brittleness in timber material and joints - modeling of system effects, gross errors - redundancy vs. robustness - estimation of system reliability - reliability/risk based requirements related to consequences of direct failure consequences and follow-up consequences	PHK, JDS, Dean Cizmar, Goran Turk
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Annex A	**Current requirements in building regulations and codes** EN1990 EN1998 (earthquake) Danish requirements Offshore JCSS ASCE	 JDS Branco JDS JDS JDS Neves
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Summary

To be written

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1 Introduction

To be written

2 Definition of structural robustness

Robustness of structures has been recognized as a desirable property because of a several system failures, such as the Ronan Point Apartment Building in 1968, where the consequences were deemed unacceptable relative to the initiating damage [21]. After the collapse of the World Trade Canter, the robustness has obtained a renewed interest, primarily because of the serious consequences related to failure of the advanced types of structures. In order to mineralize the likelihood of such disproportional structural failures many modern building codes consider the need for robustness in structures and provide strategies and methods to obtain robustness. In fact, in all modern building codes, one can find a statement (in slightly different forms): “total damage (or collapse of a large part of a structure) resulting from a hazard should not be disproportionate to the direct damage caused by this hazard”.

In Eurocode EN1990:2002 the basic requirement to robustness is given in clause 2.1 4(P):
‘A structure shall be designed and executed in such a way that it will not be damaged by events such as:
– explosion,
– impact, and
– the consequences of human errors,
to an extent disproportionate to the original cause.’

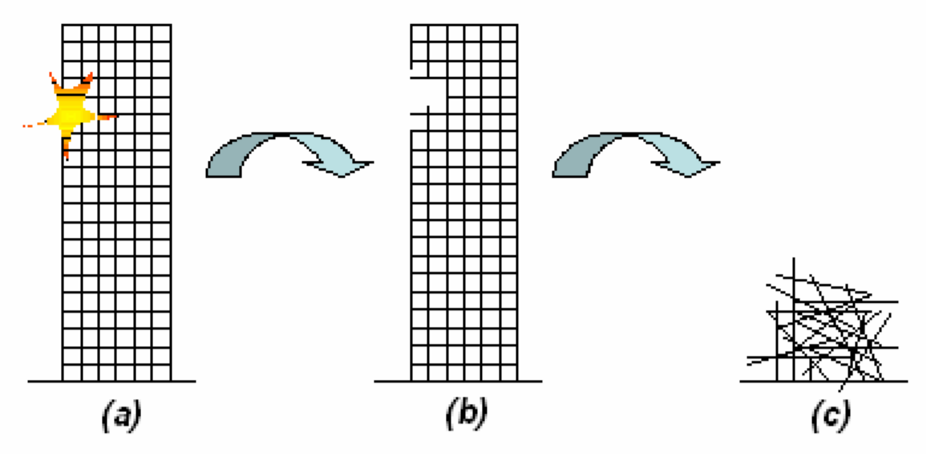


Figure 1. Illustration of the basic concepts in robustness.

This requirement will also in this guideline be used as basis for definition of robustness. Figure 1 illustrates basic aspects / steps in robustness:

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)

Robustness is especially related to precautions to prevent / reduce the indirect consequences in step c) in case of a local damage in step b).

Robustness rules can also be seen as additional rules / requirements to the basic code-specific checking of individual components / failure modes in order to secure that the structure considered as a system has a satisfactory reliability. The system consists of the structure and the environment it is placed in.

Important aspects related to robustness which will be described / discussed in the following are:

- Progressive collapse
- Redundancy
- Ductility

A progressive collapse of a building is defined as a catastrophic partial or total failure that starts from local damage, caused by a certain event/exposure, that can’t be absorbed by the structural system itself. The “normal” or “usual” structural design usually provides a certain amount of additional strength and ductility that is available to withstand abnormal loads and progressive collapse. But, due to “structural revolution” (use of computers, high performance materials and modern building systems) much of the inherent strength is taken out [4, 10]. Progressive collapse is characterised by disproportion between the magnitude of a triggering event and resulting in collapse of large part or the entire structure [20].

During the last decades there has been a significant effort to develop methods to assess robustness and to quantify aspects of robustness. When modelling robustness, system effects are very important. However, the primary criteria in building code are related to design and verification of sufficient reliability of components. It should also be noted that redundancy in systems is closely related to robustness. In principle redundant system are believed to be more robust than non-redundant systems – but this is not always the case as illustrated by the failures of ‘Ballerup super arena’ and ‘Bad Reichenhall icehall’, see annexes B1 and B2.

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:

- A risk-based robustness index based on a complete risk analysis where the consequences are divided in direct and indirect risks
- A probabilistic robustness index based on probabilities of failure of the structural system for an undamaged structure and a damaged structure
- A deterministic robustness index based on structural measures, e.g. pushover load bearing capacity of an undamaged structure and a damaged structure

In the next sections these robustness measure are described.

2.1 Robustness measures

2.1.1 Risk analysis

The basic framework for risk analysis is based on the following equation with risk contributions from local damages (direct consequences) and comprehensive damages (follow-up / indirect consequences), se figure 1, are added, see also JCSS-RA:

$$R = \sum_i \sum_j C_{ij} P\left(D_j \middle| E_i\right) P(E_i) + \sum_k \sum_i \sum_j C_{ijk} P\left(S_k \middle| D_j \cap E_i\right) P\left(D_j \middle| E_i\right) P(E_i) \tag{1}$$

where

C_{ij} consequence (cost) of damage (local failure) D_j due to exposure E_i

$P(E_i)$ probability of exposure E_i

$P\left(D_j \middle| E_i\right)$ probability of damage D_j given exposure E_i

C_{ijk} consequence (cost) of comprehensive damages (follow-up / indirect) S_k given local damage D_j due to exposure E_i

$P\left(S_k \middle| D_j \cap E_i\right)$ probability of comprehensive damages S_k given local damage D_j due to exposure E_i

The optimal design / decision is the one minimizing the risk R. A detailed description of the theoretical basis for risk analysis can be found in JCSS-RA (2008). An important step in the risk analysis is to define the system and the system boundaries.

The total probability of comprehensive damages / collapse associated to (1) is:

$$P(\text{collapse}) = \sum_i \sum_j P\left(\text{collapse} \middle| D_j \cap E_i\right) P\left(D_j \middle| E_i\right) P(E_i) \tag{2}$$

where

$P\left(\text{collapse} \middle| D_j \cap E_i\right)$ probability of collapse (comprehensive damage) given local damage D_j due to exposure E_i.

Note that compared to (1) only one comprehensive damage (collapse) is included in (2).

For damages related to key elements the probability of collapse is $P(\text{collapse} | D_j \cap E_i) \approx 1$. From equation (2) it is obvious that the probability of collapse can be by:

- Reducing one or more of the probabilities of exposures $P(E_i)$ - prevention of exposure / event control
- Reducing one or more of the probabilities of damages $P(D_j | E_i)$ - related to element behaviour
- Reducing one or more of the probabilities $P(\text{collapse} | D_j \cap E_i)$

If the consequences are included in a risk analysis then also reduction of direct (local) consequences, C_{ij} and comprehensive (indirect) consequences, C_{ijk} are important.

According to the description above and the robustness definition in EN1990, robustness is especially related to the reducing the probability $P(\text{collapse}|D_j \cap E_i)$. Increasing the robustness at the design stage will in many cases only increase the cost of the structural system marginally – the key point is often to use a reasonable combination of a suitable structural system and materials with a ductile behaviour. In other cases increased robustness will influence the cost of the structural system.

2.1.2 Risk-based robustness index

Baker, Schubert and Faber, [21] proposed a definition of a robustness index. The approach divides consequences into direct consequences associated with local component damage (that might be considered proportional to the initiating damage) and indirect consequences associated with subsequent system failure (that might be considered disproportional to the initiating damage), [13]. An index is formulated by comparing the risk associated with direct and indirect consequences. The index of robustness (I_{Rob}) is defined as

$$I_{rob} = \frac{R_{Dir}}{R_{Dir} + R_{Ind}} \tag{3}$$

where R_{Dir} and R_{Ind} are the direct and indirect risks associated with the first and the second term in equation (1). The index takes values between zero and one, with larger values indicating larger robustness. This method for assessing robustness is based on a risk assessment framework proposed by Joint Committee on Structural Safety (JCSS). The assessment begins with the consideration and modelling of exposures (*EX*) that can cause damage to the components of the structural system, see figure 3. Term “exposures” refers on extreme values of design loads, accidental loads and deterioration processes but also includes human errors in the design, execution and use of the structure. Term “damage” refers to reduced performance or failure of individual components of the structural system. After the exposure event occurs, the components of the structural system either remain in an undamaged state ($\overline{D}$) as before or change to a damage state (D). Each damage state can then either lead to the failure of the structure (F) or no failure ($\overline{F}$).

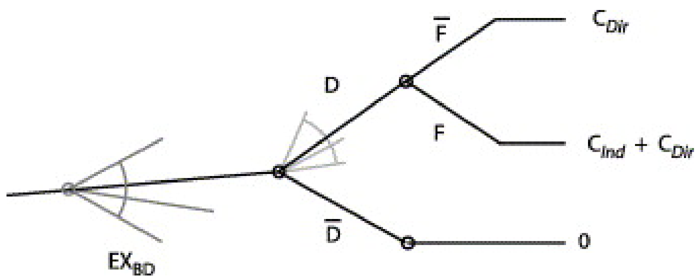


Figure 3. An event tree for robustness quantification, [21]

As stated above, consequences are associated with each of the possible damage and failure scenarios, and are classified as either direct (C_{Dir} = C_{ij}) indirect (C_{Ind} = C_{ijk}). Direct consequences are considered to result from damage states of individual component(s). Indirect consequences are incurred due to loss of system functionality or failure and can be attributed to lack of robustness [21].

Remark: as mentioned above the optimal design / decision is the one which minimizes the total risk obtained by equation (1). This could equally well be by reducing the first or the second term in equation (1). This implies that the definition of a robustness index by equation (3) is not always fully consistent with a full risk analysis. – to be discussed!

2.1.3 Reliability-based robustness index

In the early 90's Frangopol and Curley [22] proposed some probabilistic measures related to structural redundancy – which also indicates the level of robustness. A redundancy index (RI) is defined by:

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

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

They also considered the following redundancy factor:

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

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

Lind [23] proposed a generic measure of system damage tolerance, based on the increase in failure probability resulting from the occurrence of damage. The vulnerability (V) of a system is defined as:

$$V = \frac{P(r_d, S)}{P(r_o, S)} \tag{6}$$

where r_d is the resistance of the damaged system, r_0 is the resistance of the undamaged system, and S is the prospective loading on the system $P(\cdot)$ is the probability of failure of the system, as a function of the load and resistance of the system. Vulnerability parameter indicates the loss of system reliability due to damage.

2.1.4 Deterministic robustness index

A simple and practical measure of structural redundancy (and robustness) used in the offshore industry is given in [28] based on the so-called *RIF* –value (Residual Influence Factor).

A reserve strength ratio (RSR) is defined as:

$$RSR = \frac{R_c}{S_c} \tag{7}$$

where R_c denotes characteristic value of the base shear capacity of an offshore platform (typically a steel jacket) and S_c is the design load corresponding to ultimate collapse.

In order to measure the effect of full damage (or loss of functionality) of structural member no *i* on the structural capacity the so-called *RIF* –value (sometimes referred to as the Damaged Strength Ratio) is defined:

$$RIF_i = \frac{RSR_{Fi}}{RSR_{\text{intact}}} \tag{2}$$

where RSR_{intact} is the *RIF*-value of the intact structure and RSR_{Fi} is the *RIF*-value of the structure where member no *i* is failed/removed. The *RIF* can vary between 0 and 1, where the larger *RIF* stand for a more robust structure.

Another simple measure of robustness is proposed in [20] by considering:

$$R_s = \min_j \frac{\det K_j}{\det K_0} \tag{3}$$

where K_j and K_o are system stiffness matrix of the intact structure and stiffness matrix after the removal a structural element or a connection *j*, respectively. However, it seems that this robustness measure is not sufficient in this form [20]. Same authors also proposed an energy based measure of robustness and damage based measure of robustness. Energy based measure is defined as:

$$R_s = 1 - \max_j \frac{E_{r,j}}{E_{s,k}} \tag{4}$$

where $E_{r,j}$ is amount of energy released by the initial failure of a structural element *j* and available energy for the damage of the next structural element *k*, while $E_{s,k}$ is the energy required for the failure of the next structural element.

A damage based measure of robustness is defined as:

$$R_d = 1 - \frac{p}{p_{\text{lim}}}$$

where *p* is maximum extent of the damage caused by initial damage i_{lim} and p_{lim} is acceptable damage progression [20]. In order to quantify the damage extent the corresponding masses, volumes, floor areas can be used.

2.2 Robustness in building codes

In the following examples of robustness requirements in codes are described.

2.2.1 Eurocodes

Robustness requirements are described in two Eurocode parts: EN 1990: 'Basis of Structural Design' [11] and EN 1991 Part 1-7 'Accidental Actions' [12], see annex A1. The first document provides the general 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 provides strategies and methods to obtain robustness. Actions that should be considered in different design situations are, see figure 2: 1) designing against identified accidental actions, and 2) designing unidentified actions (where designing against disproportionate collapse, or for robustness, is important). The methods used to design for robustness of a structure are divided into several levels based on potential consequences of structural failure (Consequence Class). CC1 represents low consequence class with no special requirements, CC2 are structures with medium consequences that can be handled using simplified analysis, while CC3 stands for high consequence class where a reliability or risk analysis must be conducted [13]. However, there is no specific criteria which could be used to quantify the level of robustness of a structure which could have a benefit for design and analysis of structures.

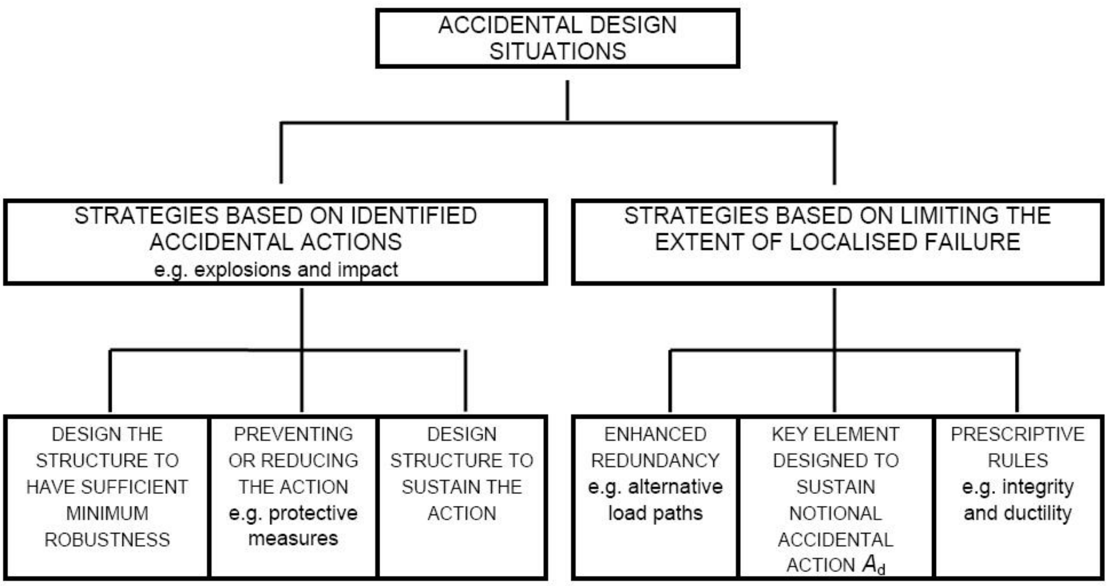


Figure 2. Design situations according to EN 1991-1-7.

2.2.2 JCSS – Probabilistic Model Code

In the Probabilistic model code [2] robustness requirement is also formulated as: “A structure shall not be damaged by events like fire, explosions or consequences of human errors, deterioration effects, etc. to an extend disproportionate to the severeness of the triggering event”, see also annex A5. In order to attain adequate reliability in relation with accidental loads, two basic strategies are proposed: non-structural (prevention, protection and mitigation) and structural measures (making the structure strong enough to withstand the loads limiting the amount of structural damage or limiting the amount of structural damage).

2.2.3 Danish robustness requirements

According to Danish design rules, robustness shall be documented for all structures where consequences of failure are serious, see annex A3. Robustness is related to scenarios where exposures result in damage to structural system. This means that a robust structure can be achieved by means of suitable choices of materials, general static layout and structural composition, and by suitable design of key elements. Robustness should be distinguished from accidental loads although some of the design procedures and measures are similar; structures should be robust regardless of the likelihood of accidental loads. A key element is defined as a limited part of the structure, which has an essential importance for the robustness of the structure such that any possible failure of the key element implies a failure of the entire structure or significant parts of it [4, 16, 17]. Examples of unintentional loads and defects are e.g. unforeseen load effects, geometrical imperfections, settlements and deterioration, unintentional deviations between the actual function of the structure and the applied computational models and between the executed project and the project material. The requirements to robustness of a structure should be related to the consequences of a failure of the structure. Therefore documentation of robustness is only required for structures in high consequence class.

Robustness is assessed by preparation of a technical review where at least one of the following criteria shall be fulfilled:

a) by demonstrating that those parts of the structure essential for the reliability only have little sensitivity with respect to unintentional loads and defects
b) by demonstrating a load case with ‘removal of a limited part of the structure’ in order to document that an extensive failure of the structure will not occur if a limited part of the structure fails
c) by demonstrating sufficient safety of key elements, such that the entire structure with one or more key elements has the same reliability as a structure where robustness is documented by b

The design procedure to document sufficient robustness can be summarized in the following steps:

1. Review of loads and possible failure modes/scenarios and determination of acceptable collapse extent
2. Review of the structural systems and identification of key elements
3. Evaluation of the sensitivity of essential parts of the structure to unintentional loads and defects
4. Documentation of robustness by ‘failure of key element’ analysis
5. Documentation of robustness by increasing the strength of key elements
   if Step 4 is not possible.

3 Quantification of robustness and methods of assessing robustness of timber structures

In the last few decades there have been intensely research concerning reliability of timber structures but consensus on the general characteristics of timber systems regarding redundancy and robustness has not yet been established. Timber material is a complex building material where several factors such as size effects, ductile/brittle behaviour, moisture effects and creep, low strength perpendicular to grain and system effects are pronounced and could be important for quantification of robustness of timber structures. An important aspect for the assessment of the performance of timber structures is the interaction of structural components in structural systems. System effects in timber structures are pronounced because of multiscale spatial variability of environmental exposures and material properties. It should also be noted that redundancy in systems is closely related to robustness. In principle redundant systems are believed to be more robust than non-redundant systems – but this is not always case as illustrated by the failures of the timber structures 'Ballerup super arena' and 'Bad Reichenhall icehall', see annexes B1 and B2. Also redistribution of load effects and possible gross errors, i.e. unintentional load and defects could have essential influence on the robustness. For the assessment of robustness of timber structures existing numerical methods used to assess the reliability of timber structures need to be evaluated for their possible application, and simplified approaches suitable for day-to-day engineering purposes must be identified. To reach a better understanding with respect to quantification of robustness and methods of assessing robustness timber structures the following issues are considered:

- modeling of ductility/brittleness in timber material and joints
- modeling of system effects, gross errors
- redundancy vs. robustness
- estimation of system reliability
- reliability/risk based requirements related to consequences of direct failure consequences and follow-up consequences

In order to discuss the modelling of issues related to the robustness of timber structures we need consensus on the general characteristics of timber systems regarding robustness. The following section outlines these characteristics based on an analysis of failed timber structures.

3.1 Robustness evaluation of failed timber structures

For the purpose of the project „Timber Frame 2000" [24] a six-storey experimental timber frame building was erected, in order to investigate the performance and economic prospects of medium-rise timber frame buildings in the UK. As a part of a testing programme the investigation of disproportionate collapse (robustness) was conducted. This evaluation is to verify that the inherent stiffness of cellular platform timber frame construction can provide the necessary robustness so that, in the event of an accident, the building will not suffer collapse to an extent disproportionate to the cause [24]. This is achieved by designing in such a way that a beam, column or section of wall can be removed without the structure above collapsing (although damage to the building is allowed). To achieve this, beams are incorporated within floor depths over external walls, or the walls themselves are made to act as beams. The building was loaded with sandbags positioned on each floor. Based on an analytical review of the building, agreed serviceability requirements and defined rules 'worst case scenario' is chosen for the test. Result obtained show that this kind of timber frame system is very robust.

Figure 5. Test of timber frame

3.1.1 Evaluation of wide span timber structures

A section with results from the two large scale TUM projects where 109 structures were evaluated.
……

3.1.2 Evaluation of timber structures

A section with results from the Scandinavian project presented in the report: *Design of safe timber structures –How can we learn from structural failures in concrete, steel and timber?*

….

3.1.3 Secondary Structures - purlins - robustness considerations

A section with results from the two papers:.

What is a robust construction? By *Jørgen Munch-Andersen,*
Secondary Structures - Purlins - Robustness Considerations by Philipp Dietsch

….

3.1.4 Assessment of timber structures

A section with results from WG1….

3.1.5 Conclusion

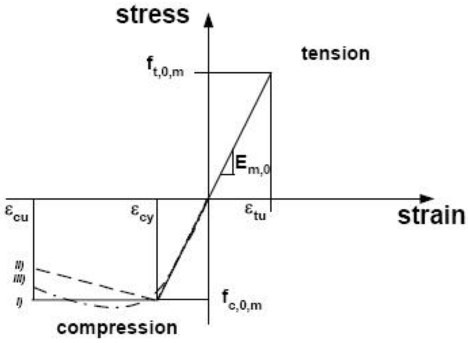
3.2 Robustness of timber structures assuming a ductile behaviour of material and joints

A section with results from the two papers:

Robustness of timber structures with Ductile Material Behaviour – theoretical investi gations' by PHK,JDS,DC
Robustness of timber structures with Ductile Material Behaviour – numerical example by DC,PHK,JDS

3.2.1 Ductile/Brittle behaviour material

Timber is considered to be a brittle material, because failure occurs suddenly, without any warning. This can be considered as an obstacle when comparing to other materials like steel. It has no or a very little ductility in the tensile area, while in compressive area linear elastic-plastic behaviour can be assumed. [7]



Typical stress strain curve of timber

…...the present section will be reformulated and extended…

3.2.2 Ductile/Brittle behaviour of connections/joints

In the aspect of timber joints all agree that the way to achieve high ductility is to take advantage of the plasticity of mechanical connectors (nails, dowels, bolts, etc.) The only certain way to create ductile structure is design in which collapse of a structure is governed by failures of mechanical joints [8]. This is especially important for the seismic behaviour of a timber structure. The definition of ductility, with respect to behaviour in joints, is:

$$D_f = \frac{u_f}{u_y}$$

where u_f denotes the deformation at which the connection looses stability and u_y is the elastic deformation [9].

…...the present section will be reformulated and extended…

3.2.3 Conclusion

3.3 Redundancy vs. Robustness

Any mechanical system may 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 [14]. If a system does not satisfy these strict definitions of ‘‘series’’ or ‘‘parallel’’ systems, the system is classified as a hybrid system model.

3.4 System modelling of timber structures

A section with results from the papers where different cases are evaluated:.

Robustness Assessment of Timber Structures by PHK,JDS
…case2…
…case3…
………

…...the present section will be reformulated and extended…

3.5 Conclusion

…...the present section will be reformulated and extended…

4 Methods of designing for robustness of timber structures

- Categories of robustness:
  - Consequence classes
  - Conventional / new, innovative structure (design and production)
  - Key elements

5 Effect of quality control

- Monitoring requirements (e.g. for in-plane and out-of-plane deformations, cracks, moisture)
- Incl. maintenance

6 Recommendations

- for code requirements/modification, EN1995
- for future R&D

Annex A. Current requirements in building regulations and codes

In this annex robustness requirements in the following codes briefly summarised:

Annex A1: EN1990: Basis of structural design
Annex A2: EN1998-1: (earthquake)
Annex A3: Robustness rules in Danish National Annex to EN1990
Annex A4: Offshore
Annex A5: JCSS
Annex A6: ASCE

Annex A1. Eurocodes: EN1990 and EN1991-1-7

In this annex the basic Eurocode requirements to robustness in EN1990 and EN1991-1-7 are presented. In Gulvanesian & Vrouwenvelder (2006) the background for the requirements is described. The following text is an extract from this description:

'The objective of design in general is to reduce risks at an economical acceptable price. Risk may be expressed in terms of the probability and the consequences of undesired events. Thus, risk-reducing measures consist of probability reducing measures and consequence reducing measures. No design, however, will be able or can be expected to counteract all actions that could arise due to an extreme cause, thus a structure should not be damaged to an extent disproportionate to the original cause. As a result of this principle, given in 4(P) of EN 1990, local failure may be accepted. For that reason, redundancy, and non-linear effects play a much larger role in design for accidental actions, than in the case of variable actions. Design for accidental design situations needs to be primarily included for structures for which a collapse may cause particularly large consequences in terms of injury to humans, damage to the environment or economic losses for the society. A convenient measure to decide what structures are to be designed for accidental situations is to arrange structures or structural components in categories according to the *consequences* of an accident.

The design for unidentified accidental load is presented in Annex A of EN1991-1-7. Rules of this type were developed from the UK Codes of Practice and regulatory requirements introduced in the early 1970s following the partial collapse of a block of flats at Ronan Point in east London caused by a gas explosion. The rules have changed little over the intervening years. They aim to provide a minimum level of building robustness as a means of safeguarding buildings against a disproportionate extent of collapse following local damage being sustained from an accidental event.'

Section A1.1 contains the main robustness requirements in EN1990. Section A1.2 contains the robustness requiremenst in EN1991-1-7 related to 'Design for consequences of localised failure in buildings from an unspecified cause'

A1.1 Robustness in 'EN1990:2002 Basis of structural design'

Section 2 Requirements
2.1 Basic requirements

(4)P A structure shall be designed and executed in such a way that it will not be damaged by events such as:
– explosion,
– impact, and
– the consequences of human errors,
to an extent disproportionate to the original cause.

NOTE 1 The events to be taken into account are those agreed for an individual project with the client and the relevant authority.
NOTE 2 Further information is given in EN 1991-1-7.

(5)P Potential damage shall be avoided or limited by appropriate choice of one or more of the following :

- 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 localised damage ;
- avoiding as far as possible structural systems that can collapse without warning ;
- tying the structural members together.

(6) The basic requirements should be met:
- by the choice of suitable materials,
- by appropriate design and detailing, and
- by specifying control procedures for design, production, execution, and use relevant to the particular project.

(7) The provisions of Section 2 should be interpreted on the basis that due skill and care appropriate to the circumstances is exercised in the design, based on such knowledge and good practice as is generally available at the time that the design of the structure is carried out.

A1.2 Robustness in 'EN1991-1-7:2005 Accidental actions'

Annex A (informative)
Design for consequences of localised failure in buildings from an unspecified cause

A.1 Scope and field of application

(1) This Annex A gives rules and methods for designing buildings to sustain an extent of localised failure from an unspecified cause without disproportionate collapse. Whilst other approaches may be equally valid, adoption of this strategy is likely to ensure that a building, depending upon the consequences class (see 3.4), is sufficiently robust to sustain a limited extent of damage or failure without collapse.

A.2 Introduction

(1) Designing a building such that neither the whole building nor a significant part of it will collapse if localised failure were sustained, is an acceptable strategy, in accordance with Section 3 of this part.
Adopting this strategy should provide a building with sufficient robustness to survive a reasonable range of undefined accidental actions.

(2) The minimum period that a building needs to survive following an accident should be that period needed to facilitate the safe evacuation and rescue of personnel from the building and its surroundings. Longer periods of survival may be required for buildings used for handling hazardous materials, provision of essential services, or for national security reasons.

A.3 Consequences classes of buildings

(1) Table A.1 provides a categorisation of building types/occupancies to consequences classes This categorisation relates to the low, medium and high consequences classes given in 3.4 (1).

Table A.1 - Categorisation of consequences classes.

Consequence	Example of categorisation of building type and occupancy
1	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1'/:, times the building height.
2a Lower Risk Group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys Retailing premises not exceeding 3 storeys of less than 1 000 m^2 floor area in each storey. Single storey educational buildings All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000 rrr at each storey.
2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys Retailing premises greater than 3 storeys but not exceeding 15 storeys Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000 m^2 but not exceeding 5000 m^2at each storey. Car parking not exceeding 6 storeys.
3	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5 000 spectators Buildings containing hazardous substances and /or processes

NOTE 1 For buildings intended for more **than** one type of use the "consequences class" should be that relating to the most onerous **type.**

NOTE 2 **In** determining the number of storeys basement storeys may be excluded provided such basement storeys fulfil the requirements of "Consequences Class 2b Upper Risk Group".

NOTE 3 Table A.1 is not exhaustive and can be adjusted

A.4 Recommended strategies

(1) Adoption of the following recommended strategies should provide a building that will have an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse.

a) For buildings in Consequences Class 1:

Provided a building has been designed and constructed in accordance with the rules given in EN 1990 to EN 1999 for satisfying stability in normal use, no further specific consideration is necessary with regard to accidental actions from unidentified causes.

b) For buildings in Consequences Class 2a (Lower Group):
In addition to the recommended strategies for Consequences Class 1, the provision of effective horizontal ties, or effective anchorage of suspended floors to walls, as defined in A.5.1 and A.5.2 respectively for framed and load-bearing wall construction should be provided.

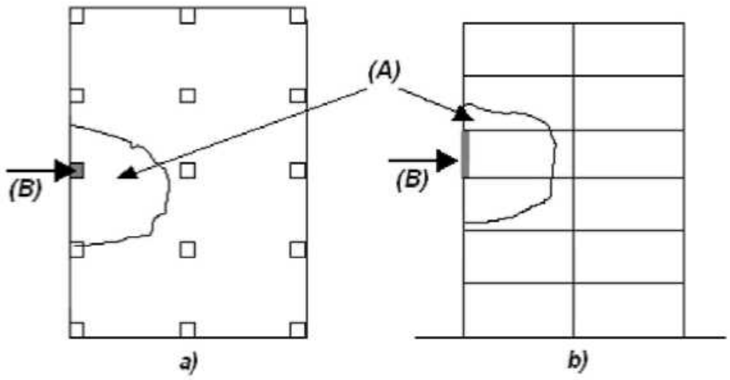
NOTE 1 Details of effective anchorage may be given in the National Annex

c) For buildings in Consequences Class 2b (Upper Group):
In addition to the recommended strategies for Consequences Class 1. the provision of:
effective horizontal ties, as defined in A.5.1 and A.5 2 respectively for framed and load-bearing wall construction (see 1.5.11), together with effective vertical ties, as defined in A.6, in all supporting columns and walls should be provided, or alternatively,
the building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall as defined in A.7 (one at a time in each storey of the building) the building remains stable and that any local damage does not exceed a certain limit
Where the notional removal of such columns and sections of walls would result in an extent of damage in excess of the agreed limit, or other such limit specified, then such elements should be designed as a "key element" (see A.8).
In the case of buildings of load-bearing wall construction, the notional removal of a section of wall, one at a time, is likely to be the most practical strategy to adopt.

d) For buildings in Consequences Class 3:
A systematic risk assessment of the building should be undertaken taking into account both foreseeable and unforeseeable hazards.

NOTE 2 Guidance on the preparation of a risk analysis is included in Annex B

NOTE 3 The limit of admissible local failure may be different for each type of building. The recommended value is 15 % of the floor, or 100 m', whichever is smaller, in each of two adjacent storeys See Figure A. 1



Key
(A) Local damage not exceeding 15 % of floor area in each of two adjacent storeys
(B) Notional column to be removed
a) Plan b) Section
Figure A.1 - Recommended limit of admissible damage

A.5 Effective horizontal ties
A.5.1 Framed structures

(1) Effective horizontal ties should be provided around the perimeter of each floor and roof level and internally in two right angle directions to tie the column and wall elements securely to the structure of the building. The ties should be continuous and be arranged as closely as practicable to the edges of floors and lines of columns and walls. At least 30 % of the ties should be located within the close vicinity of the grid lines of the columns and the walls

NOTE See the example in Figure A 2

(2) Effective horizontal ties may comprise rolled steel sections, steel bar reinforcement in concrete slabs,or steel mesh reinforcement and profiled steel sheeting in composite steel/concrete floors (if directly connected to the steel beams with shear connectors). The ties may consist of a combination of the above types.

(3) Each continuous tie, including its end connections, should be capable of sustaining a design tensile load of *"T"* for the accidental limit state in the case of internal ties, and "7",," , in the case of perimeter ties, equal to the following values:

for internal ties $T_i = 0{,}8(g_k + \psi q_k)sL$ or 75 kN, whichever is the greater.(A.1)

for perimeter ties $T_p = 0{,}4(g_k + \psi q_k)sL$ or 75 kN, whichever is the greater.(A.2)

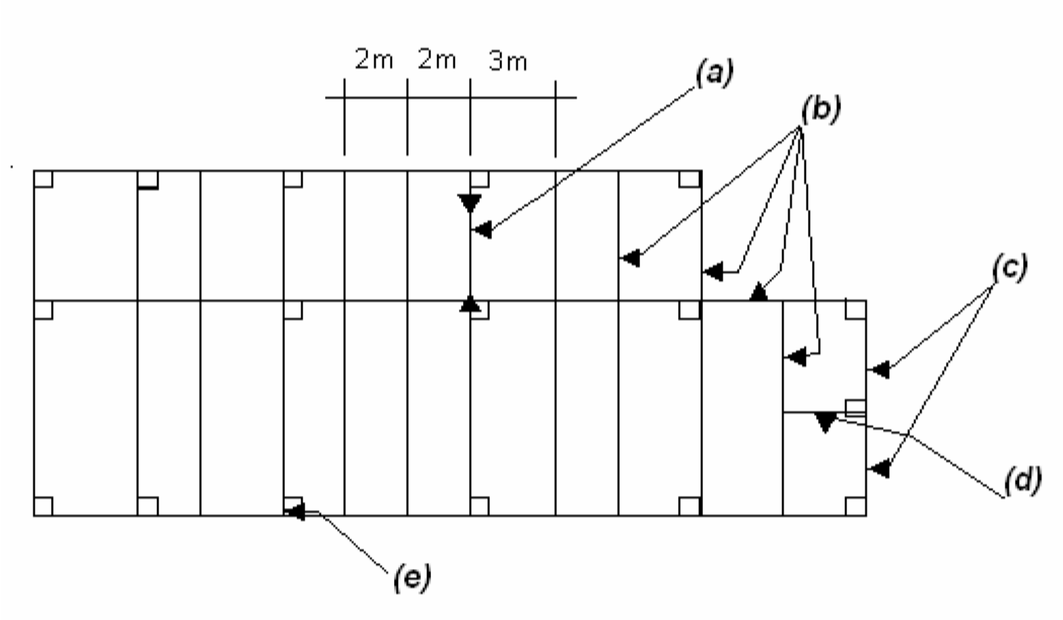
where :

s is the spacing of ties,

L is the span of the tie,

ψ is the relevant factor in the expression for combination of action effects for the accidental design situation (i.e. ψ_1 or ψ_2 in accordance with expression (6.11b) of EN 1990).

NOTE See the example in Figure A.2.



Key

(a) 6 m span beam as internal tie

(b) All beams designed to act as ties

(c) Perimeter ties

(d) Tie anchoring column

(e) Edge column

EXAMPLE The calculation of the accidental design tensile force T_i in the 6 m span beam shown in Figure A.2 assuming the following characteristic actions (e.g. for a steel frame building).

Characteristic loading : g_k = 3,0 kN/m^2 and q_k = 5,0 kN/m^2

And assuming the choice of combination coefficient ψ_1 (i.e. = 0,5) in expression (6.11a)

$$T_i = 0{,}8(3{,}00 + 0{,}5 \times 5{,}00)\,\frac{3+2}{2} \times 6{,}0 = 66\ kN \text{ (being less than 75 kN)}$$

Figure A.2 - Example of effective horizontal tying of a 6 storey framed office building.

(4) Members used for sustaining actions other than accidental actions may be utilised for the above ties.

A.5.2 Load-bearing wall construction

(1) For Class 2 buildings (Lower Risk Group), see Table A.1:

Appropriate robustness should be provided by adopting a cellular form of construction designed to facilitate interaction of all components including an appropriate means of anchoring the floor to the walls.

(2) For Class 2 buildings (Upper Risk Group), see Table A.1:

Continuous effective horizontal ties should be provided in the floors. These should be internal ties distributed throughout the floors in both orthogonal directions and peripheral ties extending around the perimeter of the floor slabs within a 1,2 m width of the slab. The design tensile load in the ties should be determined as follows:

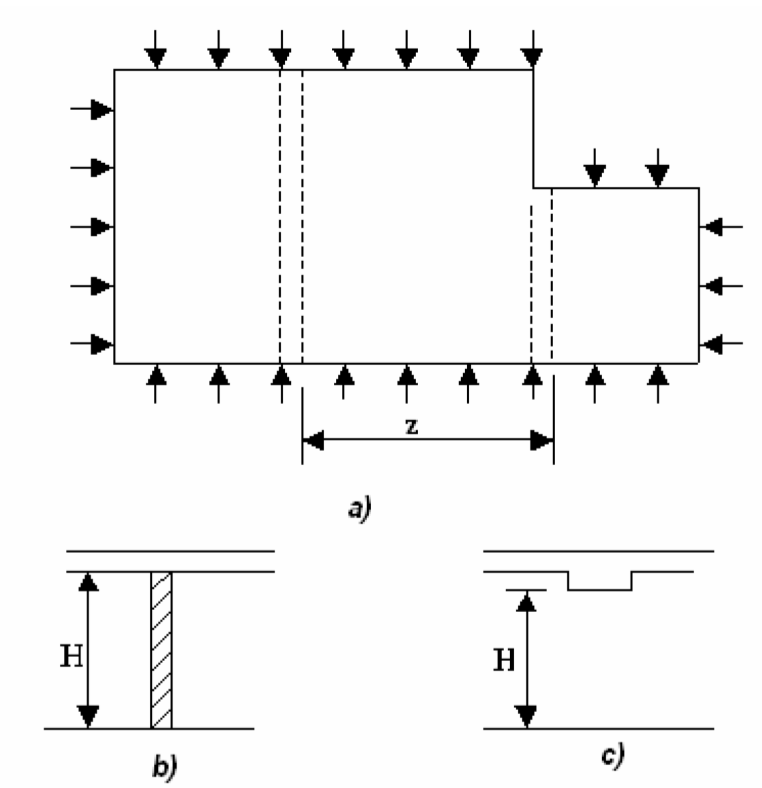
For internal ties T_I = the greater of F_t kN/m or $\frac{F_t(g_k + \psi q_k)}{7,5}\frac{z}{5}$ kN/m(A.3)

For peripheral ties $T_p = F_t$ (A.4)

where :

F_t is 60 kN/m or 20 + 4n_s kN/m, whichever is less
n_s is the number of storeys
z is the lesser of:
- 5 times the clear storey height H, or
- the greatest distance in metres in the direction of the tie, between the centres of the columns or other vertical load-bearing members whether this distance is spanned by:
  - a single slab or
  - by a system of beams and slabs.

NOTE Factors H (in metres) and z are illustrated in Figure A.3.



Key

a) Plan

b) Section: flat slab

c) Section : beam and slab

Figure A.3 – illustration of factors *H* and *z*.

A.6 Effective vertical ties

(1) Each column and wall should be tied continuously from the foundations to the roof level.

(2) In the case of framed buildings (e.g. steel or reinforced concrete structures) the columns and walls carrying vertical actions should be capable of resisting an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. Such accidental design loading should not be assumed to act simultaneously with permanent and variable actions that may be acting on the structure.

(3) For load-bearing wall construction (see 1.11.1) the vertical ties may be considered effective if:

a) for masonry walls their thickness is at least 150 mm thick and if they have a minimum compressive strength of 5 N/mm^2 in accordance with EN 1996-1-1.

b) the clear height of the wall, H, measured in metres between faces of floors or roof does not exceed $20t$, where t is the thickness of the wall in metres.

c) if they are designed to sustain the following vertical tie force T :

$$T = \frac{34A}{8000}\left(\frac{H}{t}\right)^2 \text{ N, or 100 kN/m of wall, whichever is the greater,(A.5)}$$

where:

A is the cross-sectional area in mm^2 of the wall measured on plan, excluding the non loadbearing leaf of a cavity wall.

d) the vertical ties are grouped at 5 m maximum centres along the wall and occur no greater than 2,5 m from an unrestrained end of the wall.

A.7 Nominal section of load-bearing wall

(1) The nominal length of load-bearing wall construction referred to in A.5.2 should be taken as follows:

– for a reinforced concrete wall, a length not exceeding 2,25H,

– for an external masonry, or timber or steel stud wall, the length measured between lateral supports provided by other vertical building components (e.g. columns or transverse partition walls),

– for an internal masonry, or timber or steel stud wall, a length not exceeding 2,25H

where:

H is the storey height in metres.

A.8 Key elements

(1) In accordance with 3.3(1)P, for building structures a "key element", as referred to in A.4(1)c, should be capable of sustaining an accidental design action of A_d applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components having regard to the ultimate strength of such components and their connections. Such accidental design loading should be applied in accordance with expression (6.11b) of EN 1990 and may be a concentrated or distributed load.

NOTE The recommended value of A_d for building structures is 34 kN/m^2.

Annex A2. Eurocodes: EN1998: Design of structures for earthquake resistance

According to Eurocode 8 (EN1998) the following aspects are important for design of structures exposed to earthquakes:

- Structural simplicity, uniformity and symmetry
- Bi-directional and torsional resistance
- Ductility
- Redundancy which allows for redistribution of actions and widespread energy dissipation across the structure - strong columns / weak beams principle
- Diaphamic action of floors

Redundancy requirements are described in is clause 5.2.3.5:

(1)P A high degree of redundancy accompanied by redistribution capacity shall be sought, enabling a more widely spread energy dissipation and an increased total dissipated energy. Consequently structural systems of lower static indeterminacy shall be assigned lower behaviour factors (see Table 5.1). The necessary redistribution capacity shall be achieved through the local ductility rules given in **5.4** to **5.6**.

An important descriptor of the behaviour is the ‘behaviour factor’ defined by:

Factor used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures

The ‘behaviour factor’ includes the effect of ductility through the ductility class used to classify different structures with respect to their ability to have ductile failure modes. Further the ‘behaviour factor’ includes the effect of regularity and failure mode type.

Annex A3. Denmark – Robustness requirements in national annex to EN1990

The robustness requirement used in Denmark is formulated in an informative annex in the Danish National Annex to Eurocode EN1990. Below is shown the text from this informative annex with non-conflicting, additional information.

Annex E (informative) - Additional rules for robustness
This Annex can be used by the examination of robustness, see 2.1.4(P) – 2.1.5(P).

(1) A structure is robust if:
- the parts of the structure that are decisive for the safety are only slightly sensitive to unintended effects and defects; or
- there is no extensive failure of the structure if a limited part of the structure fails.

(2) The following are examples of unintended effects and defects:
- unforeseen action effects;
- 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 subsidence;
- unforeseen degeneration.

Increased robustness can in certain cases also help to reduce the effects of any gross errors, although demonstration of robustness neither can nor may be regarded as designing against gross error.

(3) Robustness is discussed in more detail in DS/INF 146 Robustness - Background and principles (available in Danish only).

(4) The robustness of a structure shall be proportional to the consequences of a failure of the structure. Documentation of robustness is only required for structures in consequences class CC3. However, for structures in consequences class CC2 an assessment of the robustness shall be made. The amount of detail into which the assessment goes shall be increased in the case of large spans, large concentrated loads, few supports and special (rare or new) types of construction.

(5) A robust construction is achieved by an appropriate choice of materials, general static principle and construction and by appropriate design of key members. A key member is a restricted part of the structure which, in spite of its limited extent, is of central importance to the robustness of the structure such that failure of this member would result in the failure of the whole structure or significant parts of the structure.

(6) Where there is a requirement for the robustness to be documented, an expert engineering report shall be drawn up demonstrating that at least one of the robustness criteria specified in (1) is met, i.e. either:

- by demonstrating that the essential parts of the structure, i.e. key members are only slightly sensitive to unintended effects and defects, cf. (2); or
- by demonstrating that no extensive failure of the structure occurs if a limited part of the structure fails (loss of a member), see (7) – (8); or
- by demonstrating adequate safety of key members, such that the whole structure to which they belong attains at least the same level of system safety as an equivalent structure for which the robustness is documented by demonstration of adequate safety in the event of the "loss of a member".

The expert engineering report shall, in addition to the demonstration itself, contain a critical evaluation of the construction, including identification of key members and action scenarios.

Verification of the first criteria is only possible in special cases; therefore verification normally is done using one of the two last-mentioned criteria.

(7) Where robustness is demonstrated in the event of "loss of a member", the acceptable degree of collapse for multi-storey buildings with up to 15 storeys should be taken as: 15% of the floor area on two adjacent storeys in the event of the loss of a member as defined in (8), but not more than 240 m2 per storey and not more than 360 m2 in total. Adequate resistance should be demonstrated in an accidental design situation by using the formula (6.11 a/b), see Table A1.3.

(8) Robustness demonstrated in the event of "loss of a member" may, for residential and grandstand structures, be regarded as met if it can be demonstrated that the damaged structure will continue to constitute a stable system even if one or more structural members are lost. It is assumed that failure may comprise the equivalent of the accepetable permissible degree of collapse, cf. (7), including:

- either a floor or roof structure and an arbitrary pillar;
- or a foor or floor structure and an arbitrary piece of wall 3 m in length or width.

A structure's ability to retain its coherence after a failure of the specified extent is primarily conditional upon the damaged structure continuing to constitute a stable system and the structure or large parts of it not being transformed into a mechanism. If this condition is met, a rough calculation will be sufficient.

(9) Where robustness is verified by the introduction of an increased safety factor for key members, this may usually be done by using a material partial factor, _M, corresponding to the value stated in 6.3.5 increased by a factor of 1,2. In terms of a model, this corresponds to a system with key members in series having the same level of system safety as a system with members in parallel.

As a general rule, every effort should be made in the design to document the robustness of a structure as far as possible without the use of increased safety factors on the key members. Where increased safety factors are used on the key members, it should however be ensured that the resistance of the structure to unintended effects and defects is actually increased.

NOTE – For example, the robustness of hinged pillars in a residential building will not generally be sufficiently ensured by applying a factor of 1,2, unless at the same time a structural connection is arranged through each storey partition in the form of a continuous tensile and shear connector in the pillar.

(10) The structural Eurocodes may provide guidelines for adequately ensuring robustness.

Annex A4. Offshore – Robustness requirement in ISO 19902

This section describes robustness related requirements in 'ISO19902: Petroleum and Natural Gas Industries — Fixed Steel Offshore Structures'

3.2.10
robustness
Ability of a *structure* to withstand events with a reasonable likelihood of occurring without being damaged to an extent disproportionate to the original cause.

7.9 Robustness
A structure shall incorporate robustness through consideration of the effects of all hazards and their probabilities of occurrence, to ensure that consequent damage is not disproportionate to the cause. Damage from an event with a reasonable likelihood of occurrence shall not lead to complete loss of integrity of the structure. In such cases the structural integrity in the damaged state shall be sufficient to allow a process system close down and/or a safe evacuation.

Robustness is achieved by either:

a) Designing the structure in such a way that any single load bearing element exposed to the hazard can become incapable of carrying its normal design load without causing collapse of the structure or any significant part of it;

or

b) Ensuring (by design or by protective measures) that no critical component exposed to the hazard can be made ineffective.

For structures in exposure levels 1 and 2 (L1 and L2), ship impact shall be evaluated (see 10.3).

A.7.9 Robustness
The robustness concept is closely related to accidental actions, consequences of human error, and failure of equipment. Following ISO 19900 these situations are denoted 'hazardous circumstances' or briefly 'hazards'. Robustness is also important in the event of serious but unidentified fatigue damage.

Robustness is achieved by considering accidental limit states that represent the structural effects of hazards. Ideally all such likely hazards should be identified and quantified by means of rational analyses. However, in many cases it is possible based on experience and engineering judgement to identify and reasonably quantify the most important accidental limit states. They will often be those from ship impact, dropped objects, explosions and fires.

The design shall follow ISO 19900 which uses the following approach:

- careful planning of all phases of development and operation;
- avoiding the structural effects of the hazards by either eliminating the source or by bypassing and overcoming them;
- minimising the consequences, or
- designing for hazards.

When the hazard cannot reliably be avoided the designer has a choice between minimising the consequences (i.e. the consequences of loosing an element due to a hazard), or designing for the hazard (i.e. making the element strong enough to resist the hazard). In the first case the structure shall be designed in such a way that all primary load elements that can be exposed to hazards are non-critical components. In the second case critical components that can be exposed to hazards are made strong enough to resist the hazards considered.

It should be emphasised that robustness requirements do not imply that all structures shall be able to survive removal of any structural element if no hazards are likely to occur. The starting point is a hazard that is more unlikely to happen than the usual design situations, but not unlikely enough to be neglected. If there is no hazard, then there is no requirement in relation to robustness. Also, only one hazard at the time should be considered.

Annex A5. JCSS – Robustness requirement in PMC

The robustness requirement in the JCSS Probabilistic Model Code is in section 3.1 and chapter 8. Below the text from chapter 8:

8. Annex A: The Robustness Requirement

8.1. Introduction
In clause 3.1 the following robustness requirement has been formulated:

"A structure shall not be damaged by events like fire explosions or consequences of human errors, deterioration effects, etc. to an extend disproportionate to the severeness of the triggering event".

This annex is intended to give some further guidance. No attention is being paid to terrorist actions and actions of war. The general idea is that, whatever the design, proper destructive actions can always be successful.

8.2. Structural and nonstructural measures
In order to attain adequate safety in relation with accidental loads one or more of the following strategies may be followed:

1. reduction of the probability that the action occurs or reduction of the action intensity (prevention)
2. reduction of the effect of the action on the structure (protection)
3. making the structure strong enough to withstand the loads
4. limiting the amount of structural damage
5. migitation of the consequences of failure

The strategies 1, 2 and 5 are so called non-structural measures. These measures are considered as being very effective for some specific accidental action. The strategies 3 and 4 are so called structural measures. In general strategy 3 is extremely expensive in most cases. Strategy 4, on the other hand accepts some members to fail, but requires that the total damage is limited. This means that the structure should have sufficient redundancy and possibilities to mobilise so called alternative load paths.

In the ideal design procedure, the occurrence and effects of an accidental action (impact, explosion, etc.) are simulated for all possible action scenarios. The damage effect of the structural members is calculated and stability of the remaining structure assessed. Next the consequences are estimated in terms of number of casualties and economic losses. Various measures can be compared on the basis of economic criteria.

8.3. Simplified design procedure
The approach sketched in A2 has two disadvantages:

(1) it is extremely complicated
(2) it does not work for unforseenable hazards

As a result other more global design strategies have been developed, like the classical requirements on sufficient ductility and tying of elements.

Another approach is that one considers the situation that a structural element (beam, column) has been damaged, by whatever event, to such an extend that its normal load bearing capacity has vanished almost completely. For the remaining part of the structure it then required that fore some relatively short period of time (repair period T) the structure can withstand the "normal" loads with some prescribed reliability:

$$P(R < S \text{ in } T \mid \text{one element removed}) < p_{target} \quad \text{(A1)}$$

The target reliability in (A1) depends on:

- the normal safety target for the building
- the period under consideration (hours, days or months)
- the probability that the element under consideration is removed (by other causes then already considered in design).

The probability that some element is removed by some cause, not yet considered in design, depends on the sophistication of the design procedure and on the type of structure. For a conventional structure it should, at least in theory, be possible to include all relevant collapse origins in the design. Of course, it will always be possible to think of failure causes not covered by the design, but those will have a remote likelihood and may be disregarded on the basis of decision theoretical arguments. For unconventional structures this certainly will not be the case.

8.4. Recommendation

For *unconventional* structures, as for instance large structures, the probability of having some unspecified failure cause is substantial. If in addition new materials or new design concepts are used, unexpected failure causes become more likely. This would indicate that for unconventional structures the simplified approach should be recommended.

For *conventional* structures there is a choice:

(1) one might argue that, as one never succeeds in dealing with all failure causes explicitly in a satisfactory way, it has no use to make refined analyses including system effect, accidental actions and so on; this leads to the use of the simplified procedure.

(2) one might also eliminate the use of an explicit robustness requirement (A1) as much as possible by taking into the design as many aspects explicitly as possible.

Stated as such it seems that the second approach is more rational, as it offers the possibility to reduce the risks in the most economical way, e.g. by sprinklers (for fire), barriers (for collision), QA (for errors), relief openings (for explosions), artificial damping (for earth quake), maintenance (for deterioration) and so on.

Annex A6. ASCE – Robustness requirements

The robustness requirement in …

Annex B1. Case study: Ballerup Arena, Denmark

What is a robust construction?
Jørgen Munch-Andersen, Danish Timber Information, 2008-08-01

B1.1 Introduction

Failures can in general be caused by flaws in the design or construction leading to a low load bearing capacity, or by an unforeseen incident giving rise to higher loads than expected.

The strategy for ensuring robustness might be different depending on which of the two causes that is thought of. This is illustrated below by the Siemens Arena case.

B1.2 Siemens Arena

On one morning two trusses in the roof of Siemens Arena suddenly collapsed, see Figure 1. It happened just a few months after the inauguration of the arena and a few days before a major bicycle event should have taken place.

Each truss was composed by two glulam timber arches with vertical connectors, see Figure 1. The upper arch was mainly exposed to compression and the lower to tension. The horizontal component of the tension and compression forces were neutralised at the corner connections by concealed steel plates connected to both arches by embedded dowels and a few bolts, see Figure 2. The structure appeared as an elegant slim construction with a free span of 73 metres across the arena. The failure occurred suddenly at a time with almost no wind and only a few millimetres of snow.

An investigation [1] showed that the problem could be localised to one critical cross-section at the corner in the tension arch where the strength was between 25 and 30% of the required strength, see Figure 3. By mistake, this cross-section was not considered at all in the design.

Three errors explain what happened:

- A 48% too high design strength was used for the timber part
- The reduction of the height of the cross section near the ends of the arches, see Figure 2, was not considered
- The holes in the timber for steel plates, bolts and dowels, see Figure 3, were not considered

The expected short term strength at the critical cross section happened to be slightly larger than the forces from the self weight of the structure, whereas the long term strength was smaller. Therefore the collapse could take place at a time with no special external load.

The investigation also revealed that the stability of the trusses was not ensured sufficiently and that the quality of the glueing of the glulam was not as specified. These problems did not contribute to the actual failure.

The collapse did not reveal any unknown phenomenon, so the main question is how such a vital error could pass the quality assessment of the design.

Figure 1. The roof of Siemens Arena after the collapse of two trusses. An intact truss is seen to the right.

Figure 2. The corner where concealed steel plates connects the timber parts. Between the visible bolts numerous dowels are placed.

Figure 3. Rupture at the critical cross section in the corner connection. Note the dowels and steel plates.

B1.2.1 Robustness strategies

The 12 m long purlins between the trusses were only moderately fastened, such that a failure of one truss should not initiate progressive collapse. This strategy proved to work fairly well as only two of the 12 trusses collapsed. As all trusses had much lower strength than required it might be fair to conclude that the extent of the collapse was *not* disproportionate to the cause.

Another and more expensive strategy against progressive collapse could have been to design the trusses, the purlins and their fastening such that a failed truss and the roof could hang in the purlins and transfer the load to the neighbour trusses (when considered an accidental load case).

Had the cause of the failure been a huge load on one truss this strategy would have been preferable because it significantly reduces the risk of injuries. The strategy would also have worked if a leaking roof had degraded one truss because it is likely that the other trusses are unharmed.

But given the cause of the actual collapse this strategy would most likely have caused a total failure as the neighbour trusses could not have withstood the extra load.

The bracing in the longitudinal direction was ensured by two systems, one at each gable. This ensures stability of the remnant part of the building when one truss has failed, no matter which truss. This strategy also proved successful, even though there was no wind or snow to call for big demands to the bracing system. If insufficient stability of the trusses had caused a failure the division of the bracing into two systems might also help, especially if both systems can sustain the entire load. With only one system there will most likely be key-elements for which failure will cause a total collapse.

B1.3 Discussion

There is a significant difference between human errors and other incidents causing failures.

Human errors lead to a too low load bearing capacity and are therefore likely to cause failure for foreseeable loads. The ability of the structure to redistribute the load even in a parallel system might small because the other components are likely to inherent the same error and therefore also are weak. An attempt to enable redistribution might therefore cause a local failure to initiate a total collapse.

An unforeseen incident at a correctly designed and constructed construction might give rise to too high stresses, but most likely only in a small part of the structure. If the construction is a parallel system it is highly likely that the loads can be redistributed and sustained by the rest of the structure. If a series system is used this is of course not possible.

In principle robustness is aimed at reducing the risk of human injuries in the case of an unforeseen incident. It is assumed that the structure fulfils the requirements of the codes. Under these assumptions parallel systems are very attractive as they will minimise the consequences. Series systems can be overdesigned, but it is against the idea of robustness to try to design for an unforeseen incident.

But in real life most failures are related to human errors such that the structure does not fulfil the requirements. Limiting the consequences of a failure in case of a systematic human error demands that load is not significantly redistributed. Weak spots appear to be the only way to ensure that.

Therefore, when advising strategies for ensuring robustness it must be considered if the strategy might increase the consequences of human errors.

Annex B2. Case study: Bad Reichenhall, Germany

The Bad Reichenhall Ice-Arena Collapse

A contribution to COST action E55

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The first two chapters of this article are taken from: Winter, S., Kreuzinger, H., 2008 “The Bad Reichenhall ice-arena collapse and the necessary consequences for wide span timber structures”. Proceedings WCTE 2008 Conference 2008, Miyazaki, Japan.
The third chapter “Considerations on Robustness” was added to suit the specific purpose of this publication.

Annex B3.

Secondary Structures - Purlins - Robustness Considerations

A contribution to COST action E55

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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”. This article will evaluate typical secondary systems for timber roof structures against these requirements, including comparative calculations for typical purlin systems. Applying the finding that most failures of timber structures are not caused by random occurrences, e.g. low material weakness, but by systematic mistakes, it is shown that the objective of load transfer - often mentioned as preferable - should be critically analysed for such structures.

7 References

List of references will be updated and rearranged

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