Nordic Wood: Safety of Timber Structures

Summary reports

Larsen, Hans Jörgen

2002

Document Version:
Publisher's PDF, also known as Version of record

Link to publication

Citation for published version (APA):
Larsen, H. J. (2002). *Nordic Wood: Safety of Timber Structures: Summary reports*. (TVSM-3000; No. TVSM-3062). Structural Mechanics, Lund University.

Total number of authors:
1

LUND UNIVERSITY

Nordic Wood: Safety of Timber Structures SUMMARY REPORTS

HANS JØRGEN LARSEN (Editor)

Structural Mechanics

Structural Mechanics

ISRN LUTVDG/TVSM--02/3062--SE (1-47)
ISSN 0281-6679

Nordic Wood: Safety of Timber Structures

SUMMARY REPORTS

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Printed by KFS I Lund AB, Lund, Sweden, April 2002.

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Nordic Wood: Reliability of timber structures

Introduction

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The European design codes drafted by CEN, the Eurocodes presumes that the safety level is harmonized across materials.

According to European Standard "EN 1990 – Basis of Design", the numerical values of safety elements can be determined in either of two ways:

- On the basis of calibration to a long tradition of building.
- On the basis of statistical evaluation of experimental data and field observations.

The latter method is the one that has been prevailing: it has been left to specialist within in the different structural fields to assign values to the material dependent safety elements, of course within a general verification format and with material independent safety elements on the loading side.

The trend is that formal calculations are preferred to make sure that the different materials compete on an equal basis. This may be of serious disadvantage for timber structures, leading to bigger members than used today. This is the experience from the latest revisions of some of the Nordic design codes and preliminary calibrations indicate that the adoption of the Eurocode system could have the same effect.

The reason is not that there are any indications that the real level of safety is inferior to that of other structural materials even though the formal level may be so. On the contrary: experiences from practice show that timber structures behave very well, probably because secondary effects - e.g. load redistribution, moments in joints assumed pinned and lengthwise variation of strength in timber members - play a much bigger role than in other materials. The problem is that our knowledge has been insufficient to express and quantify these effects, in a form needed for the formal safety verifications.

To produce the necessary knowledge and data, *The Nordic Industrial Fund* has under its *Nordic Wood* programme supported the project *Reliability of Timber Structures.*

The main topics of this programme have been:

- Comparison of the safety systems and levels in the Nordic timber design codes and Eurocode 5.
- To collect data on the strength and stiffness properties of Nordic structural timber to determine the most appropriate distribution functions and their parameters (mean, coefficient of variation, characteristic values).
- To determine the load-duration factors for variable actions (snow, wind, imposed loads).
- To determine how it is possible to take into account the low probability that the cross-sections with maximum moments and forces coincide with the weakest cross-sections.
- To determine the effect of load sharing in statically indeterminate structures.

The main results are summarised in this report. A list of published reports giving more detailed information is given below.

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Dalsgaard-Sørensen, John & Hoffmeyer, Preben: Statistical analysis of data for timber strengths. Aalborg University, 2002.

Fonoll, A. T. & Sorinas, A. M.: Length effect of structural timber. Report TVBK-5108, Div. of Structural Engineering, Lund University, Sweden, 2001. (Masters thesis at Universitat Politecnica de Catalunya performed in Lund).

Frühwald, E. & Thelandersson S.: Reliability analysis of safety elements in Nordic Codes for timber structures. Report TVBK-3046, Div. of Structural Engineering, Lund University, Sweden, 2002.

Hansson M. & Thelandersson, S.: Effect of within member variability on the reliability of timber trusses. Paper 9.1.3, Proc. of World Conference on Timber Engineering, Whistler, Canada, 2000.

Hansson, M. & Isaksson, T.: System effects in sheathed parallel timber beam structures. Proc. CIB-W18, paper 34-8-1, Venice, Italy, August 2001.

Hansson M. & Thelandersson, S.: Assessment of probabilistic system effects on the reliability of timber trusses, 2001. To be published in Materials & Structures.

Larsen, Hans Jørgen: Properties affecting the reliability design of timber structures. COST E24 Seminar on Reliability of Timber Structures, Coimbra, Portugal, May 2001.

Larsen, Hans Jørgen: State-of-the-art Report: Safety principles and levels in the Nordic Countries. Report TVSM 3058, Structural Mechanics, Lund University. 2001

Ranta-Maunus, Alpo, Fonselius, M., Kurkela, J. & Torratti, T.: Reliability analysis of timber structures. VTT Research Notes 2109, Espoo, Finland.

Solli, Kjell Helge: Nordic Wood, Reliability of timber structures, Strength properties, background for Norwegian values, Norsk Treteknisk Institutt, Publication 2002.

Stang, Birgitte Dela, Isaksson, T. & Hansson, Martin: Experimental investigation of system effects in stressed skin elements. Danish Urban and Building Research Institute (in preparation for publication).

Stang, Birgitte Dela, Svensson, Staffan & Dalsgaard-Sørensen, John: Effect of load duration on timber structures in Denmark. Danish Urban and Building Research Institute (in preparation for publication).

Svensson, S. & Thelandersson S.: Aspects on reliability calibration of safety factors for timber structures. Proc. of the CIB/W18, 33-1-1, Delft Netherlands, 2000.

NORDIC WOOD
SAFETY OF TIMBER STRUCTURES

SUMMARY REPORTS

Safety principles and levels

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Reliability analyses

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Nordic Wood: Reliability of timber structures

Safety principles and levels in the Nordic countries and Eurocode 5

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Introduction

In this note the partial coefficient system and the values of some of the safety elements in EN 1995-1-1, "Design of timber structures, Part 1.1, General rules and rules for buildings" (Eurocode 5, 2001) are described and compared with the systems and values in the existing Nordic Timber codes as a basis for the adoption of the Eurocodes and the assignment of national values for the safety elements.

The member states of the EU and of EFTA/EEA shall adopt the general safety principles, but levels of safety of buildings and civil engineering works, including aspects of durability and economy, remain within the competence of the member states, making it possible to take into account differences in geographical or climatic conditions, or in ways of life, as well as different levels of protection that may prevail.

Conclusions

Notation

G	permanent action
Q	variable action
R	resistance, strength
S	action effect
k	factor
k_{mod}	modification factor for service and load-duration classes
γ	partial coefficient

Subscripts

k characteristic

Load cases

Only the simple case with permanent actions G and 1 variable action (Q) is covered and it is assumed that permanent actions are not dominating.

Safety format

The Eurocodes and the Nordic codes are based on the following format, where γ are partial coefficients:

$$S(\gamma_G G_k + \gamma_Q Q_k) = S_d < R_d = k_{mod} R_k / \gamma_m \qquad (1)$$

γ_m depends on the coefficient of variation for the characteristic material property, the accuracy of the design model and the uncertainty in the determination of the material parameter in the structure from standard tests. In some Nordic countries it also depends on the safety class (comparable to the Consequences Class, see below), the failure type and the control on site. In the following a structure belonging to normal safety class (Consequences Class 2), with ductile failure and normal model accuracy and uncertainty is discussed.

k_{mod} is a factor taking into account the influence of the load duration and the moisture history and is treated in the chapter "Effect on load duration on timber structures in Denmark".

For limit states where (1) applies it is possible freely to transfer part of the safety elements from one side to the other, i.e. (1) is identical to:

$$S(k\cdot\gamma_G G_k + k\cdot\gamma_Q Q_k) = k\cdot S_d < k\cdot R_d = k_{mod} R_k/(k\cdot\gamma_m) \qquad (2)$$

To make comparisons easier, a value of $\underline{(k\cdot\gamma_G) = 1}$ has been chosen.

Consequences classes and reliability differentiation

The required reliability depends on the consequences of failure or malfunction of the structure. In the Eurocodes a distinction is made between the 3 classes defined in Table 1.

Table 1 - Consequences Classes

Consequences class	Description related to consequences	Examples of building and civil engineering works
CC3	High for loss of human life and/or Very high economic, social or environmental	Grandstands Concert halls and other big and important buildings with many people
CC2	Medium for loss of human life and/or Considerable economic, social or environmental	Residential and office buildings and most other buildings
CC1	Low for loss of human life and Small or negligible economic, social or environmental	Agricultural buildings where people do not normally enter Greenhouses

In the Eurocodes it is proposed to associate the consequences classes with the required reliability as shown in Table 2, expressed either by the reliability index β or the formal probability of failure. The difference from CC2 to CC1 or from CC2 to CC3 corresponds approximately to a variation of the γ-factor on the action side (recommended in the Eurocodes) or on the material side (used in the Nordic codes) by ± 10 %.

Table 2 - Required reliability

	Reference period years	Consequences class CC1	CC2	CC3
Reliability index	50	3.3	3.8	4.3
	1	4.2	4.7	5.2
Formal probability of failure, approximately	50	10^{-3}	10^{-4}	10^{-5}
	1	10^{-5}	10^{-6}	10^{-7}
Factor to γ_m		0.9	1.0	1.1

Partial coefficients for actions

Two options for the partial coefficients are proposed in Eurocode 1. It is up to the member states to make the choice.

Option 1 is aimed at structures where the ultimate load corresponds to structural failure not involving geotechnical actions or resistances. Option2 can be used for all structures

Option 1

Expression (1) is used with γ_G = 1.35 and γ_Q = 1.50. If a reference value of $(k{\cdot}\gamma_G)$ = 1 is used, then γ_Q = 1.50/1.35 = 1.11.

Option 2

The less favourable of the following two sets of partial coefficients shall be used:

(a) γ_G = 1.35 and $\gamma_Q = 1.50\psi_0$ where ($\psi_0 Q_k$) is the combination value of O_k. For most actions ψ_0 = 0.7[1].

If a reference value of $(k{\cdot}\gamma_G)$ = 1 is used, then γ_Q = 0.7·1.50/1.35 = 0.78. This option is clearly not realistic for timber structures.

If a value of $(k{\cdot}\gamma_G)$ = 1.25 is used, then γ_Q = 1.0, and this combination may be appropriate for cases where dead load is dominating.

(b) γ_G = 1.15 and γ_Q = 1.50. If a reference value of $(k{\cdot}\gamma_G)$ = 1 is used, then γ_Q = 1.50/1.15 = 1.30.

Comparisons of safety elements

The safety elements in Eurocode 5 and the Nordic timber design codes are summarised and compared in Table 3.

The k_{mod}-values may be compared to the values determined in the chapter on Effect of load duration on timber structures in Denmark in this report:

Imposed:	0.80 - 0.85
Snow:	0.80 - 0.85
Wind:	1.10

To give an impression of the relative safety in the codes, a global safety factor, n, is calculated for typical case where $Q_k = 1.5G_k$:

$$n = (G_k + \gamma_Q Q_k)\gamma_m/(k_{mod}(G_k + Q_k)) = 0.4(1 + 1.5\gamma_Q)\ \gamma_m/k_{mod} \qquad (3)$$

[1] Exceptions are Storage load (ψ_0 = 1.0), snow for sites located at altitudes under 1000 m above mean sea level (ψ_0 = 0.5), and wind loads on buildings (ψ_0 = 0.6).

Table 3 - Safety elements in Eurocode 5 and the Nordic timber design codes

	Denmark	Finland	Norway	Sweden	Eurocode 5	
					Option 1	Option 2(b)
Actions						
Permanent	1	1	1	1	1	1
Imposed	1.3	1.3	1.25	1.3	1.11	1.3
Wind	1.5	1.3	1.25	1.3	1.11	1.3
Snow	1.5	1.25	1.25	1.3	1.11	1.3
Materials						
Timber	1.64	1.55	1.58	1.38	1.76	1.50
Glulam	1.5	1.55	1.32	1.27	1.69	1.44
LVL	1.5	1.55	1.32	1.27	1.62	1.38
Joints	1.64	1.55	1.58	1.38	1.76	1.50
k_{mod}, timber						
Storage	0.70	0.62	0.80	0.70	0.70	
Imposed actions	0.80	0.77	0.90	0.85	0.80	
Snow	0.90	0.77	0.90	0.85	0.90	
Wind	1.1	1.0	1.1	1.0	0.90	
Global safety						
Imposed actions	2.41	2.38	2.02	1.92	2.35	2.21
Snow	2.37	2.38	2.02	1.92	2.08	1.97
Wind	1.94	1.78	1.65	1.63	2.08	1.97
Overturning	1.88	1.67	1.67	1.53	1.65	

Nordic Wood: Reliability of timber structures

Reliability Analyses

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1. Introduction

In this section reliability analyses are performed in order to compare the safety level for the different Nordic timber codes (Danish, Norwegian, Swedish, Finnish) and the Eurocode. The analyses are made for both structural timber and steel. Reliability studies are also made to investigate the effect of statistical modelling of strength for structural timber. The strength data collected in the project, see Section x and references [1,2,3], are utilised directly in the analyses. Assuming that the collected data are representative, the reliability level for a variety of wood-based products is evaluated and compared. Furthermore, the effect on reliability level of proof loading or other types of quality control is investigated.

The basic tool for these studies is standard first order reliability method (FORM). The reliability analyses are performed with the program COMREL (included in STRUREL, a structural reliability analysis program package).

2. Principles and basic assumptions for the reliability analyses

In the studies presented here, the design format according to a code is combined with a probabilistically based design. Throughout this study, only one variable load is considered, giving the **limit state equation in the deterministic code format** as

$$\gamma_G G_k + \gamma_Q Q_k = C_k \frac{a_k \cdot f_k \cdot k_{\text{mod}}}{\gamma_m \gamma_n} \tag{1}$$

where index k denotes characteristic value and γ_G, γ_Q and γ_m are partial coefficients for permanent load, variable load and material strength respectively. γ_n is a partial coefficient accounting for safety class applicable for some of the codes considered.

The ratio of variable load to total load is defined as

$$\alpha = \frac{Q_k}{G_k + Q_k} \tag{2}$$

This gives the limit state design relation in the form

$$\left[(1-\alpha)\gamma_G + \alpha\gamma_Q\right](G_k + Q_k) = C_k \frac{a_k \cdot f_k \cdot k_{\text{mod}}}{\gamma_m \gamma_n} \tag{3}$$

The **limit state function in the probabilistic format** is

$$G + Q = C \cdot a \cdot f \cdot k_{\text{mod}} \tag{4}$$

where the stochastic variables are

- *G* = permanent load,
- *Q* = variable load,
- *C* = stochastic variable accounting for model uncertainty
- *a* = geometric factor which may be stochastic
- *f* = strength of the material
- k_{mod} = climate and duration of load factor, here assumed to be deterministic (k_{mod} = 1)

The strength modification factor k_{mod} is not included as a probabilistic variable in this part of the investigation. The special problem of duration of load is investigated separately in the Nordic Wood project and is summarised in Section y. The results presented in this section are intended to reflect general safety issues for the reference case with short term loads and for service conditions corresponding to k_{mod} =1. However, the strength reduction factor as it is given in the codes can have some bearing on the interpretation of the results since the values decided for k_{mod} in some codes can have been influenced by general safety considerations besides the pure physical influence of climate conditions and duration of load effects.

The limit state function (4) can be written as

$$h(G,Q,C,a,f) = C \cdot a \cdot f \cdot k_{\mathrm{mod}} - G - Q = 0 \tag{5}$$

where $h < 0$ implies failure.

The statistical properties assumed for the governing stochastic variables in the study are given in Table 1. All results presented later are derived under these assumptions, unless stated otherwise.

Table 1: Statistical Properties of the main variables.

Variable	Distribution type	COV [%]	Characteristic value equals
G	Normal	5	Mean value
Q	Gumbel	40	98 % percentile
C	Normal	10	Mean value = 1
a	Normal	2	Mean value = 1
f	To be varied	To be varied	5^{th} percentile

The ratio of variable load to total load, α, is varied from 0.2 to 1.0. The reliability index β is plotted against α, showing the effect of the load. For timber structures, the portion of dead load is generally small, which means that the majority of timber structures in practice falls in the range $0.5 < \alpha < 1.0$.

It should be noted that the assumptions in Table 1 are different from those historically used in the Nordic countries set up by NKB [4]. This means that absolute values of *the safety index β presented in the following should not be compared with the official target safety index given in the different countries.* In Sweden for instance, the target safety index for normal safety class (safety class 2 in the Swedish code) is set to 4.3, provided that it is evaluated according to the assumptions specified in NKB [4]. If instead the assumptions given in Table 1 are used, the target safety index for safety class 2 should be 3.7, as shown in [5].

Based on the failure function (5) and statistical modelling of the main variables, the associated safety index β can be calculated and represented as a function of the loading ratio α. The standard program COMREL is used for this. The relative magnitude of strength and load variables is determined by the associated code design equation (3). A more detailed description of the methodology is given in a separate report [6].

The effect on the reliability index α of the following aspects will be investigated in this study
- different statistical descriptions of strength
- different wood-based products and structural timber
- different Nordic codes and the Eurocode
- wood vs. steel
- effect of quality control (truncated distribution functions)

3. Effect of statistical modelling of the material

The idea with this investigation is to use the comprehensive strength database described in the section on Material Properties to see whether new and relevant statistical information about the lower tail of the strength distribution can be extracted. The statistical models of strength were directly derived from test data, and the effect of the statistical distribution function chosen to describe the test data on reliability was studied. The input data was taken from an experimental investigation of Finnish structural timber (spruce and pine) documented in [1], Table 2.10 and 2.11. In this study, 902 boards were visually graded to C24 and 1327 boards were machine graded to C30. The (non-parametric)

COV:s are 21.9% for visually graded C24 and 22.3 % for the machine graded C30. The test data is described by different statistical distribution functions with parameters fitted either using all data ("all data") or the lowest 10% of the data ("tail data"). The statistical models considered, were

- Normal distribution, tail data
- Lognormal distribution, all data
- Lognormal distribution, tail data
- 2-parameter Weibull distribution, tail data
- 3-parameter Weibull distribution, tail data

From safety point of view the values at the lower tail is most interesting. According to [1], the best fit to test data at the lower tail was obtained for Normal, 2-p Weibull and 3-p Weibull with parameters fitted to the lowest 10% of the data. Distributions fitted to all test data generally did not fit test data so well at the lower tail.

It is assumed that the 5th percentile of each distribution is exactly the characteristic value f_k given in the code and used in the design equation (3). The analyses were performed according to the Swedish Code, BKR99, [7], safety class 2 (normal). The corresponding partial coefficients are given in Table 2:

Table 2: Partial coefficients according to the Swedish Code BKR 99.

Swedish Code	
Partial Coefficient	Value
γ_G	1.0
γ_Q	1.3
γ_m	1.25
γ_n	1.1

The reliability index β is calculated and plotted against $\alpha$for the alternative statistical models. For the sake of comparison, a reference curve was also calculated for the case log-normal distribution with COV = 20%. This reference curve corresponds to a common assumption made in code calibration for structural timber, see [8].

Plots of β versus α for C24 and C30 are shown in Figures 1 and 2. Mean values of the reliability index for the interval 0.5<α<0.9 are given in Tables 3 and 4.

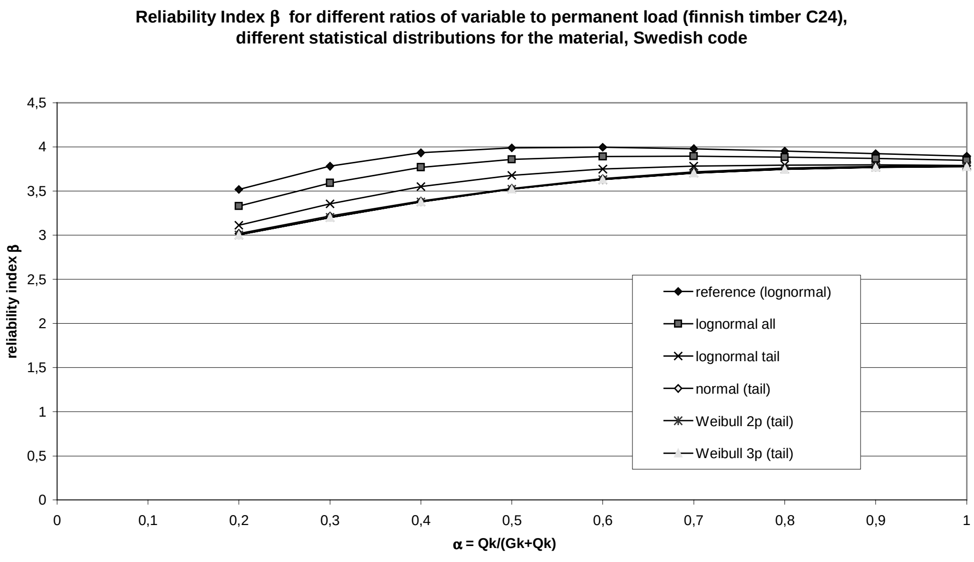

Figure 1: Reliability index β against ratio α for different statistical models of data for Finnish structural timber (C24).

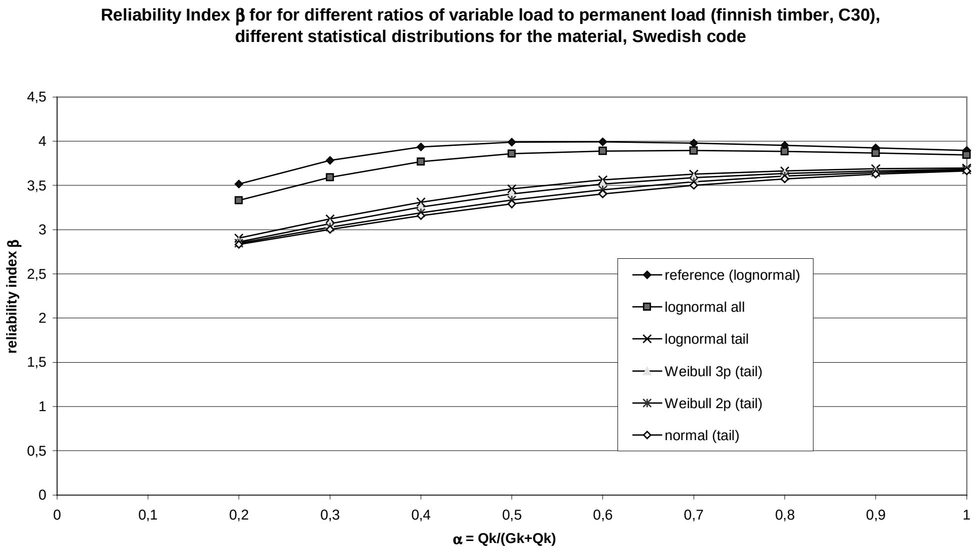

Figure 2: Reliability index β against ratio α for different statistical models of data for Finnish structural timber (C30).

Table 3: Mean reliability index β in the range 0.5<α<0.9 for different statistical models for Finnish structural timber C24.

material	Mean reliability index β C24
reference curve	3,9682
lognormal all	3,8792
lognormal tail	3,7598
normal tail	3,6852
Weibull 2p tail	3,6734
Weibull 3p tail	3,6732

Table 4: Mean reliability index β in the range 0.5<α<0.9 for different statistical distribution functions for Finnish structural timber C30.

material	Mean reliability index β C30
reference curve	3,9682
lognormal all	3,8672
lognormal tail	3,6014
Weibull 3p tail	3,5614
Weibull 2p tail	3,5162
normal tail	3,4802

The following observations can be made:

- The effect on safety index obtained by changing the statistical model for the same data is not very significant in the range $\alpha > 0.5$.
- The reference case (lognormal distribution with COV=20%) gives the highest values for β. One reason for this is that the actual COV of the tested material is slightly higher than 20%.

- When the parameters in the distribution are determined by fitting for the lower 10% of the data the reliability index is almost independent of the type of distribution.

- Statistical descriptions derived from tail data give generally lower safety index than the lognormal distribution with parameters derived from all data.

Although the lognormal distribution does not give the best fit of the lower tail of test data for structural timber it appears that this distribution describes data very well for other wood based products, such as glulam, LVL and plywood [1]. For this reason, strength is described by lognormal distributions in the further investigations presented in this report.

4. Comparison of different wood based materials

In this section, reliability indices of different wood-based materials and loading modes, calculated according to the Swedish code BKR 99, [7], and the Danish structural code, [9], will be compared. In all cases the statistical models of strength were directly based on material test data fitted to the lognormal distribution (all data). The characteristic value for the strength of each material is assumed be precisely the 5th percentile of the fitted distribution.

The materials considered were, see also Annexe 1,

- structural timber in bending (Finnish data [1], Danish data [2])
- structural timber in tension (Danish data [2])
- glulam (Norwegian data [3])
- finger joints (Norwegian data [3])
- LVL (Finnish data [1])
- Plywood (Finnish data [1])
- I-beams (Norwegian data [3])

For certain wood-based products (glulam, plywood, LVL and I-beams), the codes allow lower partial coefficients on the grounds that the material is controlled accurately under production and/or that they display low variability. For these products, $\gamma_m\gamma_n$ =1.27 in the Swedish code compared to $\gamma_m\gamma_n$ =1.38 valid for structural timber and joints (γ_n=1.1). In the Danish code the corresponding values are $\gamma_m\gamma_n$ =1.5 and $\gamma_m\gamma_n$ =1.64. The partial coefficient for variable load is higher in Denmark (γ_Q=1.5) than in Sweden (γ_Q=1.3).

Due to these differences between the two codes, the reliability index is generally higher for the Danish code than for the Swedish code, as can be seen in Figures 3 and 4, as well as in Tables 5 and 6.

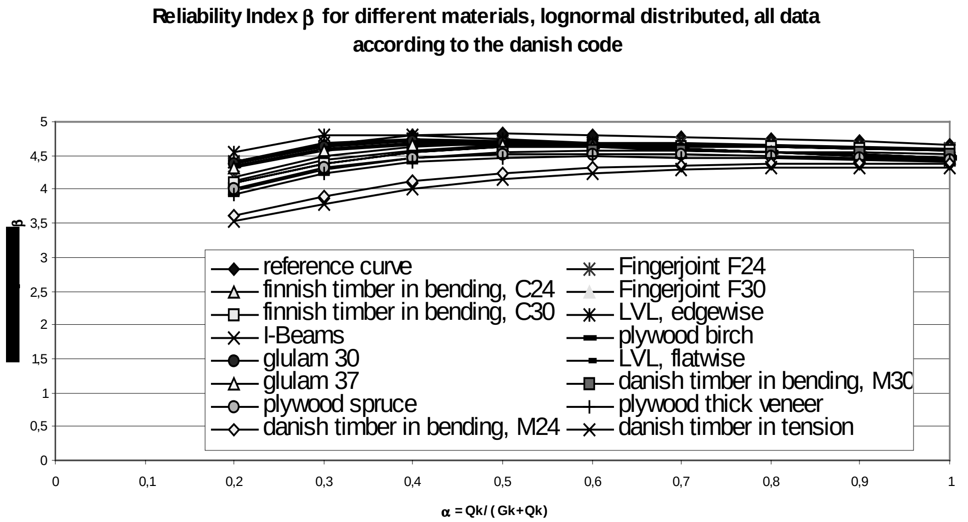

Figure 3: Reliability index for different wood-based materials, lognormal distribution, Danish code.

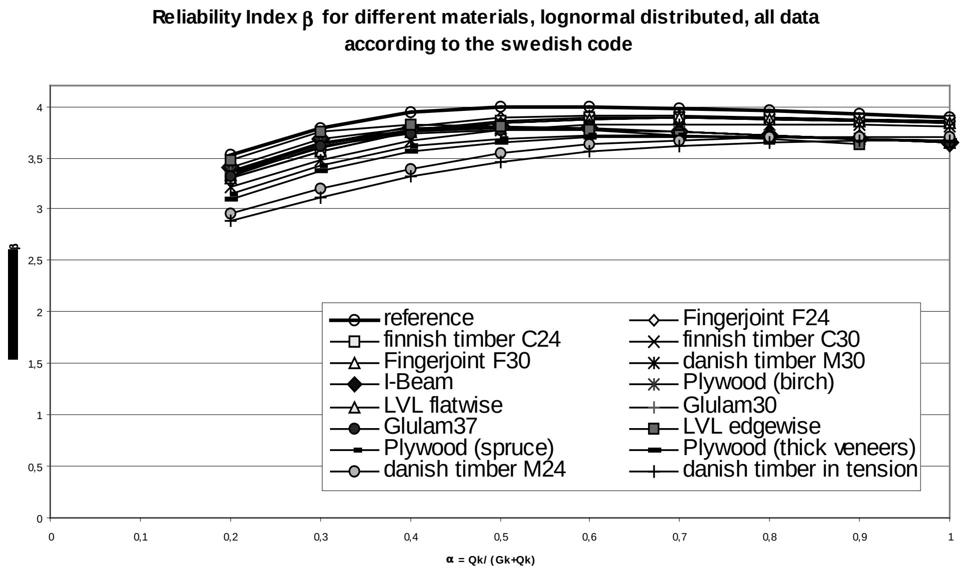

Figure 4: Reliability index for different wood-based materials, lognormal distributed material, according to the Swedish code.

Table 5: Mean reliability index β for 0.5<α<0.9 for different materials (all data) according to the Danish code.

material	Mean reliability index β for α= 0.5…0.9
Reference curve (structural timber)	4,7728
Fingerjoints F24	4,6734
Finnish timber in bending C24	4,6482
Fingerjoints F30	4,6266
Finnish timber in bending C30	4,6262
LVL edgewise	4,6122
I-beams	4,6080
Plywood Birch	4,6060
Glulam G30	4,6004
LVL flatwise	4,5950
Glulam G37	4,5884
Danish timber in bending M30	4,5572
Plywood Spruce	4,4994
Plywood Thick Veneer	4,4624
Danish timber in bending M24	4,3346
Danish timber in tension	4,2602

Table 6: Mean reliability index β for 0.5<α<0.9 for different materials (all data) according to the Swedish code.

material	Mean reliability index β for α = 0.5…0.9
Reference curve (structural timber)	3,9682
Fingerjoints F24	3,8946
Finnish timber in bending C24	3,8792
Finnish timber in bending C30	3,8672
Fingerjoints F30	3,8652
Danish timber in bending M30	3,8094
I-beams	3,7498
Plywood Birch	3,7402
LVL flatwise	3,7388
Glulam G30	3,7386
Glulam G37	3,7378
LVL edgewise	3,7200
Plywood Spruce	3,7028
Plywood Thick Veneer	3,6838
Danish timber in bending M24	3,6436
Danish timber in tension	3,5898

The following conclusions can be made on the basis of these results:

- The reliability level among this wide spectrum of materials is surprisingly uniform
- The results confirm that the concept of having a lower partial coefficient for materials like glulam, LVL, plywood and I-beams is reasonable.
- There is a small tendency, however, that wood products with high partial coefficient for the material (structural timber, joints) have higher reliability indices than those wood-based products with lower material partial coefficient.

- The reliability index according to the Danish code is generally higher than that for the Swedish code. The average difference is 0.80 with COV=7%.

The present study is focussed on the general safety elements in the structural codes relevant for wood based materials. In general the requirements for a particular product are governed through control of the characteristic value of strength. For a few of the materials in this study it was found that the characteristic value determined from tests did not meet the level required for the given strength class. This is not acceptable, but such problems must be eliminated by improved grading methods and general control programs or alternatively by adjusting the characteristic value. It is not logical to change the partial safety coefficients to deal with such problems.

5. Comparison of different Nordic codes and Eurocode

As every national building code has different partial coefficients on the load and the resistance side, the reliability level for a structure depends on the particular code used to design the structure.. The Danish and Swedish codes operate with three different safety classes (low, normal and high), whereas the Norwegian and Finnish codes as well as the Eurocode [10] have only one safety class (corresponding to the highest one) for all structures. Thus, for a relevant comparison, the highest safety class has to be used for the Danish and Swedish codes. The partial coefficients for materials, after transformation so that the partial coefficient for the permanent load becomes γ_G=1.0, are shown in Table 7.

Table 7: Partial coefficient, γ_m, for different materials and codes, after transformation so that γ_G=1.0

Material	Danish code	Finnish code	Norwegian code	Swedish code	Eurocode
Structural timber, Finger joints.	1,80	1,55	1,58	1,50	1,76
Glulam	1,65	1,55	1,32	1,38	1,69
Plywood, LVL	1,65	1,55	1,32	1,38	1,62
I-beam	1,65	1,55	1,32	1,38	1,62

The partial coefficient for the variable load is defined as the partial coefficient for snow load in this study, so that a comparison of the different codes is possible. The following values are obtained for the partial coefficient on the variable load, see Table 8.

Table 8: Partial coefficient for variable load (snow), γ_Q, for different codes, after transformation so that γ_G=1.0.

Danish code	Finnish code	Norwegian code	Swedish code	Eurocode
1,5	1,25	1,25	1,3	1,11

In the comparisons between codes the reference case for strength, i.e. lognormal distribution with COV=20%, is used. The corresponding code format used is that for structural timber, see Table 7. The results are displayed in Figure 5 and Table 9.

Table 9: Mean reliability index γ in the range α = 0.5-0.9 for different codes at the highest safety class. Material according to the reference case (structural timber, lognormally distributed, COV=20%).

Country Code	Mean reliability index β for α = 0.5…0.9
Danish	5,0436
Eurocode	4,3452
Norwegian	4,2812
Finnish	4,2268
Swedish	4,2232

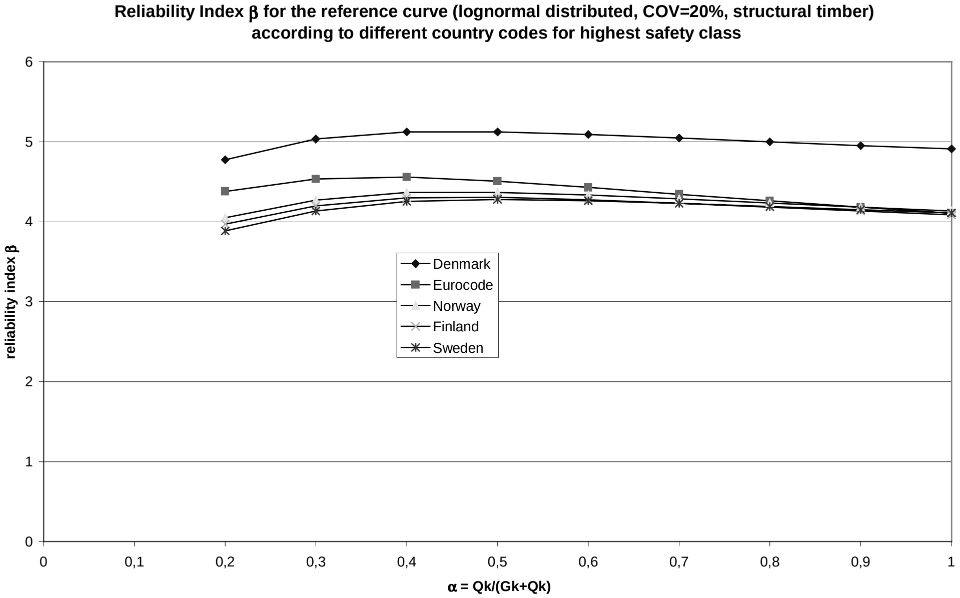

Figure 5: Reliability index for the reference curve (structural timber, LN) according to different country codes and the highest safety class.

The following conclusions can be drawn from the results:

- The reliability levels for the Eurocode and the current Swedish, Norwegian and Finnish codes are very close to each other for $\alpha > 0.4$.
- The reliability level for the Danish code is significantly higher than for all the others.
- The reliability level for Eurocode tends to be somewhat higher for small values of α, due to the relatively low value of the partial coefficient for variable load used in Eurocode.
- The reliability level tends to be fairly uniform with respect to the loading ratio α.

6. Comparison of wood and steel

To compare the reliability of timber structures with that of structures made from other materials, analyses of the reliability of steel structures is made here. A benchmarking against steel structures may indicate whether differences between countries can be attributed to a general difference in safety level among the codes or if it has to do with the specific rules set down for wood based structures.

The studied material here is steel S235 with the following assumptions:

- The characteristic (nominal) value equals the 5th percentile
- The strength of steel is assumed lognormal with a coefficient of variation COV=7%, according to the model code published by the Joint Committee on Structural Safety (JCSS), [11].
- For the Swedish Code, also the case with f_k = 1 %-percentile is studied according to the definition of the characteristic yield strength in BKR 99 [7].

The corresponding partial safety coefficients for loads and for resistance of steel are summarised in Table10. The values are valid for the high safety class in the Danish and Swedish codes.

Table 10: Partial safety coefficients for the different codes.

	Partial safety coefficient for steel		
Country Code	γ_m (steel)	γ_G	γ_Q
Danish	1.29	1.0	1.5
Finnish	1.00	1.2	1.6
Norwegian	1.10	1.2	1.5
Swedish	1.20	1.0	1.3
Eurocode	1.10	1.35	1.5

To simplify the reliability analyses, all safety coefficients are related to γ_G =1.0, so that the values in Table 11 are obtained.

Table 11: Partial safety coefficients for the different codes (for steel), related to γ_G =1.0.

	Partial safety coefficient for steel		
Country Code	γ_m	γ_G	γ_Q
Danish	1.29	1	1.50
Finnish	1.20	1	1.33
Norwegian	1.32	1	1.25
Swedish	1.20	1	1.30
Eurocode	1.485	1	1.11

The reliability index for steel as a function of the ratio α of variable load to total load is displayed in Figure 6. The average values of β for α in the range 0.5 to 0.9 for wood (reference case taken from Table 9) and steel are summarised in Table 12.

Table 12: Mean reliability indices β_{mean} for wood (reference curve, structural timber) and steel. Ratio between values for wood and steel.

Country Code	$\beta_{mean,wood}$	$\beta_{mean,steel}$	$\beta_{mean,wood}$./$\beta_{mean,steel}$
Danish	5,0436	4,0620	1,242
Eurocode	4,3452	3,8036	1,142
Norwegian	4,2812	3,6946	1,159
Finnish	4,2268	3,5274	1,198
Swedish ($f_{k,steel}$=1% percentile)	4,2232	3,6310	1,163
Swedish ($f_{k,steel}$=5% percentile)	4,2232	3,4716	1,217

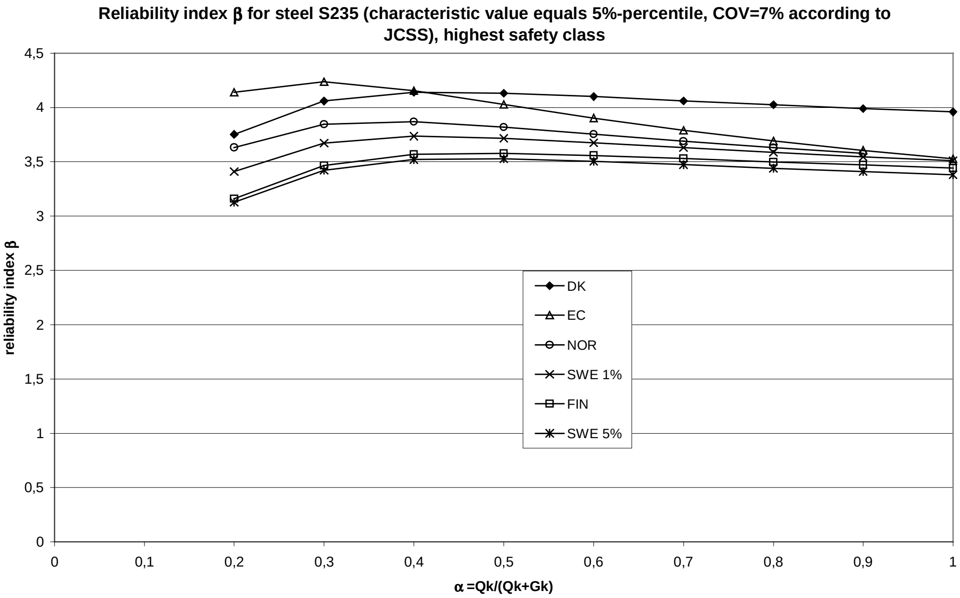

Figure 6: Reliability index β for steel S235 for different country codes, highest safety class.

The following conclusions can be drawn from these results:

- For all codes, the reliability level is significantly higher for timber than for steel.
- This difference is larger for the Danish code, than for Eurocode and the other Nordic codes.
- The high reliability level observed for timber in the Danish code is consequently only partly related to a generally higher safety level for structures used in Denmark.
- For the Swedish code the low partial coefficient used for steel is motivated by the assumption that the characteristic value corresponds to the 0.01 percentile, instead of the 0.05 percentile.

These results lead to two important questions:

1. Why is the reliability level so much higher for timber than for steel in all the codes?
2. What is the background for the extra high safety level chosen for timber in the Danish code?

7. Effect of proof loading and quality control

The effect of quality control can be simulated with truncated distributions. It is often argued that boards with very low strength will automatically be detected and taken away when structural timber is graded by machines based on measurement of stiffness in flatwise bending. An alternative, which is sometimes advocated, is to perform proof loading of all timber up to a certain stress level, so that all boards with strength below this level are taken away. To investigate such effects, a reliability analysis with a truncated 2-parameter-Weibull-distribution was performed for structural timber C30 (Finnish data). The non-parametric characteristic value (5^{th} percentile) for this sample of 1327 boards is 30.6 MPa. To study the effect, different lower boundaries *a* for truncation were used and the results were compared with those obtained with the untruncated 2-parameter Weibull distribution. The results are plotted in Figure 7. The average safety levels in the range 0.5-0.9 for the loading ratio α are shown in Table 13.

General conclusions from this study are:

- The reliability index increases with increasing value for the lower boundary of the distribution
- For low lower boundaries (e.g. a=15 MPa, which is about half of the characteristic value), the reliability index is only marginally higher than for the untruncated Weibull-distribution
- For boundaries corresponding to 20 and 25 MPa the truncation gives a significant effect on the reliability level.

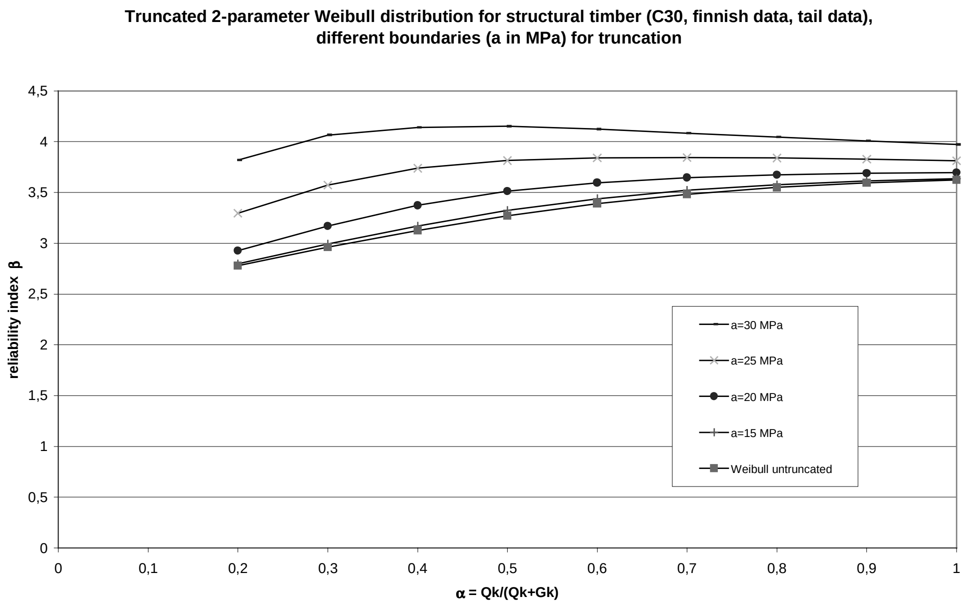

Figure 7: Reliability index for truncated Weibull distribution with different values for lower and upper boundaries.

Table 13: Mean reliability index β for α = 0.5…0.9 for truncated 2-parameter Weibull-distribution (Finnish structural timber, f_k =30.6 MPa).

Truncation boundary [MPa]	Mean reliability index for α = 0.5…0.9	Increase in reliability index %
30	4,08	18
25	3,83	11
20	3,62	5
15	3,50	1
Original Weibull	3,46	0

8. References

[1] Ranta-Maunus A., Fonselius M., Kurkela J. and Toratti T. *Reliability analysis of timber structures*, VTT Building and Transport, Research Notes, Draft, March, 2001, Espoo 2001.

[2] Dalsgaard Sörensen, J., Hoffmeyer, P. *Statistical Analysis of Data for Timber Strengths*. Draft report, 2001.

[3] Norsk Treteknisk Institutt. Norwegian strength data, Draft report, 2001

[4] Nordic committee for Building Structures (NKB) ”Recommendation for Loading and Safety Regulations for Structural Design”, NKB Report no. 55, 1987.

[5] Svensson, S. Thelandersson, S., Aspects on reliability calibration of safety factors for timber structures. Holz als Roh und Werkstoff, 2001. In press.

[6] Frühwald, E., Thelandersson, S., Reliability analysis of safety elements in Nordic codes for timber structures. Div. of Structural Engineering, Lund University. To be published, 2001.

[7] BKR 99: Boverkets konstruktionsregler, BFS1993:58-BFS 1998:39, 1999.

[8] Thelandersson S., Larsen H. J., Isaksson T., Svensson S., Säkerhetsnivåer för trä och träprodukter i konstruktioner.(inSwedish). Report TVBK-3039, Div. of Structural Engineering, Lund University, 1999.

[9] Reference to Danish code for structures.
[
10] Eurocode 5. Design of timber structures. Part 1-1. Final Draft 2001-04-09.

[11] Joint Committee of Structural Safety ”JCSS Probabilistic Model Code”, Internet Publication, 2001.

Annex 1: Databases used in the study

- **Finnish structural timber in bending**
    - Finnish data (VTT-Report, Tables 2.10 and 2.11)
    - Machine graded (C30) and visually graded (C24)
- **LVL**
    - Finnish data (VTT-Report, Tables 2.13 and 2.14)
    - Bending strength edgewise and flatwise
- **Plywood**
    - Reference [1] (VTT-Report, Tables 2.17 and 2.18)
    - Birch, spruce, plywood of thick veneers
    - Flatwise bending tests
- **Structural timber in bending, Danish data**
    - Reference [2] (Sörensen, Hoffmeyer, Database F)
    - Norway spruce, scots pine, bending test, Dynagrade, M24 and M30
- **Structural timber in tension, Danish data**
    - Reference [2] (Sörensen, Hoffmeyer, Database A)
    - Norway spruce glulam laminations, tension test, Cook-Bolinder / Computermatic, M30
- **Glulam**
    - Norwegian data (Norsk Treteknisk Institutt)
    - G 30, G 37, bending tests
- **Fingerjoint**
    - Norwegian data (NTI)
    - F 24, F 30, edgewise bending tests
- **I-Beams**
    - Norwegian data (NTI)
    - Bending tests, bending stresses converted to tension and compression stresses in the flanges

Nordic Wood: Reliability of timber structures

Summary report on existing strength data

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1 Modelling strength data for reliability analysis

The purpose of the strength data collection and analysis was to find more information, which is needed in the reliability analysis of timber structures. First, a sensitivity analysis revealed that the calculated reliability is sensitive to the lowest strength values, whereas the values around the mean have no effect. Therefore, in order to get correct information on the reliability of structures, we need an adequate sample size, and the distribution function should fit well to the lowest values in the relevant population. Considering the test data available, a population of 1 000 test data can be considered adequate, and the population can consist of a combination of different test series. Then the distribution functions can be fitted to the lower tail, e.g. 10%, of the values, and used both to determine the characteristic 5th percentile value and to estimate the structural reliability.

In some cases it was observed that machine-graded sawn timber had too low a 5th percentile value, which is supposed to be a signal of an error in grading. This has to be counteracted by improving the calibration of grading machines or grading technology. We do not propose that this kind of error in research equipment functions should be considered in the structural reliability analysis.

2 Summary of strength data

Nordic project partners have collected and analysed such existing strength data of timber materials to which they have access. We have analysed the bending data of sawn and round timber, LVL, glulam, finger joints, I-beams and plywood. The tension strength results of glulam lamellae, and compression data of round timber have also been analysed in the project

In this summary report, only the results obtained from the largest samples are included. From the sawn timber data, only machine-graded timber with a sample size N > 500 is included, with the exception of a sample of Irish-grown sitka spruce (N = 386), in order to include some results other than Scandinavian. The results of visually graded timber are not included because of the low yield of the method. The largest population of sawn timber we analysed comprised 1 300 specimens.

From Kerto LVL we have nearly 2 000 quality control specimens both in edge-wise and flat-wise bending. From tension tests of glulam lammellae a sample of 1 000 specimens were available, and 600 for bending of finger joints. For small-diameter round timber, about 600 bending and compression test samples have been analysed.

The samples for other materials are unfortunately smaller. Since no other information was available, the following samples are also reported here: plywood (281), glulam (126 + 109), and I-beam (294).

Strength distributions are illustrated on a relative scale in Figure 1, where all strength values are divided by the 5th percentile. For comparison, curves for lognormal distribution with COV = 10, 20 and 30% are shown as well. The upper figure with linear probability scale shows the differences above characteristic value, whereas the smallest strength values can be compared when logarithmic scale is used (lower figure).

The results are reported in detail by Ranta-Maunus et al. (2001), Sorensen and Hoffmeyer (2001) and NTI (2001). A summary is provided here concerning:

- how well the samples meet the target characteristic (5 percentile) value, and
- the parameters of distribution functions fitted to the data in terms of COV.

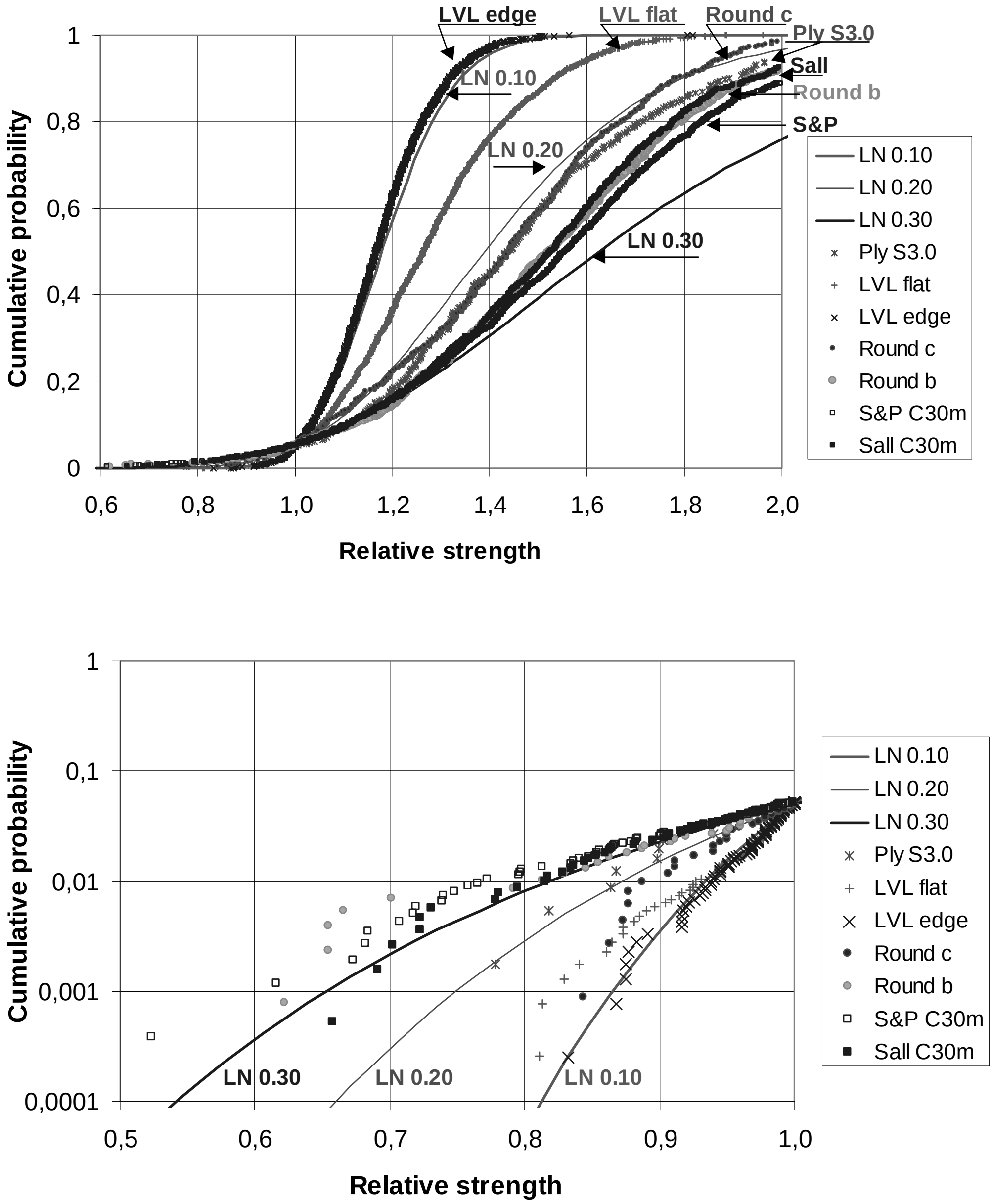

Figure 1. Cumulative probability distributions of relative strength (strength per 5th percentile) of sawn and round timber, LVL and plywood on linear and logarithmic scale as well as lognormal function with COV = 0.1, 0.2, and 0.3.

When strength data are used in the reliability analysis, it is essential that the distribution function used fits well with the lower strength values; otherwise the reliability values are misleading. Therefore, we fitted distribution functions separately to all the data and to the lower tail, 10 % in many cases. If both fittings gave nearly the same result, we concluded that this material follows the distribution in question, and we could use the parameters obtained from any of the fittings. For LVL and plywood we obtained nearly the same COV when fitting lognormal distribution to all the data and to the lower tail. On the other hand, the sawn timber results show a flatter tail than that of lognormal distribution, only a little steeper than normal distribution. When we used lognormal distribution to describe this data, we used COV based on the fitting to lower tail.

The results of analysis are shown in Tables 1 (sawn timber) and 2 (others) concerning the 5 percentile value observed versus the target value of the grade, and the COV parameters of normal, lognormal and 2-parameter Weibull distributions fitted to the lower tail data. The 5th percentile value is based to non-parametric distribution, which is the method used in the EN-standards for sawn timber. In most cases, the 5th percentile is close (a little above) the target value with some exceptions:

- one set of sawn timber data gives 10 % too low a 5th percentile
- two sets of finger joint data give 5 and 17 % too low a value
- two sets of tension lamellae give 3 to 4 % too low a value
- glulam and plywood had a 10% higher 5th percentile than the grade value.

The result is contradictory for glulam: the testing made by its constituents, lamellae and finger joints, suggests that there could be a problem in the strength of glulam, whereas the strength of glulam exceeds the code value. We should obtain more data on glulam in order to be able to draw conclusions on the tail data.

Visually graded sawn timber, which is not reported here, gives normally a higher 5th percentile value than needed for the grade. Therefore, this traditional method can be considered conservative but uneconomic. Another problem associated with the tests of visually graded timber is that the grading is made in the laboratory, indicating the conservatism of the grading rules rather than the high strength of commercially produced material.

When normal, lognormal and 2-parameter Weibull distributions are fitted to the lowest 10% of the results, the COV parameters related to these functions are quite different, as shown in Tables 1 and 2. Sawn timber, which has COV of the whole test data from 21 to 29 %, has the COV parameters of tail-fitted distributions as follows:
Normal distribution: 18 – 24 %
Lognormal distribution 29 – 35 %
2-parameter Weibull 14 – 21 %

Distributions fitted to the lower tail of the tension strength of lamellae, and bending strength of finger joints and round timber has similar COV parameters as sawn timber.

Engineered wood products, and round timber in compression had smaller COV values. Ungraded small-diameter round timber, which had a COV of all data of 23 %, obtained a tail-fitted COV parameter of lognormal distribution as low as 18%. In engineered products, the COV of tail-fitted lognormal distribution is close to the COV of the entire data. For LVL we obtained a COV around 10%. For other EWPs the sample size should be larger so that we can draw firm conclusions on the shape of distribution tail.

3 Recommendations

Based on the analyses performed, the following recommendations are made:

The data available suggests that engineered wood products follow well the lognormal distribution, and sawn timber could be better described by normal or Weibull distribution. However, it is suggested that lognormal distribution is used for all timber materials in structural reliability analysis, because it is widely used for other materials and because it seems to be the best for timber materials used for long span structures as well.

When more specific information is unavailable, the COV parameters of lognormal distribution can be taken from Table 3. It has to be observed that the data used in this work was based on the testing of:

- mainly Nordic sawn timber
- Kerto-LVL
- Finnish 3 mm-ply spruce plywood (only 300 specimens)
- Norwegian I-beams (only 300 specimens)
- Norwegian glulam (only 100 + 100 specimens).

It would be valuable, especially for glulam, which is used in long-span structures, if a much bigger population were analysed.

Table 3. Suggested values for COV-parameter of lognormal distribution when used in structural reliability analysis.

Material	COV [%]
Machine graded sawn timber	30
Plywood*)	20
Glulam*), I-beam*)	15
LVL	10

*) inadequate population (N < 300).

References

Dalsgaard Sorensen J., Hoffmeyer P., 2001, Statistical Analysis of Data for Timber Strengths. Manuscript.

Ranta-Maunus A., Fonselius M., Kurkela J., Toratti T., 2001, Reliability analysis of timber structures. VTT Research Notes 2109. Espoo, Finland. 102 p + app. 3 p

NTI, 2001,Internal database. Communication in Nordic project.

Table 1. Collection of machine-graded sawn timber bending strength distribution data, and bending and compression data of ungraded small- diameter round timber. The type of distribution fitted to the lower tail data is given as well as the COV parameter of the fitted distribution, the tail used for fitting as % of total sample and $f_{0.05}$ based on nonparametric distribution.

Species	Origin	Grade	$f_{0.05}$ [N/mm^2]	Grading method	Sample size	Tail fitted [%]	Distribution type	COV [%]	Reference
Spruce	Finland	M30	30.5	Bending	496	10	Normal Lognormal 2-P Weibull	18 29 14	Ranta-Maunus et al. 2001 S-1 in Table 2.3
Spruce	Finland	M30	31.3	Bending	986	10	Normal Lognormal 2-P Weibull	19 31 15	Ranta-Maunus et al. 2001 S-1 to S-99 in Table 2.7, “Sall” in Figure 1
Spruce and pine	Finland, Sweden	M30	30.6	Bending	1327	10	Normal Lognormal 2-P Weibull	20 35 17	Ranta-Maunus et al. 2001 Table 2.10, “S&P” in Figure 1
Spruce and pine	Sweden and Finland	M24	24.6	Dynamic	819		Normal Lognormal 2-P Weibull	24 35 21	Dalsgaard Sorensen, Hoffmeyer Table 6.11, Series F all
Sitka spruce	Ireland	M30	27.1	Bending	386	30	Normal Lognormal 2-P Weibull	23 34 21	Dalsgaard Sorensen, Hoffmeyer Table 7.2, Series H, Cook Bolinder
Small round timber, bending	Finland, UK, Austria		36.6	None	660	10	Normal Lognormal 2-P Weibull	20 34 16	Ranta-Maunus et al. 2001 Table 2.20, Spruce and pine, “Round b” in Figure 1.
Small round timber, compression	Finland, UK		17.8	None	575	10	Normal Lognormal 2-P Weibull	13 18 9	Ranta-Maunus et al. 2001 Table 2.20, spruce and pine, “Round c” in Figure 1.

Table 2. Collection of EWP (plywood, LVL, I-beam, glulam) strength distribution data together with lamellae tension and finger joint bending results. The type of distribution fitted to the lower tail data is given as well as the CO -parameter of the fitted distribution, the tail used for fitting as % of total sample and $f_{0.05}$ based on nonparametric distribution. Grade value is the expected 5th percentile strength according to the grade.

Product	Origin	Target $f_{0.05}$ [N/mm^2]	$f_{0.05}$ in test [N/mm^2]	Explanation of test	Sample size	Tail fitted [%]	Fitting distribution	COV [%]	Reference
I-beam	Norway	24	25.8	Tension / compression of flange in standard bending test	294	10	Normal Lognormal	12 17	NTI
Finger joint	Norway	24	22.8	Edgewise bending	620	10	Normal Lognormal	20 33	NTI
Finger joint	Norway	30	24.9	Edgewise bending	220	10	Normal Lognormal	27 57	NTI
Glulam	Norway	30	33.5	Edgewise bending	126	10	Normal Lognormal	11 13	NTI
Glulam	Norway	37	39.9	Edgewise bending	109	10	Normal Lognormal	14 19	NTI
Glulam lamellae	Scandinavia	20	19.2	Tension	1098	30	Normal Lognormal 2-P Weibull	21 30 18	Dalsgaard Sorensen, Hoffmeyer Table 3.15, Cook Bolinder
Glulam lamellae	Scandinavia	20	19.4	Tension	1079	30	Normal Lognormal 2-P Weibull	21 30 18	Dalsgaard Sorensen, Hoffmeyer Table 3.16, Computermatic
Glulam lamellae	Scandinavia	16	17.0	Tension	549	30	Normal Lognormal 2-P Weibull	22 33 20	Dalsgaard Sorensen, Hoffmeyer Table 3.17, Dynadrade
LVL	Spruce, Kerto	50	51.3	Edgewise bending	1968	10	Normal Lognormal 2-P Weibull	8 9 5	Ranta-Maunus et al. Table 2.13, “LVL edge” in Figure 1.
LVL	Spruce, Kerto	50	50.3	Flatwise bending	1963	10	Normal Lognormal 2-P Weibull	10 12 6	Ranta-Maunus et al. Table 2.13, “LVL flat” in Figure 1.
Plywood	Spruce, 3 mm ply	30	33.6	Flatwise bending	281	10	Normal Lognormal 2-P Weibull	16 23 11	Ranta-Maunus et al. Table 2.17, “Ply S3.0” in Figure 1.

Nordic Wood:Træ og sikkerhed

Effect of load duration on Timber structures in Denmark

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Introduction

Strength reducing effects due to long term loading at high stress levels are known for most materials. This effect is referred to as creep-rupture effects. Wood materials are highly affected by this reduction in strength with time. Therefore, a strength-reducing factor is typically used in the design of timber structures. Many codes refer to this strength modification as k_{mod}. Traditionally, the modification factor is determined empirically from experience on timber structures and knowledge of material properties of wood. In this report, a probabilistic safety philosophy is applied to determine the modification factor on a theoretical background.

Three types of loading, live load, snow load, and wind load have been considered representing medium-long term, medium-short term, and instant load, respectively. Also, load combinations of 25% permanent load and 75% variable load have been investigated.

Procedure of calibrating the reduction factor k_{mod} using probabilistic analysis

Most building codes, national and international, are based on a probabilistic safety philosophy. The required safety is, however, usually not accomplished by using probabilistic theories in everyday design. Instead, partial safety factors in a deterministic design format are calibrated for standard cases against probabilistic analyses for the same cases. The condition for calibration is that the probabilistic analysis and the deterministic code fulfil the same requirements of safety.

Presented here, the procedure to calibrate k_{mod} is based on the same probabilistic safety theory and makes use of the same probabilistic method as the calibration of code values. To perform probabilistic analyses requires statistical information describing the studied parameters, e.g. load and load carrying capacity. Obtaining statistical data of the influence of duration of load on the load carrying capacity for solid wood and transforming this into a parameter suitable for probabilistic analysis are accomplished.

Definition

The factor k_{mod} is defined in the Danish code (DS413) as a modification factor taking into account the effects of load duration and ambient climate on the strength parameters of structural elements. Another definition, sometimes preferred, is that k_{mod} insures that the safety of a structure, designed for annual maximum load prevails also for the loads duration and in combination with action caused by climate. The modification factor is of course only used in cases where the materials utilized, as part of the load carrying structural elements, are sensitive to or damaged by duration of load and/or ambient climate. The code requirements for limit state design may be written as:

$$\frac{k_{mod} R_k}{\gamma_R} \geq S_k \cdot \gamma_S \qquad (1)$$

where R_k is the characteristic load carrying capacity, S_k is the characteristic load, γ_R and γ_S are the partial safety factors for load carrying capacity and loads, respectively. The characteristic value of the load carrying capacity originates from the 5% fractile value of the short-term strength of the studied structural element, in this study bending strength of solid wood. The characteristic value of the load is determined using the distribution function of the maximum load level per year (maximum annual load). The fractile value for characteristic load depends on load type e.g. the mean value is used for permanent load and the 98% fractile value is used for time variable load such as snow load, wind load and live load.

A material used in a load carrying structure will have the same initial characteristic features independently of the load situation. In a design situation, the load carrying capacity is determined for maximum annual load (characteristic load). Hence, the whole effect of load duration is accounted for by the k_{mod} factor in Equation 1. This implies that k_{mod} is the ratio between the characteristic value of the load carrying capacity, R_{k1}, required to sustain annual maximum (short-term) loads and the characteristic value of the (initial) load carrying capacity, R_{kT}, required to sustain load sequences over a period of time, T. Assuming that the coefficient of variation for the initial load carrying capacity is independent of duration of load, evaluation of equation 1 for the case with load duration will lead to the following mathematical expression of k_{mod} (Svensson et al. 1999).

$$k_{mod} = \frac{R_{k1}}{R_{kT}} = \cdots = \frac{\mu_R}{\mu_{RT}} \qquad (2)$$

where μ_R is the mean value of the required load carrying capacity for maximum load and μ_{RT} is the mean value of required load carrying capacity for a load with duration, T. The period for duration of load is in this study equal to the service life of the structure, which is in Denmark 50 years.

The mean values of required load carrying capacities for the two load cases might be determined by using traditional probabilistic analyses. For the case of required capacity for maximum load this is straightforward procedure when the annual maximum loads are known. For the case of required capacity for load with duration it is in the present investigation decided to work with an equivalent load. The equivalent load represents the effects given to a defined structural element by a load with duration. In the following the method of deriving an equivalent load will be explained.

Damage accumulation models

Damage models are used to mathematically describe the damage as a function of stress level and duration of loading. In the literature a small selection of damage accumulation models are found. Thelandersson *et al.* (1999) provides a more detailed description of some damage models. In this study two models are chosen. These models are probably the most frequently used and have been evaluated thoroughly in other studies as well. The characteristics of the damage models are that α is defined as the degree of damage, i.e. $\alpha = 0$ stands for no damage and $\alpha = 1$ stands for total damage or failure. Equation 3 shows the damage accumulation model presented by Gerhards (1979).

$$\frac{d\alpha}{dt} = \exp\left\{ A + B\frac{S(t)}{R_{ini}} \right\} \qquad (3)$$

where A and B are constants, $S(t)$ is the load in time and R_{ini} is the initial load carrying capacity of the studied beam. This model is one of the models used in the study to determine effects of load duration. The other damage accumulation model applied here is the one presented by Barrett and Foschi (1978). This model has the following mathematical expression,

$$\frac{d\alpha}{dt} = A\left(\frac{S(t)}{R_{ini}} - \eta\right)^{B} + C\alpha \quad ; \frac{S(t)}{R_{ini}} > \eta$$
$$\frac{d\alpha}{dt} = 0 \quad ; \frac{S(t)}{R_{ini}} \le \eta \qquad (4)$$

where A, B, C are constants, η is a threshold ratio, $S(t)$ is the load in time and R_{ini} is the initial load carrying capacity of the studied beam.
Both equations 3 and 4 have been modified from the original proposal where the equations were written from a stress and strength perspective. The modified equations have, for convenience, a load and load carrying capacity perspective.

The constants describing the influence of time on long term strength in the damage models are determined by curve fitting on test results. Often, reference to data is made to the well-known Madison curve (Wood 1951), which is based on results from bending tests on clear wood specimens. Lately, a test program was carried out on structural timber (Norway spruce) in 4 point bending (Hoffmeyer 2001). Traditionally, long-term tests have been made in bending, but according to Wood Handbook (1999) a similar behaviour can be found for tension and compression along the fibres. The constants used for the damage models in the present investigation are shown in Table 1.

The fit constants for Nordic grown timber are found by the Maximum Likelihood Method. A stochastic variable (e) representing "lack of fit" was included in the curve fits. The mean of e is 0 and the standard deviation is 0.02 for Gerhards' model and 0.008 for the model by Barret and Foschi. The constants are all determined with a cov of less than 5%. These uncertainties of the damage models have not been taken directly into consideration in the model. The models are, however, including an over-all model uncertainty (see Table 4).

Table 1Results on curve fits on Madison data and on Nordic data for two damage models. The constants refer to time measured in hours

	Madison data	Nordic data	
	Gerhards model	Gerhards model	Barret and Forschi's model
A	32.5	41.9	$2.74{\cdot}10^{6}$
B	36.0	46.5	15,9
C	-	-	$1.08{\cdot}10^{-4}$

Load generation

The load models for time variable load such as snow, wind, and live load used in this study are described later in this report. When a load model is determined and the statistical description of load magnitude, (annual) occurrence frequency and duration are known it is possible to generate any load sequence representative for the studied load. Here, simple Monte-Carlo simulations are used to randomly select loads based on the statistical description of the particular load. These loads assembled over the service life of the structure in question are the load sequence used when determining the influence of load duration on the structure. An example of a load sequence of annual snow load in combination with permanent load is shown in Figure 1.

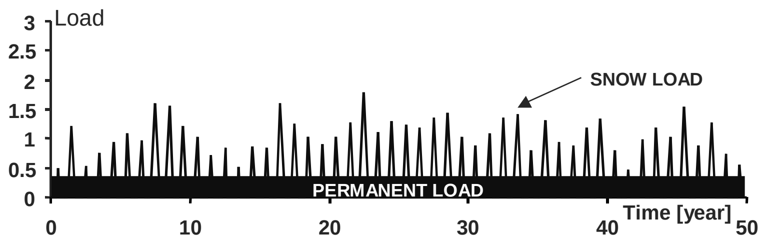

Figure 1 Load representing permanent load and snow load for a service life (50 years) of a building. The load sequence is simulated on the basis of measured data.

Equivalent load

With a damage accumulation model it is possible to calculate the damage in a timber structure caused by arbitrary load acting over a time period, e.g. a sequence of annual snow loads. For a given sequence, i, of annual loads over N years a structure will survive if the initial load carrying capacity is larger than a minimum value $R_{min,i}$. This minimum value is determined for each random load sequence by use of a numerical method proposed in Forsythe et. al. (1976). By this procedure a given load sequence, i, can be transformed to an equivalent time independent load $S_{equ,i}$ given by:

$$S_{equ,i} = R_{min,i} \tag{5}$$

Load sequences are generated over a period of 50 years by 200 simulations for each of the variable loads, snow, wind, and live load. Similar sequences are generated for load combinations of 25% permanent load and 75% variable load (snow, wind, and live load, respectively). This is done for each of the two damage models presented above. For every simulated load sequence the equivalent load is derived. This makes the equivalent load, S_{equ}, for each variable load or variable load in combination with permanent load a random variable with expected value, μ_{Sequ}, and a standard deviation, σ_{Sequ}.

Required bending capacity for long-term load and k_{mod}

To determine the required bending capacity with respect to the equivalent load the same procedure applied to determine the required bending capacity for the annual maximum load may be used. The reference time of the load is, however, changed from one year for the annual maximum load, S, to fifty years for the equivalent load, S_{equ}. This gives another required probability of failure for the structure. If the annual loads are assumed un-correlated, the probability of failure for a reference time of fifty years, p_{f50}, can be determined as:

$$p_{f50} = 1 - (1 - p_f)^{50} \tag{6}$$

where p_f is the probability of failure for loads with a reference time of one year. When required bending capacity is determined with respect to the equivalent load, the effects of load duration on structural elements are known. The factor k_{mod} is then found by solving equation (2).

Live loads

The live load model used in this study is the model proposed in (CIB 1989) and recommended by Joint Commission of Structural Safety. The model contains two parts, sustained live load and intermittent live load. The sustained live load covers ordinary live load such as furniture, average utilisation by persons, etc. The intermittent live load describes the exceptional load peaks, e.g. furniture assembly while re-modelling, people gathering for special occasions, etc. The live load model is by no means a precise description of the real load process, but it is a good description of the effects caused on a structure by the load action. In most cases, a so-called Equivalent Uniformly Distributed Load (EUDL) can describe the live load parts. The EUDL load has a mean value, which can be seen as the mean load for an area of a given reference size in buildings of the same type, office, classroom, residence, etc. The variation of the EUDL-load is defined in two parts, one variation between the reference areas and one spatial variation within the reference area. This may be described with a fairly simple hierarchy load equation as shown in Equation 7.

$$E[q_{EUDL}] = \mu$$
$$V[q_{EUDL}] = \sigma_r^2 + \sigma_{sp}^2 \cdot \kappa \cdot \frac{A_0}{A} \tag{7}$$

where μ is the mean value and σ is the standard deviation for which the numerical values will vary. Indices r and sp denote reference area and spatial area, respectively. The mean value and the standard deviation are depending on the type of load. Data are shown in table 2. A_0 is the FBC area, defined as an area of approximately 2 m^2 where no variation is considered, A is the load contributing area for the structural element studied, and κ is the peak factor defined in Equation 8.

$$\kappa = A \frac{\int_A I(x,y)^2 \, dA}{\left\{ \int_A I(x,y)\, dA \right\}^2} \tag{8}$$

where $I(x,y)$ stands for the influence function of the studied load effect. Considering a floor structure based on wood joists and plywood sheathing the influence function may be assumed to have the shape of a pyramid. The base, A, of the pyramid is covering the spacing of two joists and the span of the joists as sketched in Figure 2. The peak factor, κ, in the centre of A is equal to 1.778 for the pyramidal shape and according to Equation 8.

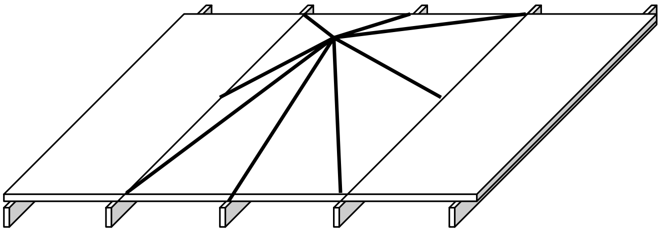

Figure 2. Influence function visualised for the mid-span bending moment of a joist in the floor structure.

Table 2. Data on live load according to JCSS. In the table, μ and σ refer to mean load and standard deviation in Equation 7. The factor $1/\lambda$ is the average duration, and ν is the occurrence rate. Indices *sus* and *int* denote sustained load and intermittent load, respectively, whereas indices r and sp refer to reference area and spatial area.

Type of building	Ref. Area	μ_{sus}	$\sigma_{sus,r}$	$\sigma_{sus,sp}$	$1/\lambda_{sus}$	μ_{int}	$\sigma_{int,sp}$	ν_{int}	$1/\lambda_{int}$
	[m^2]	[kN/m^2]	[kN/m^2]	[kN/m^2]	[year]	[kN/m^2]	[kN/m^2]	[year^{-1}]	[hour]
Residence	20	0.3	0.15	0.3	7	0.3	0.6	1	48
Office	20	0.5	0.3	0.6	5	0.2	0.4	3.3	48

Both sustained load and intermittent load are modelled by Equation 7. The two loading types are added together providing that the total live load for the studied cases is derived. In the study, a floor structure based on joists with a span of 3.5-5.5 m and a spacing of 0.6 m is considered. The reference area, A, is assumed to cover the span and twice the spacing of the joists.

Both sustained load and intermittent load intensity are assumed gamma distributed. The duration of the loads are assumed to be exponential distributed and the occurrence of the intermittent load follows the Poisson distribution. The numerical values are presented in Table 2 for the different distributions used to describe intensity, duration, and occurrence. Table 3 shows some results derived when using the model for live load described above.

Table 3. Loads according to CIB (1989) and JCSS normalized against mean total load

Type of building	Annual max			50 year max	
	mean	COV	98% quantile	mean	COV
	[kN/m^2]	[-]	[kN/m^2]	[kN/m^2]	[-]
Residence	0.55	0.97	2.31	3.31	0.28
Office	0.96	0.78	2.98	3.77	0.29

Snow load in Denmark

The investigation includes an estimate of snow load in Denmark and the corresponding duration of each snow pack. The snow load is modelled from meteorological data determined at 5 locations in Denmark over a period of 32 years. The meteorological data are used in a model described in Stang *et.al.* (2001) to form the snow load. In the calibration of k_{mod} a snow pack is defined as a period where the ground is covered by snow. Load is assumed to have a triangular shape within the snow packs i.e. a linear increase and decrease of snow load is considered. This facilitates the generation of load sequences as only duration and maximum load of the snow packs are needed.

The Danish code (DS410:1998) makes use of shape factors to take into consideration additional load on roofs due to snowdrifts. The shape factors are based on large variations i.e. snow is unlikely to drift in the same manner at every snowfall. Shape factors are not considered in the present calibration of k_{mod}. This implies that the calibration of k_{mod} is based on a worst case scenario where snowdrifts are of the same size and place in every snow pack for 50 years. Hence, the results from the calibrations are most likely very conservative and the resulting k_{mod} too low.

A Gumbel distribution has been used to fit the maximum snow load per snow pack with a mean value equal to 328 N/m^2 and coefficient of variation of 0.64. A total number of 189 snow packs have been registered on the 5 locations over a period of 32 years. An exponential distribution has been used to fit the results on duration of the snow packs having mean value equal to 25.9 days and coefficient of variation equal to 1.0. Hence, the average occurrence of snow is 1.18 per year. Considering an average duration of 25.9 days per snow pack this corresponds to an average of 30.5 days of snow per year.

It may seem reasonable that snow load and snow duration are correlated, i.e. large snow load goes with long periods of snow. Nevertheless, the coefficient of correlation between duration of snow and magnitude of snow load is 0.67. Hence, neither full correlation nor un-correlated snow packs can be used to describe the snow load and duration in Denmark. The investigation is based on both full correlation and fully un-correlated snow packs, representing the theoretical bounds.

Wind load in Denmark

Wind load on buildings is influenced by the wind velocity combined with factors such as turbulence and roughness and topology of the surroundings. The wind velocity is determined from measurements using sensors typically placed at a height of 10 m above flat farmland or sea in order to reduce disturbance by nearby trees, buildings, hills etc. (CIB 1996). Measurements are stored as 10 min-mean wind velocities in a fixed window and taken over all wind directions. The turbulent flow of the wind forces on buildings is dealt with by factors multiplied by the reference 10 min-mean wind load. These factors are used to account for fluctuations of wind having peaks much higher than the averaged 10-minutes wind velocity determined from measurements. In the Danish code, the amplitude of fluctuations is referring to 1 second-values.

The wind data used in the present investigation are obtained from Sprogø, a small island situated in Great Belt. Measurements have been made in a 16-year period. It is recognised that the measuring period is very short compared to the reference period of 50 years used in this investigation. However, the results from Sprogø have been compared to results from other locations in Denmark. These, less accurate, measurements have been made for decades and show similar results. Thus, it is assumed that the data from the 16-year period on Sprogø represent the wind velocity in Denmark reasonably well. The same set of data was used to calibrate the characteristic wind velocity and the partial safety factors for wind in the new Danish code (DS410:1998).

Only 10 min-mean wind velocities of more than 13 m/s (storms) are included in the data used for the analysis. A total number of 840.000 10 minute-periods within 16 years is used to determine the distribution of wind load. The parameters for the Gumbel distribution used to fit the results are mean wind load (10 min-mean): -31.25 and coefficient of variation: 3%. The fit is made with emphasis on a good description of large loads. Calibration of k_{mod} is based on these data. However, the design load according to the code is based on values for 1 second and, therefore, the calculated value for k_{mod} has to be modified. Stang *et.al.* (2001) have determined a multiplication factor k* equal to 1.16, to compensate for the mismatch between design loads corresponding to 1 sec values and the calibration of k_{mod} corresponding to 10 min. values.

The shape factor is determined from tests on models in a wind tunnel and is assumed to be independent of the magnitude of wind velocity pressure. The variation of shape factors and the wind direction are disregarded in the present investigations. Hence, the analysis is made on a conservative basis.

Results

The results presented in table 5 are determined using the method described above for calibrating k_{mod}. Each k_{mod} value presented is based on 200 distinctive load sequences where one sequence represents 50 years of load. In figure 3 the maximum load for every sequence is shown together with an equivalent load representing the 50-year load action considering load duration.

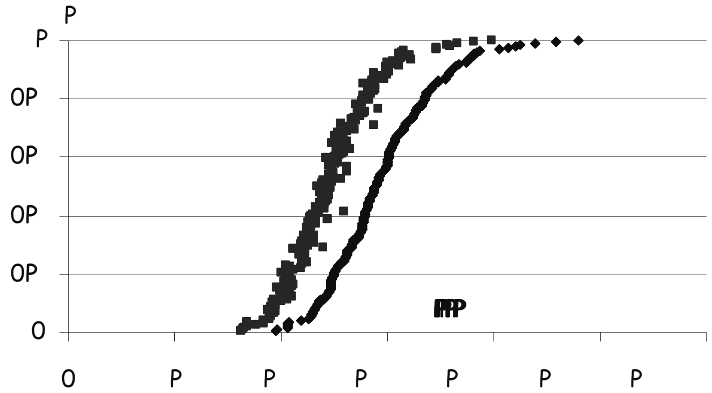

Figure 3. Maximum annual snow load under 50 years (left curve) simulated for two hundred 50 years sequences and equivalent load (right curve) for the same sequences comprising the effects of load duration for each sequence. The equivalent load was derived using Barrett and Foschi's damage accumulation model (Barrett and Foschi 1978) fitted to Nordic data (Hoffmeyer 2001).

Four levels of safety for annual return period are investigated (β = 3.5, 4.0, 4.5, and 5.0). However, very little difference is found between the four safety levels, and only results for β=5.0 are presented. The calibration is made on a single element (a solid wood beam) with two end supports and uniformly distributed load. No system effect such as load sharing between beams is considered. In addition to the loads described above the analysis of load carrying capacity make use of the coefficients of variations given in Table 4.

Table 4. Coefficients of variation used in the analysis

Short term strength	0.20
Model uncertainty	0.10
Geometry	0.02

For Nordic grown timber, a coefficient of variation of 20% has been found for short term bending strength (Ranta-Maunus *et.al.* 2001 and Sørensen & Hoffmeyer 2001).

Two load cases are investigated for every load type, one case with only variable load and one case with combined permanent load (dead weight) and variable load. For the latter case the permanent load contribution is set to 25% of the mean annual total load.

Table 5. Results from calibrations of k_{mod} for an annual safety index β equal to 5.0. Results are shown for 100% variable loads and for 25% permanent load plus 75% variable load. The calibrations are performed for two different damage models fitted to two sets of data (see Table 1).

	Var/perm	Live load		Snow load		Wind load	
β = 5.0	[%]	Residents	Office	No corr.	Full corr.	k_{mod}	k^*_{mod}
Nordic data, Barret and Fochi	100/0	0.83	0.81	0.84	0.80	0.94	1.09
	75/25	0.83	0.78	0.84	0.80	0.94	1.09
Nordic data, Gerhards	100/0	0.83	0.80	0.84	0.78	0.96	1.11
	75/25	0.84	0.80	0.83	0.78	0.95	1.10
Madison, Gerhards	100/0	0.84	0.82	0.82	0.83	0.96	1.11
	75/25	0.84	0.82			0.96	1.11

Discussion and concluding remarks

The modification factor k_{mod} for duration of load on timber structures has been determined by a probabilistic analysis taking into consideration the load and duration of load for live load, snow load, and wind load. The analysis has been performed using two different damage models calibrated against two sets of experimental data (clear wood and Nordic grown structural timber). Every analysis has been made for two load cases: one based on full loading by the load type investigated, and one having 75% of this load and 25% permanent load. The analyses have been carried out for 4 different annual safety indexes: 3.5, 4.0, 4.5, and 5.0, but very little difference was found. Therefore, only results from the safety index of 5.0 are shown in this report. The Danish target safety index is 4.8.

Snow load

The analysis on snow load is carried out for fully correlated load and duration and for uncorrelated load and duration. In reality, the measurements show an in-between correlation of 0.67. The results of the analysis show smaller k_{mod} for full correlation. The results for k_{mod} on snow load are ranging from 0.78 to 0.84. An average result for k_{mod} is 0.83 for no correlation and 0.80 full correlation between load and duration.

The data used for snow load and duration are based on meteorological data on the ground. Snow on a building is likely to melt away faster than snow on the ground. This is due to higher temperature on the roof due to heat being transported through walls and roof. Hence, large snow loads that are built up over several days have shorter duration than the model prediction. Shape factors were assumed to be the same for every snow pack. Hence, the calibration shown here may significantly overestimate the loads within 50 years. Based on this fact it has been decided to make a new calibration of k_{mod} for snow. The existing code-value for snow is 0.90 (DS413:1998).

Wind load

The wind load employed originates from wind loads caused by measured mean wind velocities exceeding 13 m/s over a period of 10 minutes. No structure shape factors are taken into consideration. The modification factors, k^*_{mod} , on wind load are compensated for the difference in design load based 1 second-values and damage accumulation based on 10 minute-values. The compensated value, k^*_{mod} , for wind is 1.10. This result is equal to k_{mod} for wind in the Danish code (DS413:1998).

Live load

The calibration of k_{mod} for live load has been carried out for residential buildings and for office buildings. The results are ranging from 0.83 to 0.84 (residents) and from 0.78 to 0.82 (office). For residential buildings, the mean value is 0.83 and for offices the mean value is 0.81.
In the Danish code, no distinction is made between residential buildings and office buildings. The existing value for live load in general is 0.80, which is slightly on the safe side according to the results presented here.

References

Barrett, J.D. and Foschi, R.O. (1978). Duration of load and probability of failure in wood. Part 1: modelling creep rupture. *Canadian J. of civil Engineering* 5(4). pp 505-514.

Barrett, J.D. and Foschi, R.O. (1978). Duration of load and probability of failure in wood. Part 2: constant, ramp, and cyclic loadings. *Canadian J. of civil Engineering* 5(4). pp 515-532.

CIB Commision W81. (1996). *Actions on Structures – Windloads.* CIB Publication 193.

CIB Commision W81. (1989). *Actions on Structures – Liveloads in Buildings.* CIB Publication 116.

Dansk Standard. (1998). *Norm for sikkerhedsbestemmelser for konstruktioner DS409(2.1)* (*Code of Practice for Saftey of Structures, in Danish)*. Copenhagen.

Dansk Standard. (1998). *Norm for last på konstruktioner DS410(4.1)* (*Code of Practice for Loads for the Design of Structures, in Danish).* Copenhagen.

Dansk Standard. (1998). *Norm for Trækonstruktioner DS413(4.1)* (*Code of Practice for the Structural use of timber, in Danish)*. Copenhagen.

Forsythe, G.E., Malcolm, M.A., and Moler, C.B. (1976). *Computer Methods for Mathematical Computations.* Prentice-Hall.

Gerhards, C.C. (1979). Time-related effects on wood strength: a linear cumulative damage theory. *Wood Science* 11(3). pp 139-144

Hoffmeyer, P. (2001). Strength under Long-Term Loading. Chapter of textbook in Swedish Ph.D.-course on Timber Engineering. pp. 25. To be published with Wiley.

Pearson, R.G. (1972). The effects of duration of load on the bending strength of wood. *Holz Forschung* 26(4). pp 153-158.

Ranta-Maunus, A., Fonselius, M., Kurkela, J., Toratti, T. (2001). Reliability analysis of Timber Structures. Research Notes, VTT Building and Transport, Espoo, Finland.

Svensson, S., Thelandersson, S., and Larsen, H.J. (1999). Reliability of timer structures under long term loads. *Materials and Structures* 32, pp 755-760.

Stang, B.D., Svensson, S., Sørensen, J.D. (2002). Safety of Timber Structures - Effect of load duration on Timber structures in Denmark, Research report to be published with Danish Building and Urban Research, Hørsholm, Denmark.

Sørensen, J.D., Hoffmeyer, P. (2002). Statistical Analysis of Data for Timber Strengths, Research report to be published with Danish Building and Urban Research, Hørsholm, Denmark.

Thelandersson, T., Larsen, H.J., Östlund, L., Isaksson, T., Svensson, S. (1999). *Säkerhetsnivåer för trä och träprodukter i konstruktioner*, Report TVBK-3039, Avdelningen för Konstruktionsteknik, Lund University. Sweden: Lund University

Wood Handbook (1999). *Wood as an engineering material*. General Techn. Rept FPL-GTR-113. USDA Forest Service. Forest Products Laboratory, Madison, Wisconsin

Wood, L.W. (1951). *Relation of strength of wood to duration of load.* Rept. 1916. USDA Forest Service. Forest Products Laboratory, Madison, Wisconsin

Nordic Wood: Reliability of timber structures

System effect

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Introduction

Structural timber, as opposed to other structural materials such as steel, shows a significant variability in material properties, both within and between members. This variability results in phenomena such as length and load configuration effects and system effects. In the design codes timber elements are regarded as homogeneous, i.e. the properties are constant within the member. In Eurocode and most national codes the properties are based on the bending strength of the weakest section within the member. This simplified strength model in general underestimates the load-carrying capacity of timber, i.e. the timber member has a higher reliability level in reality.

The positive effect of the variability within and between elements on the load carrying capacity of a structural system can be referred to as a system effect. Normally the effect on a single structural member is referred to as volume and load configuration effects while for a system of single members the effect is referred to as load sharing or system effect. The Nordic countries deal with these effects to different extents. Some (Sweden, Finland) only account for a height effect for glued laminated timber while others (Norway) are similar to Eurocode 5. The latter includes a member size effect and load-sharing factor.

The scope of this section is to suggest some definitions regarding system effects. Further, a few investigations on system effect reported in the literature are reviewed. Finally, an effort is made to conclude and quantify system effects for structural timber and propose possible implementation in codes.

Definitions

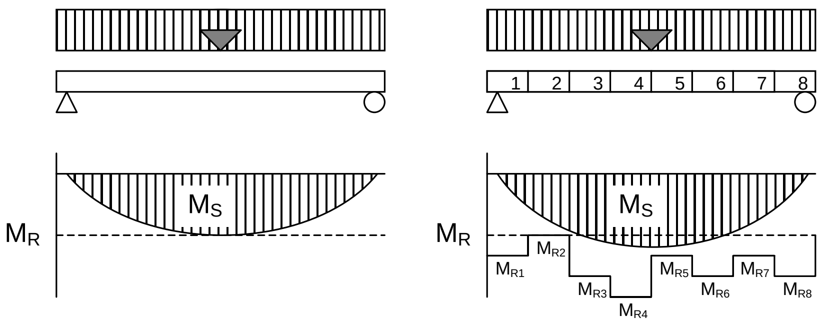

Figure 1. Comparison between two definitions of an element. Left: The beam is one element. Right: The beam is a system built up by 8 elements. M_S is moment due to action and M_R is the moment capacity.

Before starting the discussing on system effects, there is a need for making some definitions. These are by no means commonly accepted, but more a help when writing about system effects on this specific occasion. The following definitions are used:

- Element: A volume of material with constant properties.
- Member: Built up by elements.
- System: Built up by members.

There are several possible levels on where to define the element. In most codes the beam is used as an element and a factor is introduced to account for possible volume effects within the element. For research purposes and understanding of the system effects this is quite rudimentary. Understanding the effect of variability within and between elements necessitates dividing the beam into smaller parts. Since the scope is structural timber or timber products, there is generally no need for using the small defect free volume as the basic element. The element can be defined as a volume for which the properties can be regarded as constant without any serious consequences for the evaluation on the member level. In Figure 1 two different definitions of an element are shown. The left shows what is commonly regarded as an element in the code and the right the definition used in this presentation. In this example the member (beam) is built up by 8 elements. The volume of the elements does not have to be uniform. For a glue-laminated member the definition of the elements of a member can differ significantly from the example shown in Figure 1. The element can for example be identified as the single lamella or part of a single lamella.

Using the definitions above there is one system effect when assembling elements to a member and another when assembling a system of members.

Figure 2 shows an example of a parallel assembling of members to a system. The beam members are connected by a load-distributing system (sheathing). The connection between beam and sheathing can be more or less rigid. Another example of a system is a trussed rafter, which is more of a series system, i.e. the most utilised member of the system governs the load-carrying capacity of the whole structure. A roof structure built up by several trussed rafters can be regarded as a parallel system similar to the one shown in Figure 2 where each beam is now a trussed rafter. The load-distributing system can be sheathing or secondary beams.

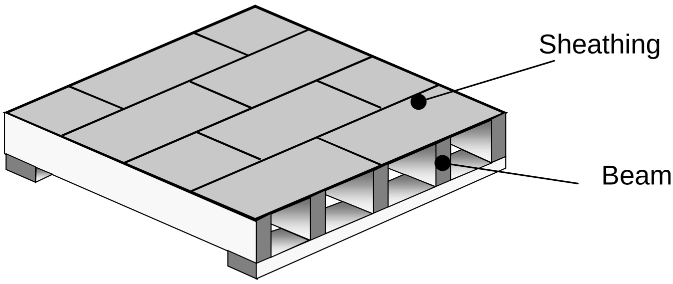

Figure 2. Parallel system (Hansson and Isaksson 2001).

Volume and load configuration effects on members

The strength of a timber beam depends on the volume (test span, cross section) and type of loading. The longer and higher the beam and the more uniform the stress, the lower the strength. The weakest link theory (Weibull) is the most commonly used method to evaluate and quantify this phenomenon. Table 1 presents a summary of how the Eurocode 5 and the national codes of the Nordic countries account for the size effect in the bending mode. There are similar expressions available for the tension mode. The values are not directly comparable since γ_M and characteristic strength values differ between the codes.

The effect of load configuration is seldom dealt with in the codes but engineering rules of thumbs are used, i.e. moment distribution in the upper cord of a trussed rafter can be adjusted to account for the lower probability of having a weak section at the moment peaks (Riberholt 1990) and also account for the width of the support of the chord.

Isaksson (1999) and Källsner et al (1994, 1997) introduced a statistical model that describes the variability within and between members in the bending mode. The variability was split in one part due to within member variability and one part due to between member variability. The variability within a member is due to the variability between the elements assembling to a member. Using this model, Isaksson (1999) studied the length and load configuration effects. Compared to using Weibull theory, the effects were lower when using reliability methods to quantify these effects.

Table 1. Size effects in the bending mode for Eurocode 5 and the national codes of the Nordic countries. *h* is the height of the cross section. The minimum value of k_h is 1.0 (with some exceptions [3)]). Note that the design strength will also depend on γ_M and the characteristic strength of the reference member size.

	Eurocode 5	BKR 99 (Sweden)	NS 3470 (Norway)	DS413 (Denmark)	RIS 120-2001 (Finland)
Solid timber	$k_h = \min\left\{\begin{matrix}(150/h)^{0.2} \\ 1.3\end{matrix}\right.$ for $h \le 150$ mm	$k_h = 1$	$k_h = \min\left\{\begin{matrix}(150/h)^{0.2} \\ 1.3\end{matrix}\right.$ for $h \le 150$ mm	$k_h = 1$	$k_h = 1$
Glued lam. timber	$k_h = \min\left\{\begin{matrix}(600/h)^{0.1} \\ 1.1\end{matrix}\right.$ for $h \le 600$ mm	$k_h = \min\left\{\begin{matrix}(600/h)^{0.2} \\ 1.15\end{matrix}\right.$ for $h \le 600$ mm	$k_h = 1$	$k_h = 1$	$k_h = (300/h)^{0.11}$ $h \ge 300$ mm 3)
Lam. veneer lumber	$k_h = \min\left\{\begin{matrix}(300/h)^{s} \\ 1.2\end{matrix}\right.$ for $h \le 300$ mm 2)	$k_h = (300/h)^{0.11}$ $300 \le h \le 900$ 1) 3)	$k_h = (300/h)^{0.14}$ 4) for $h \le 300$	$k_h = 1$	$k_h = (300/h)^{0.11}$ $300 \le h \le 900$ mm for $h > 900$ $k_h = 0.88$ 3)

1) Not given in the code but used in design guidelines
2) The size effect exponent *s* for LVL shall be declared by the producers
3) Reduction factor, i.e $k_h \le 1.0$.
4) According to NBI Teknisk Godkjenning. Under revision.

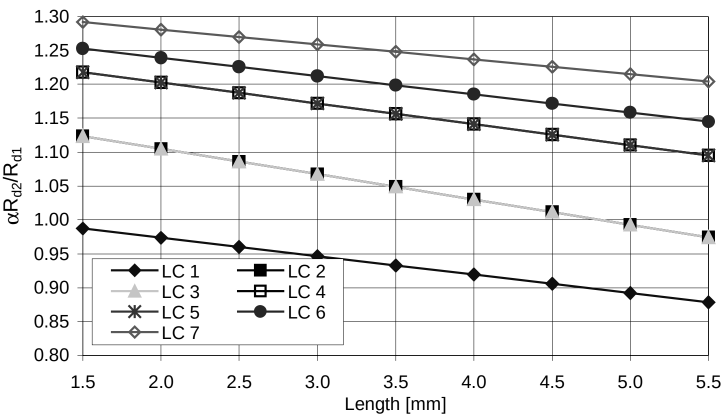

Figure 3. The ratio between αR_{d2} and R_{d1} as a function of length and load configuration. LC1 corresponds to a constant bending moment, LC2 to four point bending, LC3 to uniform load, LC4 to single point load, LC5 to single point load and fixed supports, LC6 to four point bending and fixed supports, and LC7 touniform load and fixed supports (Isaksson 1999).

Using the model a calibration against a code format can be performed. In Thelandersson et al (1999) and Isaksson (1999) a study is presented where a function, see eq. (1), is introduced to account for length and type of loading in the bending mode. The function accounts for the length *L* and type of loading ξ. ξ reflects the fullness of the moment distribution, i.e. a constant bending moment corresponds to ξ=1 and a three point bending ξ=0.5.

$$\alpha = 1.4 - 0.45\xi - 0.03L \qquad \text{eq. (1)}$$

The ratio between the designing resistance R_d with and without the α-function, $\alpha R_{d2}/R_{d1}$, is presented in Figure 3. As can be seen, for most combinations of length and load the α- function is above 1, i.e. the strength of the member is not fully utilised in today's codes. In average an increase in strength of 10 % is possible.

Load sharing effects on systems

Floors and flat roof elements

In 1989 Foschi et al presented a study on system design of floors and flat roofs. The study presented a modification factor to be used to increase the single member strengths and thus account for the system and load sharing effect. The behaviour of the beams was linear and failure of the system corresponds to first failure of any beam in the system (method A). Figure 4 shows the possible increase in beam span when considering the system behaviour. The system effect showed a mean value of 1.34. The system effect includes the effect of composite behaviour between beam and sheathing.

In Hansson and Isaksson (2001) a study similar to Foschi et al (1989) was performed using the statistical model of within and between member variability of timber members (Isaksson 1999). A tri-linear model of the beam member behaviour was used and failure of the system corresponds to failure of two neighbouring beams or any three beams in the system. The results of the simulations showed a significantly lower variability (coefficient of variation, *COV*) in the failure load of a system compared to a single member, 9.3 % and 21.5 % respectively. The analysis of system effects is under development, but the results indicate a system factor in the order of 1.15.

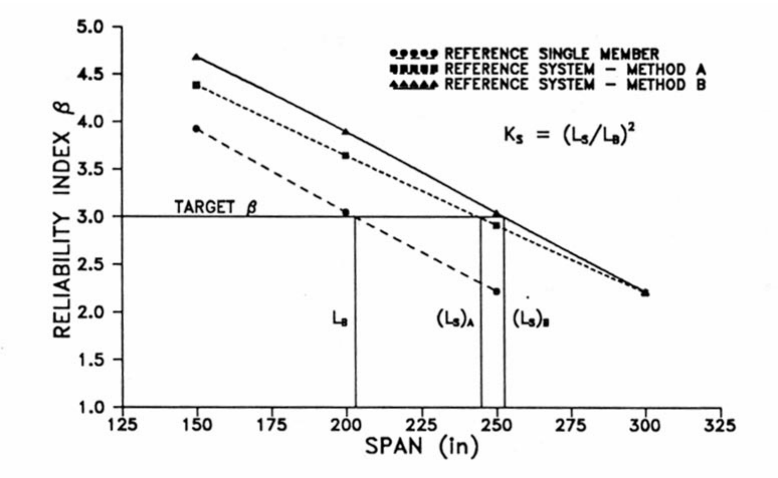

Figure 4. The system effect according to Foschi et al (1989).

Table 2 shows a summary of how the system effect is accounted for in the Eurocode and the national codes of the Nordic countries. There are a few conditions that must be fulfilled before the system effect can be used. In general there must be a continuous load distributing system that connects the beams laterally.

Table 2. System effect according to Eurocode 5 and the national codes of the Nordic countries.

	Eurocode 5	BKR 99 (Sweden)	NS 3470 (Norway)	DS413 (Denmark)	RIS 120-2001 (Finland)
System effect	$k_{sys} = 1.1*$	$k_{sys} = 1$	$k_{sys} = 1.1$	$k_{sys} = 1$	$k_{sys} = 1$

*strength verification assuming short term load and γ_M=1.0

Roof trusses

The distribution of forces and moments in a truss often results in quite narrow peaks. This implies that the benefit of the within member variability would be applicable, i.e. the lower probability of the weakest section to coincide with a moment peak. In an investigation by Hansson (2001) this effect was found to be around 15 %, i.e. the failure load is approximately 15 % higher for a truss when you account for the within member variability.

Concluding remarks

Most studies on strength of timber and timber products show that indeed the strength is size dependent. This is to various extents included in codes. There is however a need to make the correction methods more uniform. At least the correction factor should be a voluntary way to increase strength and not a reduction factor that must be included.

The effect of load configuration on the strength has not been experimentally verified to the same extent as size effect. However, the nature of timber makes the size and load configuration effect strongly related. If size effects are used it is logical to also include load configuration effects.

The system effect for parallel systems is confirmed by several investigations and an increase of about 15 % seems to be the general conclusion.

In general, there is no need to make the design of timber structures more complicated by introducing more factors. It is however necessary to understand the behavior of timber in different situations when deciding how the design of timber should be represented in future codes. For products made of timber or timber products where larger volumes are manufactured it could be profitable to use a more detailed design, i.e. including size, load and system effects.

References

BKR 99, (1999). Boverkets konstruktionsregler (Swedish code). Boverket BFS1993:58.

DS413 (4.1) (1998). Dansk Standard. Norm for Trækonstruktioner (Danish Code).

Eurcode 5, (2001). *Design of timber structures. Part 1.1*. Final draft prEN 1995-1-1.

Foschi, R.O., Folz, B.R. and Yao, F.J., (1989). *Reliability based design of wood structures*. Structural research series, report No 34. University of British Columbia, Vancouver, Canada.

Hansson, M., Isaksson, T., (2001). *System effects in sheathed parallel timber beam structures*. CIB-W18, Meeting 34, paper 34-8-1, Venice, Italy.

Isaksson, T., (1999). *Modelling the variability of bending strength in structural timber – length and load configuration effects*. Report TVBK-1015, Div. of Structural Engineering, Lund University, Sweden.

NS 3470 Prosjektering av trekonstruksjoner – Beregnings- og konstruksjonsregler. Del 1: Allmenne regler (Norwegian code).

RIL 120-2001 Puurakenteiden suunnitteluohjeet (Design of wood structures). Finlands Byggnadsingenjörers Förbund, Helsinki 2001. (Finnish code)

Riberholt, H., (1990). *Proposal for Eurocode 5 text on timber trussed rafters*. CIB-W18, Meeting 23, paper 23-14-2, Lisbon, Portugal.

Thelandersson, S., Larsen, H.J., Östlund, L., Isaksson, T., Svensson, S., (1999). *Säkerhetsnivåer för trä och träprodukter i konstruktioner (Safety levels of structural timber and timberproducts).* Report TVBK-3039, Div of Structural Engineering, Lund University, Sweden.