COST Action E55 – Modelling of the Performance of Timber Structures

6th Workshop

University of Ljubljana
Slovenia

21-22 September 2009

Draft
Summary and Minutes of the workshop

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Minutes of the Ljubljana Workshop

1. Organisation

The 6th workshop of COST E55 was held jointly with COST TU0601 (Robustness of Structures) in Ljubljana and was organised by Goran Turk. His excellent work in organizing the event is much appreciated.

2. Participation

The following COST E55 participants participated in the meeting:

Baltrusaitis, Antanas (LT)	Frese, Matthias (DE)	Pranckeviciene, Vilija (LT)
Blass Hans Joachim (DE)	Harte, Annette (IE)	Rajcic, Vlatka (HR)
Bogensperger, Thomas (AT)	Jorissen, Andre (NL)	Rodd, Peter (IE)
Branco, Jorge (PT)	Kirkegaard, Poul Henning (DK)	Rodriguez, Vladimir (ES)
Brandner, Reinhard (AT)	Kohler, Jochen (CH)	Schoenmakers, J (NL)
Brunner, Maurice (CH)	Kreuzinger, Heinrich (DE)	Sigrist, Christophe (CH)
Bruhl, Frank (DE)	Malo, Kjell Arne (NO)	Sorensen, John Dalsgaard (DK)
Cavaco, Eduardo (PT)	Mulders, Sigrid (NL)	Sprcic, Jelena (SI)
Cizmar, Dean (HR)	Muller, Andreas (CH)	Steiger, Rene (CH)
Cruz, Helena (PT)	Munch-Andersen, Jorgen (DK)	Svensson, Staffan (DK)
Dias, Alfredo (PT)	Neves, Luis (PT)	Tomasi, Roberto (IT)
Dietsch, Phillip (DE)	Norvydas, Valdas (LT)	Tononi, Davide (SI)
Dujic, Bruno (SI)	Palma, Pedro (PT)	Turk, Goran (SI)
Fink, Gerhard (AT)	Pazlar, Tomaz (SI)	Usardi, Irene (SI)
Fortino, Stefania (FI)	Plos, Mitja (SI)	Zhang, Ben (UK)
Fragiacomo, Massimo (IT)		

The following COST TU0601 participants participated in the meeting:

Enrico Rizzuto (IT)	Zenon Drabowicz (PL)	Victor Mircea Bucur (RO)
Radu Bancila (RO)	Bernt Leira (NO)	Geoffrey Decan (BE)
Inger B Kroon (DK)	Jana Markova (CZ)	Miguel Pereira (UK)
Traian Badea (PL)	Miroslav Sykora (CZ)	Michael Byfield (UK)
Krzysztof Cichocki (BL)	Luis Costa Neves (PT)	Ulrike Kuhlmann (DE)
Harikrishna Narasimhan (CH)	Ton Vrouwenvelder (NL)	Leslaw Kwasniewski (PL)
Carmen Bucur (RO)	M. Chryssanthopoulos (UK)	Luis Canhoto Neves (PT)
Eduardo Cavaco (PT)	Victoria Janssens (EI)	Selcuk Toprak (TR)
Petros Christou (CY)	Fulop Ludovic (FL)	Daniel Honfi (SE)
Christos Bisbos (GR)		

3. Workshop content – Day 1

The workshop was opened by the organiser Goran Turk, who welcomed participants of the two actions to Ljubljana.

3.1 Introductory remarks and workshop overview

On behalf of Action TU0601, Michael Faber, Chairman, welcomed the participants to the first joint meeting. The workshop, which will facilitate a discussion on robustness by both parties, was strongly supported by the rapporteur, Kiril Gramatikov. This meeting is important for the Commission as well as for us. The two actions are already having impact with respect to the international research community. One visible sign of this is that many international conferences now have special sessions devoted to robustness of structures. This is a very positive outcome of our work.

Jochen Kohler, Chairman of COST E55, spoke about the interrelation of two actions. COST E55 has three working group, one of which is devoted to the study of robustness of timber structures. Of the other two working groups, one aims to enhance the basis of modelling and the other is considering assessment of failures and malfunctions. The main issues have been identified and further research is underway to provide information to be utilised in the robustness calculations.

Michael Faber introduced the programme for the two day meeting.
The programme is as follows:

DAY 1: SEPTEMBER 21, 2009 – MONDAY
Registration
Welcome, introductory remarks and workshop overview

Joint Workshop with COST Action TU0601 – Presentation of Factsheets

Theme 1: Experience
Assessment of failure and malfunctions – general (Radu Bancila) TU0601
Assessment of failure and malfunctions – timber structures (Matthias Frese) E55
Failure template (Tomi Toratti) E55
Robustness 'experiences' from failed timber structures (Jorgen Munch-Andersen, Phillip Dietsch) E55

Theme 2: Principles
Robustness – theoretical framework (John Sørensen) TU0601/E55
Robustness – acceptance criteria (Enrico Rizzuto) TU0601

Theme 3: Exposures
Probabilistic modelling of exposure conditions (Ton Vrouwenvelder) TU0601
Modelling of human errors (Ton Vrouwenvelder, Milan Holicky and Miroslav Sykora) TU0601
Moisture (Jochen Köhler, Gerhard Fink) E55

Theme 4: Vulnerability
General aspects, modelling and analysis (Leslaw Kwasniewski) TU0601
Material – Timber, General (Sven Thelandersson and Daniel Honfi) TU0601
Material – Timber, Ductility (André Jorissen, Massimo Fragiacomo) E55

Theme 5: Robustness assessment and implementation
Robustness requirement / criteria in codes, regulations and best practice guides (Dimitris Diamantidis) TU0601
Categorisation and assessment of robustness related provisions in European standards
(Harikrishna Narasimhan and Michael Faber) TU0601

Actions TU0601 and E55 Steering Committee Meeting
Chairs : Michael Faber and Jochen Köhler

Day 2:
Joint Workshop with COST Action TU0601– Presentation of Factsheets - continued
Theme 6: Implementation
Earthquake and robustness for timber structures (Jorge Branco) E55
System reliability – ductility and redundancy (Poul Henning Kirkegaard) E55
Robustness design of timber structures (Phillip Dietsch) E55
Discussion

General Discussion on Identified Topics – Joint Session with Action TU0601

Parallel Working Group (WG) Meetings E55 – Discussion, presentation and planning of work

WG 2 Meeting - Moisture induced stresses	Chair : Staffan Svensson
WG 2 Meeting – Ductility	Chair : André Jorissen
WG 3 Meeting – Robustness	Chair : John Sørensen

Joint E55 Session and Closure

MC Meeting Joint E55 Session and Closure

3.2 Joint Workshop with COST TU0601 – Presentation of Factsheets

Theme 1: Experience

1.1 Assessment of failure and malfunctions – general
(Radu Bancila, TU0601)
The first step in the consideration of robustness is the assessment of defects, malfunctions and failures. These result from different causes including material failure, human error, or natural perils. Possible project errors are inadequate choice of material properties, change in operating conditions, extreme environmental conditions, or traffic accidents.
Romania's railway network is 14300 km long and has 4290 steel bridges of which 30% are welded structures. A programme of continuous maintenance is very important. This incorporates non-destructive bridge inspections involving visual inspection, magnetic particle inspection, liquid penetration inspection, and ultrasonic inspection. Defects were widespread

and amplification due to climate and polluting factors was found. Cracks were detected in the main girders, stringers, deck and bracing. In the newer bridges, steel sheet exfoliation or lamellar tearing was an issue so fatigue tests were carried out. Some earthquakes caused damage but the bearing capacity of the bridges was not affected. With regular inspection, cracks can be detected and most failures prevented.

The present methodology for verification of existing bridge capacity involves three steps: initial estimation of load capacity using simplified approach, accurate determination of stresses using computer models and material testing and finally in-situ testing. The final result of the calculations is a cumulative damage parameter D_p. For D_p greater than 1.0, there is no remaining service life. This usually results in additional inspections, traffic restrictions and strengthening measures. Depending on the values of D_p, more or less repair and monitoring is needed. It must be emphasized that a strengthened structure is not a new structure. With traditional static (2D) analysis, load bearing capacity is overestimated. Some basics of fracture mechanics were also provided. A worked example on a steel railway bridge was presented.

1.2 Failures of timber structures - considerations on the causes of the fault

(Matthias Frese E55)

Results of a German study of failures of timber structures were presented. The first step was the collection of failure data on 428 structures from expert's and glulam manufacturers' reports. The 2^{nd} step was the identification of primary damages, followed by the determination of the causes of the primary damages and the final step was the consideration of the relationship between the causes and the damage. 70% of the primary damages were due to cracks parallel to the grain direction. The failures examined were from locations spread over most of Germany but few reports from the former Eastern Germany are available yet. It is hoped to expand on this very extensive collection of data. The analysis and results are only for the German case but nevertheless should give a rough idea of way things are in Europe.

The following causes of failure were identified: Alternating climates, Building physics, Construction methods, Construction details, Overload, Material quality, Planning, Shrinking or swelling and Other. Some examples of failures shown were: failure due to poor positioning of dowels (construction methods), discolouration due to condensation (building physics), failure of end-notched beam due to lack of reinforcement (planning/disregard for design rules), open finger joints (material quality), poor glue-lines between laminations (material quality), collapse due to accumulation of rainwater on roof (overload), cracks in grain direction due to hindrance of shrinkage (swelling/shrinking), cracks in grain direction (alternating climate). Some problems can be avoided by using a different approach e.g. in roof trusses using steel for tension members, using cladding on timber bridges to prevent decay. Half of the damages were associated with alternating climates, overload and construction errors.

Discussion.

JK asked about the type of failure included in the German study and whether it was only collapses or were issues such as durability considered. HB stated that there was no serviceability failure included and most of the failures were local damage. SS asked what is the difference between alternating climate and shrinkage/swelling? MF explained that for shrinkage and swelling the moisture content was not alternating but either increasing or decreasing. He said that they are closely related and perhaps shouldn't be differentiated.

1.3 A proposal for a failure template
(Ludovic Fulop TU0601)

This presentation was made on behalf of Tomi Toratti, who was unable to attend. Failure assessments have been carried out in a number of European countries but these have not been carried out in a uniform manner. In this presentation, a proposed common standardized format for failure assessment was presented. The objectives of the failure template are to ensure a uniform quality and uniform level of detail of assessments, to help the expert carrying out the assessment and to provide data for analysis so that design and construction methods can be continually improved. It should be kept in mind that not all structural failures can be reached with these assessments due to the fact that they are not assessed or even considered as failures e.g. durability cases. Another issue relates to serviceability cases – failures due to excessive floor vibration are often not made public. While the template may be used in both public and confidential assessment situations, this issue is not addressed in the failure template procedures.
The template was used on a well-known example of failure in Finland, the Jyvaskyla Fair Centre. At the start, some general information about the type of structure is given. Then, information on the location and type of failure and when it occurred is given. This is followed by an assessment of the progressive nature of the failure and robustness. The final part of the template relates to the causes of failure. Failure cause classification is considered under four headings: related to structural design, related to building materials, related to construction on-site and related to building use. There is an article discussing the proposed template written by Tomi Toratti in a recent publication by VTT, Finland. The publication 'Scientific activities in safety and security 2009' can be downloaded from the website: http://www.vtt.fi/research/tic/

1.4 Robustness experiences from two failed timber structures
(Phillip Dietsch & Jorgen Munch-Andersen E55)

Robustness experiences from two failed timber structures (Bad Reichenhall Ice Arena, Germany, and Siemens Arena, Ballerup, Denmark) were presented. These examples show different design strategies, which had different consequences for robustness.
The Bad Reichenhall Ice-Arena was built in 1971/72. The main girders, which had a span of 48m and height of 2.87m, were manufactured in 16m sections and jointed on site. The girders comprised top and bottom glulam girders with Kampf web boards. In 2006, the structure experienced a progressive failure resulting in 15 casualties. The reasons for failure included: (i) an error in the structural calculations – no strength reduction for the finger joints resulted in a reduction of the safety factor to 1.5 (ii) the Kampf web boards were not compliant (iii) unreliable on-site gluing process and (iv) loss of the adhesive effect of UF glue due exposure to high moisture content. It is believed that collapse initiated in one of the three main girders on the east side, and then the high stiffness of the K-bracing resulted in a redistribution of the loads from the failing girder to the adjacent ones, leading to progressive collapse within seconds. A better approach to the design would have been to have a partition between secondary system and bracing against torsional buckling, to use a statically determinate secondary system with more flexible connections and to compartmentalize the k-bracing. This would give the possibility of one girder failing without progressive collapse.
The roof of Siemens cycling arena comprised 12 'cigar' shaped trusses with a span of 73m and with a spacing of 12m. Simply supported purlins spanned between the trusses. Two of the

trusses fell down without warning a few months after the opening of the arena. At failure, there was almost no wind or snow. Luckily, there were no people present at the time. Failure occurred in the tension chord at the end dowelled joint. The reason for failure was due to a number of critical design errors including the following: the design strength used was 48% too high, the reduction in the height of the cross-section was not considered and the reduction in the cross-section due to holes for plates and dowels was not taken into account. As the purlins were only moderately fastened to trusses progressive collapse was avoided and only two trusses failed. An alternative robustness strategy could have been adopted for this structure: trusses, purlins and connections could have been designed to permit a failed truss to hang from the neighbouring trusses. This would have led to a progressive collapse with the design errors that occurred.
In comparing the two collapses it can be concluded that no strategy can ensure robustness in all cases. Different scenarios must be considered especially systematic error or unforeseeable incident. Independent checking of design and construction are needed to avoid human error. Neither of projects had undergone independent checking. This was introduced in Denmark after the collapse of the Siemens arena.

Discussion
PD was asked about snow loading on Bad Reichenhall arena. He replied that snow loading at time was the actuator but it was not excessive. A snow load of 1.5 was used in the design but the actual load at failure was less than this.
For the Bad Riechenall case, before the structure was built the factor of safety came down from 2 to 1.5 due to design errors and during the life of the structure this was further reduced.
The question was asked whether it was possible to capture that something was wrong beforehand by carrying out monitoring. For the Siemens arena, the failure was brittle so monitoring would not have helped but with the Bad Reichenall arena the deterioration was progressive deterioration so monitoring may have helped.
AJ asked about the glue used for finger joints – UF was used between the upper and lower glulams
MF expressed the view that third party control was not necessary if the first design was done well. Responsibility is still with original designer and not with the checker. Others expressed the opinion that 3rd party checking is now very effective.

Presentation of Factsheets
Theme 2: Principles

2.1 Robustness – theoretical framework
(John D Sorensen, E55 & TU0601)

This fact sheet will start with a consideration of why robustness is an issue giving as examples the Oklahoma bombing in 1995, the World Trade Centre collapse in 2001 and the terminal collapse at Charles de Gaulle airport in 2004. The reasons for failure are unlikely to relate to extreme high loads or low strengths as these are well covered in the codes. More likely reasons are design errors, execution errors, deterioration of critical structural elements, unexpected hazards and systems effects. EN1990 and EN1991-1-7 require that a structure be designed and executed in such a way that it will not be damaged to an extent disproportional to the original cause.

A probabilistic model for robustness was presented that expressed the probability of collapse in terms of the product of the individual probabilities of exposure, the damage due to that exposure and the consequences of collapse. Using a risk-based model for robustness, the total risk is the sum of the direct risk (related to local damage) and the indirect risk (related to collapse). A number of different robustness indicators were defined: risk-based, reliability-based, deterministic and conditional risk-based. A number of measures by which potential damage can be avoided or limited were discussed. In order to achieve robustness, it is not always a good idea to use redundant systems (i.e. tie elements together). Sometimes, the use of statically determinate (series) systems can be better. The robustness strategy depends on the exposure type, the correlation of exposure between elements, the type of structural system, the load bearing capacity and the load type.
In code based design, there are three threads: the standard code format, which is component based, robustness requirements, which are system based and quality control requirements to detect human error and to track deterioration.
He concluded by giving a list of the six fact sheets on robustness that are being prepared:

1. Robustness – theoretical framework
2. Robustness – acceptance criteria
3. Earthquake and robustness for timber structures
4. System reliability – ductility and redundancy
5. Robustness design of timber structures
6. Robustness ‘experiences’ from failed timber structures.

2.2 Robustness – thoughts on acceptance criteria
(Enrico Rizzuto TU0601)

He is not sure whether we are able to make some conclusions yet on acceptance criteria. There are indicators available but there is still work to do to understand the quantification and see what to do with it. The total risk is the only performance indicator for performance based design. The robustness index is: $I_{rob} = R_{dir}/(R_{dir}+R_{ind})$. This is not only a property of the structural system in narrow sense but also of its ‘universe’. The same system exposed to collision of more cars is less robust than that subject to possible collision of few cars.
Question: can we judge (accept or reject) a structural system on the basis of robustness considerations alone? Robustness is not used as a performance indicator – the real performance indicator should be the ‘total’ risk (total expected loss). We should concentrate on total risk and not robustness.
As far as acceptance criteria are concerned, the key point is to find a general framework for structural decision making. The same structure can have different indices depending on where it is intended to work so different acceptable values should be provided for different ‘exposure’ scenarios.

Discussion
JMA feels that it is quite complex to calculate the probability and the human error estimation is quite tricky. MFa said that by decomposing the total probability in the different components, in some cases the probability of exposure may become very low and negligible. UK said that, in her opinion, the robustness index is just a means to compare structures and that having to evaluate the total risk may lead to nothing in the end. MFa said that using the approach based on probability would probably be premature right now due to the complexity of the models. The

variability of the direct risk is bounded by the codes. The problem is the indirect risk. The idea is that we should reduce the indirect risk.

Presentation of Factsheets
Theme 3: Exposures

3.1 Probabilistic modelling of exposure conditions
(Ton Vrouwenvelder TU0601)

In risk based robustness analysis, the risk is determined from the product of probabilities. For this we require hazard models, member models and post failure models. These models include both physics and statistics. Hazards can be divided into foreseeable and unforeseeable hazards. The foreseeable actions include natural accidental actions (e.g. earthquake, flood), manmade accidental actions (e.g. explosion, fire), human influences (e.g. vandalism), normal loads (e.g. imposed, snow) and human errors (e.g. design and construction errors).
The JCSS Probabilistic Model Code Parts 2 & 3 covers the modelling of loads and structural properties. Data was presented for probabilistic collision force calculations. A model for calculating the equivalent static pressure in the case of internal natural gas explosions was discussed and UK statistics on the strength of explosions and their probability of occurrence were presented. The observed scatter in the severity of the explosion force was related to parameters such as window area.
In the case of unidentified conditions (unforeseeable, unforeseen or not foreseen), the question is what is a reasonable probability of their effects. Data is the difficult issue. There are many papers discussing collapses that can be used but they are not enough. What is needed is to combine intuition with the data – risk type thinking. To finish, a quote from Thomas Bayes: The more data the better but no data = no excuse.

Discussion
JK For timber, there are different resistances for different directions and load types so we may need to look at different approach. TV said that if he is sent the problem definition, he will look at it.

3.2 Modelling of human errors
(Ton Vrouwenvelder TU0601)

In a study by Scheider and Matousek, which considered 500 cases of human error, 25% were attributed to lack of knowledge, 30% to careless engineering, 15% to real error and 20% to accepted risk. In another study, Imam and Chryssanthopoulos examined 156 failures in steel bridges and concluded that 24% were design errors, 23% due to limited knowledge, 19% due to natural hazards, 14% due to human error and 13% were accidents. Errors may occur at the planning and design stage (40%), during execution (40%) or when the structure is in use (20%).
The risk is the product of the probability of human error, the probability of damage due to human error, the probability of collapse due to damage and the cost of collapse. A human error model was described that expressed the probability of failure in terms of the probability of human error and the probability of failure with and without human error. Numerical values of the probability of error depend on professional skill / experience, the complexity of the task,

physical and mental conditions (stress and time pressure), social factors and adaptation of technology to human beings. Gutman and Swain [1983] give estimates for the probability of making errors based on five classes of tasks. P(error) = 10^{-class}. Some researchers have investigated the percentage of human errors based on the type of structural member (foundation, column, beam, etc.) but no consistency was found between the different results. All the hazards (human error, fire, explosion) should be considered together in P(H) as increasing the resistance to account for one form of hazard/error may influence the outcome in other areas.
Quality insurance is a way to reduce the error. A design with nine intentional errors was sent out for peer review to a number of companies and 50% of them didn't find the biggest error. The probability of detecting errors was found to increase with time available. It was shown that by increasing robustness the effect of the human error on the probability of failure can be significantly reduced. Conclusions: human error can be modelled like other hazards however data is limited and numbers have to be estimated. Difficult points are correlation and experience. Optimization of Quality Assurance is possible.

Discussion
Quick checks are of little use. You need time to check calculations properly. MFa If you are an experienced engineer, you will have a feeling for the structural size and this may help reduce human error and to detect a mistake.

3.3 Timber structures exposed to moisture
(Jochen Köhler E55)

The load bearing capacity of timber structural elements is affected by moisture and moisture variations and also by the time history of applied loading. This is handled in the Eurocodes by using a strength modification factor, k_{mod}, which is dependent on the service class and the load – duration class. The effect of moisture on the load-bearing behaviour is two-fold. Firstly, as timber is a hygroscopic material, stresses can arise due to restrained shrinkage and swelling (Type I). The second effect is accelerated aging as a result of moisture state and history (Type II). Examples of Type I behavior discussed were: dowel type fasteners installed at a high moisture content result in cracks due to shrinkage perpendicular to grain as the timber dries out; in a large glulam section, shrinkage cracks occur as drying takes place; in the case of a very dry glulam, when wetting occurs the outside swells and not the inside so cracks develop from the inside. From analysis of failures, moisture induced stresses were found to be a large part of the problem.
For Type I behaviour, which arises from restrained shrinkage at connections or in large cross-sections, the moisture induced stresses are dependent on the maximum difference in relative humidity during the service life. For designers, specific regulations on dowel arrangements perpendicular to grain, initial and in-service moisture contents should suffice for most cases. Moisture stress design should only be for key components. Type II behaviour is the aging that arises from the combined effect of load and relative humidity history and is time dependent. In this case, use of the k_{mod} principle seems okay but further development of the model basis is necessary and the load duration and climate classification classes should be refined.
Some data on indoor climate from the Sibelius hall and from an Ice skating hall were presented. With regards to outdoor climate, there is a lot of data available. For both indoor and outdoor climate, we must decide on what time resolution should we take to model moisture. It may be

possible to model the indoor climate in terms of the outdoor climate, the indoor moisture and heat production and the ventilation using humidity equilibrium equations. Some reference values for moisture production rates and indoor humidity levels were shown .

Discussion
MFr We should try to identify a number of climatic regions with Europe – max 10 – 20 climatic regions. If we look at 10 different types of buildings, the number of combinations gets quite large so it would be very useful if we can establish a relationship between indoor and outdoor conditions.
PD In riding arenas, sprinklers are used to keep dust down so the outside climate is not as important as production of moisture internally.

Presentation of Factsheets
Theme 4: Vulnerability

4.1 General aspects, modelling and analysis
(Leslaw Kwasniewski TU0601)

The objectives of robustness analysis are to estimate the potential for progressive collapse and to mitigate the effects of disproportional damage. There are differing requirements depending on the purpose of the model: for design need simplicity, for robustness analysis need efficiency and for research purposes need accuracy. The quantities of interest are the internal forces after local damage, effect of secondary damage and the overall structure behaviour after primary failure.
There is a need to define the loading configurations including abnormal loads, global failure criteria quantitatively defining the collapse phenomenon, and adequate analysis methods. En1990 defines the load configuration for accidental loading. In the US, standards are defined by the general Services Administration (GSA) and the Department of Defence. Abnormal loading scenarios include notional removal of major load bearing structural elements.
Analysis methods may be categorized as linear or nonlinear and static or dynamic. A linear analysis is sufficient for most civil structures under service loads. Nonlinearities arise from a number of sources including material behaviour, large deformations, loading and boundary conditions and would be used for example to model the removal of a column due to a gas explosion. The nonlinearities may be smooth or rough. In the second case, it is very difficult to get convergence. Analyses may be further categorized as Static, Quasi-static or Dynamic analyses depending on whether the response is time dependent. For dynamic problems, there are two approaches to time integration: explicit or implicit. In the implicit approach, a smaller number of more computationally expensive iterations is involved than the explicit approach, which uses a large number of less expensive integration cycles. In terms of space discretisation, the finite element method is dominant with most models based on beam elements. Multilevel approaches are becoming more widespread and they are well suited to parallel processing.
A study by Marjanishvili which compared linear and nonlinear, static and dynamic analyses for progressive collapse was discussed. Nonlinear dynamic analyses are more accurate; however, being the most complex, they are prone to errors and should be used only by very experienced users. Other finite element models were presented using different commercial software. The conclusion is that there is no need to develop specialized software, and the approach with beams and columns is most common.

Discussion:
The additional cost of analysis is not known. In following the approach of the GSA and carrying out the analysis for the case of the removal of an element such as a column, a nonlinear static analysis is more suitable than dynamic analysis. A full dynamic analysis is for research only; it is not suitable for design. When doing a dynamic analysis, one can speed up the computation by using parallel systems. 3D complex modelling can predict accurately a vulnerability scenario only if complemented by experimental testing.
Local effects may be very important. Local buckling can lead to propagation of damage and delamination effects are also important. The problems are not as complex as some military applications, so we should not be afraid to use these tools.

4.2 Behaviour and modelling of timber structures with reference to robustness
(Daniel Honfi TU0601)

This paper was presented on behalf of Sven Thelandersson who was unable to attend. The fact sheet will contain: an overview of the behaviour of timber structures in relation to the requirements for robustness, a description of the structural properties of timber elements and systems, basic material characteristics and a review of failures modes. From a robustness perspective, the main use of structural models is to describe the degree of sensitivity of structural systems to extraordinary types of exposures often leading to local failure. For structural timber, spatial and random variability in strength and stiffness should be considered. Strength is defined on the element level rather than on the material level and the values for bending, tension and compression are different.
For robustness, you need to be able to redistribute stresses in the structural system and this requires large deformation capacity. At a local level, timber elements have low deformation capacity and behave in a brittle manner so it is difficult to utilize redundancy and achieve robustness. Compression behaviour is ductile but tension, shear and bending responses are brittle. Some ductility can be achieved in dowel type joints when the load is transferred through compression of the timber and when there is plastic yielding of the dowels. Energy absorbing and ductile adhesive joints are under development. Some structural systems, such as timber frame buildings or systems based on solid planar wood panels, are favourable for providing ductility and redundancy. Two strategies may be used to avoid progressive failure in long-span timber structures: the isolation of collapsing sections or the provision of alternative load paths. Good results can be obtained from structural analysis using linear elastic models for members and nonlinear models for joints.

Discussion
Comment: With the move to using threaded nails, which are brittle, can we say that robustness is increased? HB stated that these nails are not hardened and so are ductile. Problems may, however, occur with high strength screws.

4.3 Material – Timber, Ductility
(André Jorissen E55)

It is proposed to produce four factsheets on ductility. The first will be' General notes on ductility' and will cover the reasons why ductility is needed, including providing warning before failure, for redistribution of loads, for redistribution of stresses, for energy dissipation for earthquake design and for robustness. It will consider both member and connection ductility. The second fact sheet will deal with 'static ductility'. This fact sheet will provide definitions of ductility (currently 10 have been identified) and will cover load-slip analyses. The third fact sheet will be about 'dynamic ductility' and will consider earthquake design and bridges. For earthquake design, two definitions of ductility are available, both based on energy dissipation. An additional definition based on earthquake design is the overstrength, which is useful to prevent premature failure of a brittle member. Bridges are subject to reverse loading, which could result in problems with fatigue and pinching in the connections. Elastic design only is allowed. The final factsheet will give some 'benchmark examples'. An example was shown of a four-span continuous beam with semi-rigid connections in the members. For this example, redistribution of load is possible, no full mechanism will develop so there will be no large displacements, plastic deformations of the connections will provide a warning in the case of possible overload and energy dissipation is possible.

Discussion
TV pointed out that the overstrength should be evaluated starting from a given reliability target and then deriving the overstrength factor. BD pointed out that it is of great interest to evaluate also the yield point and the initial stiffness. UK was confused by the definitions. Firstly, she stated that overstrength is not only for dynamic ductility but also for ductility design of the system. The other issue is that fatigue has nothing to do with ductility. It starts with a crack and is quite brittle and it is a normal service load that is considered. AJ said that for timber bridges, elastic calculation of loading is required by the code. HK stated that this is also required at the ultimate limit state. Members are designed elastically but joints can be plastic. UK said that plastic design should be allowed for exceptional load combinations. KAM stated that pinching will happen when you have reverse loading and you have to design for it.

Presentation of Factsheets
Theme 5: Robustness assessment and implementation

5.1 Robustness requirement / criteria in codes, regulations and best practice guides (Dimitris Diamantidis TU0601)

Robustness is a performance characteristic of a structure representing its insensitivity to local failure. Resistance is usually considered on local level. Codes do not include a probabilistic approach when considering global behaviour.
In 2004, the Norwegian codes introduced provisions related to resistance to accidental loads and resistance in the damaged condition. After the Mont Blanc Tunnel fire in 1999, the codes for new long tunnels introduced the requirement that the probability of explosion and fire should be considered in design. New European guidelines for tunnels define five hazard probability levels depending on the frequency and associated probability levels and five hazard severity levels. These are combined in a table to determine the acceptability level.
The building codes contain lots of guidelines on robustness. EN 1991-1-7 Annexe A defines three classes of buildings from class 1 (low rise) to class 3 (more than 15 storeys). Class 3 is the most

important and risk analysis is recommended and/or advanced mechanical models. Structural integrity is provided through peripheral ties at each floor, vertical ties and internal ties.
ASCE 7 (2005) includes design provisions for extraordinary events. There are two approaches: the direct method provides for an alternate load path and the indirect method involves provision of redundancy, ductility and other measures. Performance based design in the US is used for providing seismic and blast resistance of new buildings and for seismic upgrade of existing buildings. This method uses a risk acceptability matrix that depends on the performance objective and the hazard level.
Performance objectives for global failure are required. Guidelines for practicing engineers should be developed. Risk based rules are needed for important structures.

Discussion
When designer looks at the different methods, he/she can use the indirect method, which is simply a set of details or the direct method that is more complex. What should you do? DD When looking at the entire spectrum of buildings, there is no a single solution. It will be necessary to collect experience internationally in order to be able to make the appropriate choice. A problem with the tie method is that it is only based on force and does not have a specified deformation. Tying is not always a solution and sometimes has no benefit. However, ties are better than nothing – it is a minimum.

5.2 Categorisation and assessment of robustness related provisions in European standards (Harikrishna Narasimhan TU0601)

A well defined separation does not currently exist in modern codes between standard safety formats and provisions for robustness. A review of the European Standards for design, execution, material aspects and maintenance of concrete and steel structures was carried out. Provisions were examined under the following headings: approach to risk treatment, nature of control risk, relationship with event/exposure, manner of reducing risk and phase of life cycle of structure in which the provision is applicable. Commonly found measures for risk treatment include increasing local resistance, tying systems, ductility and redundancy. However, no link is made between these and an achieved level of robustness. The majority of measures relate to the planning, design and implementation phases of the structure and these are no provisions for monitoring during the operational phase.
The ideal approach would involve a joint optimization of standard safety formats and robustness provisions. The robustness provisions can be optimized by considering a complete risk model based on exposures, direct consequences and indirect consequences.

Discussion
It is intended that the current Eurocode provisions be assessed to see which are cost effective and which are not. It was suggested that the study be extended to include timber structures.

4. Workshop content – Day 2

Michael Faber and Jochen Kohler introduced the day and informed the participants about the 11th International Conference on Applications of Statistics and Probability in Soil and Structural

Engineering (ICASP11) which will be held in Zurich on August 1-4 2011. This will provide an opportunity to disseminate some of the results of our actions.

Presentation of Factsheets
Theme 6: Implementation

6.1 Earthquake and robustness for timber structures
(Jorge Branco E55)

As there are common concepts in seismic engineering and robustness analysis, is it possible to apply established seismic design concepts to robustness analysis? The common concepts between the two areas include: classification of buildings according to their importance, redundancy and ductility are key factors, both are system based and unexpected and seismic events are both extremely rare and therefore difficult to quantify. Importance classes have been defined for different buildings for seismic design in the Eurocodes. The main difference is that robustness affects a single building and earthquakes affect an entire region. Redundancy is a key aspect for seismic design, as well as bi-directional resistance and stiffness. Timber seismic design is different to steel and concrete as timber elements behave linearly so you must rely on the connections for energy dissipation.
EC8 defines ductility classes for timber structures as having low, medium or high capacity to dissipate energy depending on the ductility of the connections. Limitations on the design of connections exist to guarantee ductility. Overstrength, redundancy and interconnectivity requirements are specified. Robustness can be improved by selective overstrength (strong column/weak beam), increased ductility, and increasing redundancy. It was shown that design for seismic resistance would have not prevented the two collapses of the Bad Reichenhall and the Siemens arena because it would have increased the transversal stiffness and increases the consequences. However, design for earthquake resistance would have probably prevented failure in the Alfred Murrah Federal building and the Ronan Point building. In general, therefore, seismic design and design for robustness are not fully equivalent.

6.2 System reliability – ductility and redundancy
(Poul Henning Kirkegaard E55)

At the start of his presentation, Poul informed the meeting about the 7th International Conference on Engineering Computational Technology which will be held in Valencia, Spain, on 14-17 September 2010.
The fact sheet on 'System Reliability – ductility and redundancy' was discussed. Series, parallel and hybrid system reliability models were described. The structural failure modes are important for assessing the robustness of a structural system. The two extremes are brittle and ductile failure modes. Simplified system modelling of ductile/brittle structural systems was described. Some analyses were performed were the reliability index was calculated for increased number of elements. For a ductile material, as the number of elements increases the system becomes more robust. Therefore ductility plays a very important role. Some differences between the terms redundancy, static indeterminacy, and robustness were underlined. For timber, the stress-strain behaviour in tension is brittle and in compression is ductile. The higher the grade of the timber the greater is the ductility. For timber joints, the problem is that there as many ductility definitions. A relative ductility measure was used to represent the ductility of timber

members and to carry out a parametric study aimed at investigating the influence of ductility on the probability of failure and, therefore, robustness.

6.3 Robustness design of timber structures – secondary structures – purlin systems (Phillip Dietsch E55)

As part of the general robustness requirements, a structure should be insensitive to local failure and progressive collapse shall be prevented. In this presentation, this is investigated by examining the impact of removal of a limited part of the structure.
The evaluated structure is a typical single-storey wide-span building was considered, with different purlin configurations: simply supported, gerber, continuous and lap jointed. The main span is 20m and purlins span 6m between the main beams. Two cases of limited removal are examined: the removal of a purlin between two supports and the removal of one support (equivalent to failure of main beam). Failure of one purlin did not result in overloading of the remaining members for statically determinate secondary systems but led to a stress increase of up to 50% in the remaining purlins for redundant secondary systems. Failure of one main member resulted in an additional load of up to 82% on the remaining main members.
However, numerous studies showed that the responsibility for failure was in most cases either systematic mistakes or global deterioration. Mistakes during planning or constructions are likely to be repeated in all identical elements leading to structures that will be prone to progressive collapse. Global weakening should be avoided by limiting failure to local level by using determinate secondary systems with flexible connections, for example.

Discussion:
MB questioned the aim of robustness: it was postulated that it is better to have redundancy, but then all the good examples presented were not redundant. UK commented that a high degree of statical indeterminacy does not necessarily mean that the structure is more redundant. HK commented that as engineers we have learned to make structures economical where the resistance must be equal to the load so there is no chance to redistribute the load. Also, as the main structures in timber are simply supported beams or trusses, how can they be made robust? Is it a matter of overdesign? Robustness must be seen in relation to the damage resulting – the engineer should aim for failure proportional to the cause. UK said that the key issue is in the connection since with a small effort you can get continuity and this will give redundancy and also help deflection.

General Discussion on Identified Topics – Joint Session with Action TU0601

1. Human error – checking engineers –control/review

HB stated that in Germany, checking engineers are used and the system makes sense. This is based on the experience of finding so many errors and mistakes can be minimized by having an independent checking system. MFa asked if we have any benchmarks to show that it helps or is it just an idea that the system is good. Do we know if control schemes add value? Why do we have a negative opinion of our ability?
There were a large number of contributions related to this topic including the following:

It was underlined that the independent checking is useful to force the designer to document his/her design very well.
In Norway, all bridges need checking.
In Denmark, checking is now standard. In Denmark, the insured losses are 10% of construction cost while the design costs are only 3% so maybe we can afford checking.
In Italy, a report is required but not necessarily detailed checking.
In the UK, there is a checking system for bridge projects and a similar system for buildings is proposed. Problems may arise due to current procurement methods. With different people doing different tasks, there is the potential for many errors at the interface. Designers not always involved in construction.
Switzerland has made a significant contribution to the development of timber structures and the main reason is that the timber code does not have many restrictions so having a checker could have the effect of reducing innovation.
The Construction Products Directive specifies that 3rd party control is needed. Important buildings should also have this. There was a feeling that there should be independent control because 85% of the failure is due to human errors. Education is also an important factor in reducing human errors.

2. Robust design – education, codes
Where to put provisions for robust design? Should it be in a code, or left open to engineers? JMA said that it may be a good idea to include some general statement in the code. At the moment, the code doesn't tell anything about how to make a design robust or what is the meaning of robustness. TV said that, due to weak codification, the same building will be considered as robust in one part of the Netherlands and non robust in another. Therefore it may be a good idea to have provisions in the code in order to avoid this problem. UK said that it is not clear, right now, what is considered as a robust structure. Perhaps the level of robustness should be left to the contract to the client, exactly like for serviceability limit state. ULS is mandatory, however SLS is not, and robustness is pretty much dependent on the level of the consequences. TV expressed the view that we should be careful of leaving the level of robustness to the client, and pointed out that robustness is connected to life safety and as such shouldn't be left to the choice of the client. MFa agreed with this view. The best practice is currently missing in the code and better rules should be included in the code.

5. Closure of Joint Workshop

The joint meeting was then closed. Michael Faber said that the factsheets will be used to prove the effectiveness of our joint meeting. These will be published in the near future and can be reused again.