CIB-W18 Timber Structures

– A review of meetings 1–43

Part 2: Material Properties

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CONTENT

2.1 COMPRESSION PERPENDICULAR TO GRAIN

9-12-1 F Wassipaul, R Lackner
Experiments to provide for elevated forces at the supports of wooden beams with particular regard to shearing stresses and long-term loadings

Introduction

After the preliminary examinations had been finished, which had the purpose to find a method to reinforce the supports of wooden beams in order to improve their resistance to transversal compression loads, some questions were still unanswered.

These examinations didn't show undoubtedly, which arrangements and numbers of the reinforcing elements were the best to reduce indentation. For this reason, two test series were carried through in order to complete the preliminary examinations.

A further question was particularly the behaviour of the reinforcing elements during the static long-time test. According to DIN 1052 the strength of wooden structural elements decreases essentially with increasing duration of loading. Various explanations are given for this fact and it was attempted to determine the influence of long-time loading on the strength of reinforced wooden structural elements in comparison with unmodified ones.

Further on, it is known that the strength values obtained by testing depend substantially on the size of the samples. Since with model tests conditions may be caused by optimum choice of the wood and by optimum machining, as they as a rule cannot be achieved in practice, and further on, the so-called sample-volume (e.g. ratio of support distance to sample height or cross-section of sample) is influenced, it is necessary in most cases to carry through experiments on roughly shaped samples of commercial size in order to prove the results obtained by the model tests. For this purpose, 5 straight laminated beams made of wood, 16 cm wide, 80 cm high and 350 cm long were for their behaviour in a static long-time test with variations of their support areas. Finally it was to be elucidated whether the reinforcing elements were able to contribute to the absorption of the often high shearing stresses, which may occur within a beam in the zone of the supports.

This effect would have to be anticipated and had been mentioned in literature. In order to prove the effect, a method described in literature was used. The cross-section, which is deciding for the ability to absorb shearing stresses, was reduced within the zone of the highest shearing stresses; thus an improvement of the ability to absorb shearing stresses, caused by the reinforcing elements, would have had the possibility to show itself.

Summary

The low resistance of spruce wood, which is frequently used for wood constructions, against transversal compression loads, $\sigma_{c,90}$, often causes difficulties with rating of structural elements particularly if high main loads or supplementary loads (f. e. snow and wind loads) are to be considered.

In this report which completes previous reports on the research project mentioned at the beginning, a method to reinforce the supports of laminated beams, developed at the Wood Research Institute, Vienna is described.

For reinforcing, elements being able to absorb the forces initially, are used, chiefly nails, dowels made of steel and wood, which are arranged in the wood of the beam in the direction of the supporting forces and which lead the forces impacting at the support into the core of the beam. The reinforcing elements were tested single and in the compound for their effectiveness, particularly the transmission of forces, the size and arrangement of the elements in the static short-time test and with long time load were investigated.

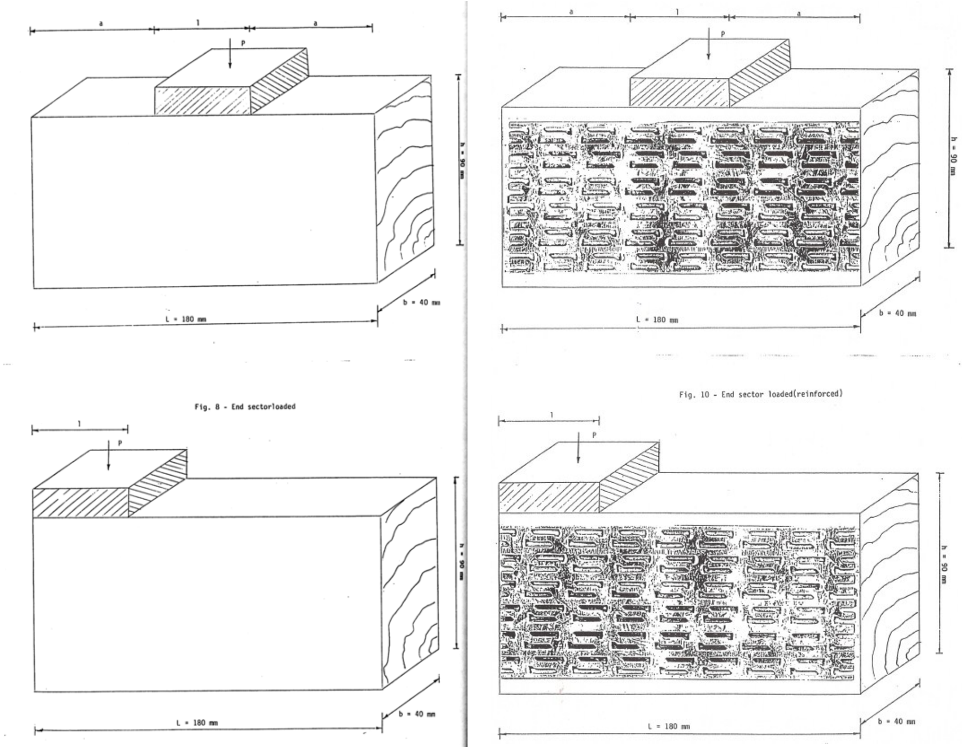
Inserting nails with special shaped shafts (special nails for wooden constructions) as well as inserting dowels made of those made of steel with coil profile results in a reduction of the indentations perpendicular to the grain at the supports of laminated beams made of wood.

A statement on beams made of solid wood cannot be made due to too low a number of samples.

According to ÖNORM B 4100, 2.Part) with good timber for constructional use compression stresses perpendicular to the grain up to 25 kp/cm^2 are permissible, provided shallow indentations are unobjectionable". If the indentations, caused by these stresses at the not reinforced support, are considered as a criterion, inserting the above-mentioned reinforcing elements results in an improvement, i.e. an increase or equal indentation.

23-6-1 U Korin
Timber in compression perpendicular to grain

Summary
The compressive strength of timber perpendicular to grain was investigated. The study reports tests conducted on whitewood specimens 40 mm x 90 mm x 180 mm in size in central loading or end loading for varying lengths of loading sectors *l/L*. Upgrading experiment of bearing capacity of the timber by Nail-plates reinforcement was found to be unsuccessful.



Discussion and Conclusions

1 It is necessary to agree on a clear strength-bearing capacity criterion for timber in compression perpendicular to gain.

2 Eurocode 5 brings a parameter $K_{c,90}$:
$\sigma_{c,90,d} = K_{c,90} f_{c,90,d}$
$K_{c,90}$ depends upon the length of the overhangs of the loaded member. The following table presents a comparison between the values of $K_{c,90}$ calculated for the loading conditions in this investigation and the experimental results of this investigation. The differences between the compared values are very large and should be further studied.

3 The investigation covered so far only one kind of timber and one cross section (*d/b*=2). There is a room for a much wider investigation on the subject.

Table 2 Comparison between $K_{c,90}$ and experimental results.

l/*L*	Central Sector loaded		End sector loaded	
	K_c	Experimental results	K_c	Experimental results
1	1	1	1	1
0.875	1	1.063	1	1.063
0.75	1.006	1.188	1	1.156
0.625	1.025	1.375	1	1.281
0.50	1.061	1.625	1	1.438
0.375	1.124	1.969	1	1.625
0.25	1.237	2.344	1	1.875
0.125	1.478	2.781	1	2.156

27-6-1 C Lum, E Karacabeyli
Development of the "critical bearing" design clause in CSA-086.1

Abstract
There are currently inconsistencies in the compression perpendicular-to-grain (C-perp) design procedures in the Canadian code for Engineering Design In Wood, CSA-086.1-M89. For example, a single specified design value is given for a species group irrespective of the loading condition, or the lumber grade. On the other hand, different values are assigned to Douglas-fir lumber and Douglas-fir glulam (glued-laminated timber). These inconsistencies not only make the design code confusing to designers, but they also hinder the introduction of appropriate C-perp strength values for products such as machine stress rated lumber. A research program focused on resolving these inconsistencies in the C-perp design procedures

for solid sawn lumber and glulam has recently been completed. The work consisted of short term and 2-month constant compression perpendicular-to-grain load tests on two commercial species groups. Finite element analyses of typical C-perp stress conditions were also performed to study the stress distributions for various C-perp applications. This work resulted in proposed changes to the compression perpendicular-to-grain design clause for the next revision of CSA-086.1. One of the significant changes proposed is the addition of a separate check for compression perpendicular-to-grain stresses produced by loads applied near beam supports. In this paper, the work done to develop this particular clause is presented.

Code implementation
The findings of this study were considered in the code change proposal for the 1994 edition of C SA-086.1. The proposed change consists of three major revisions: an increase in C-perp capacity for loading on the wide face; a check on "critical" C-perp loading as opposed to "contact" type loading; and revisions to the characteristic strength values for C-perp design. As part of the review of characteristic C-perp design values, a 10% reduction in the C-perp design value for the Douglas-fir/larch species group was recommended. This was judged to be sufficient to account for the discrepancy in the constant load performance between the Douglas-fir/larch and spruce-pine species group tested in this study.

The code proposal submitted to the CSA Technical Committee on Engineering Design in Wood is included. Highlights of the code change proposal that came about as a result of this study are:

– C-perp design value increase factor of 15% for C-perp loading on the wide face
– retention of the current design code equation for "contact" type C-perp loading
– addition of the "critical" C-perp check for those cases where load is applied near the support
– use of a 2/3 reduction factor for "critical" loading
– reintroduction of a duration of load factor for C-perp to account for permanent type loading

Some example applications of the proposed design procedure for C-perp are included. These examples cover some of the common C-perp applications found in engineering wood construction. Emphasis has been placed on glued-laminated timber and metal plate connected wood truss applications.

The proposed code change on C-perp design has been submitted and has been voted on by members of the Canadian technical committee on engineering design in wood. However, ballot results and comments have yet to be released. If accepted, this proposal would result in a more rational design procedure for C-perp design in CSA-086. I -M94. It will also allow the introduction of higher C-perp design values for products such as MSR lumber. Furthermore, the refinement to the design procedures has been done incrementally and without making the procedure more complicated than the current procedures.

31-6-4 L Damkilde, P Hoffmeyer, T N Pedersen
Compression strength perpendicular to grain of structural timber and glulam

Abstract
The characteristic strength values for compression perpendicular to grain as they appear in EN 338 (structural timber) and EN 1194 (glulam) are currently up for discussion. The present paper provides experimental results based on EN 1193 that may assist in the correct assignment of such strength values. The dominant failure mode of glulam specimens is shown to be fundamentally different from that of structural timber specimens. Glulam specimens often show tension perpendicular to grain failure before the compression strength value is reached. Such failure mode is not seen for structural timber. Nonetheless test results show that the levels of characteristic compression strength perpendicular to grain are of the same order for structural timber and glulam. The values are slightly lower than those appearing in EN 1194 and less than half of those appearing in EN 338. The paper presents a numerical analysis to prove the significant role of tension perpendicular to grain stresses in the failure mode of the glulam specimens.

Conclusions
– The compression strength of structural timber and glulam of Nordic origin and tested according to EN 1193 lies within the limits from 2.0 MPa to 4.0 MPa. The mean value for both structural timber and glulam

is $f_{t,90,mean}$ = 2.9 MPa. The 5-percentile values for both structural timber and glulam is $f_{t,90,k}$= 2.3 - 2.4 MPa.
- As a rough estimate of compression strength perpendicular to grain the following regression equation may be used for both average and 5-percentile values for both structural timber and glulam:
- STRENGTH (MPa) = 0,006 DENSITY (kg/m^3)
- For the assessment of strength, the laborious measurement of strain prescribed by EN 1193 may be replaced with a measurement of the relative movements of the crossheads of the test machine. However, for the assessment of modulus of elasticity it is necessary to adhere to the prescriptions laid down by the standard.
- The average modulus of elasticity across the grain is of the order $E_{90,mean}$ = 300 MPa for both structural timber and glulam.
- Finite element modelling proves that the inhomogeneous distribution of modulus of elasticity $E_{c,90}$ across a given specimen results in a complicated stress distribution.
- The radial stresses (direction of applied force) are in compression and the level in the middle is higher than at the sides; the stress level increases with depth towards the lower part of the specimen. This wedge shape form of carrying stresses is a result of the annual ring orientation of the laminations. The maximum compressive stress is about four times that of the reference stress level.
- The tangential stress distribution (transverse direction) is somewhat more complicated and results in both compression and tension. High levels of tension stresses are found in many small areas, typically in the bonds between boards where the E-moduli change discontinuously. The stress level is of the order 40 % of the compression stress, which explains the fracture pattern.

31-6-5 R H Leicester, H Fordham, H Breitinger
Bearing strength of timber beams

Abstract
In testing timber to determine design values for bearing pressures it is necessary to choose values for both strength and deformation limit states. This paper discusses various test configurations that may be used for in-grade measurements, and also discusses the question of applying a deformation knit in assessing strength properties.

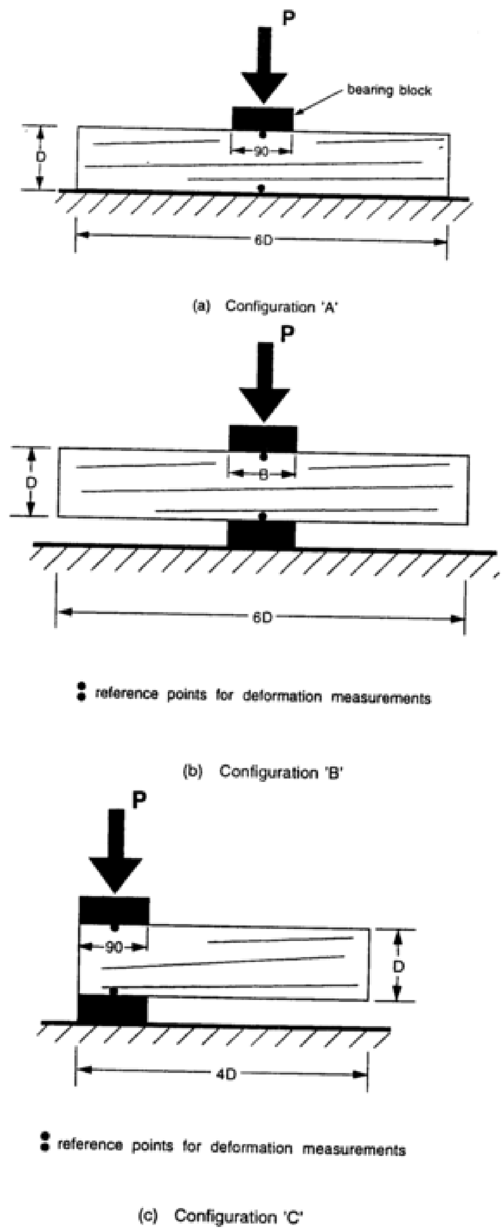


Figure 3. Configuration for testing structural size timber.

The discussion is supplemented by test data from a project undertaken to develop an in-grade test to evaluate design bearing strengths for timber beams.

Conclusions

In the selection of a configuration for a standard in-grade test, both Configurations 'A' and 'B', shown in Figure 3, are good candidates; the preference is probably for Configuration 'C' because it models a very practical in service condition.

With respect to the data recorded, both the stiffness k and 2 mm offset stress are useful parameters for the design of serviceability limit states.

For laboratory evaluations of characteristic values for strength limit states, it is suggested that some deformation limit be included. This is because at large deformations, very rigid restraints are required to prevent- the test specimen from collapsing. For test configurations in which specimens experience local bearing pressures from only one side, such as Configuration A, the deformation limit should be 5 or 10 mm. For test configurations involving local bearing pressures from two sides, such as Configuration 'B', the limit may be doubled.

40-6-2 M Poussa, P Tukiainen, A Ranta-Maunus
Experimental study of compression and shear strength of spruce timber

Introduction

The European standards EN 338 and EN 384 provide a strength class system for structural timber and strength profiles (characteristic strength values for bending, tension, compression and shear stresses) for each strength class. Basis of the system when placing a piece of timber to a strength class is bending strength supported by modulus of elasticity and density which are also grade determining properties. All other characteristic values are determined by calculation from bending strength or density.

Characteristic shear strength values of EN 338 are based on the relation

$$f_{v,k} = 0{,}2 f_{m,k}^{0,8} \qquad \text{with max 3,8 MPa}$$

which indicates that shear strength increases with increasing bending strength until $f_{m,k}$ =40 MPa. Only few test results based on test method of EN408 are published. German experiments give characteristic shear strength value of 3,8 MPa and COV = 0,20 for spruce from C18 to C30, on average. There are also other test results which suggest that shear strength of timber is practically independent of grade or is even higher for lower grades. In this paper new results are published on shear strength of timber when tested by the use of different test methods.

Characteristic compression strength values perpendicular to grain of EN 338 are based on density of wood:

$$f_{c,90,k} = 0{,}007 \rho_k$$

and characteristic compression strength in grain direction is given as function of bending strength

$$f_{c,0,k} = 5 f_{m,k}^{0,45}$$

In this paper new tested compression strength values are published and different testing standards compared. Results are obtained in "Combigrade" project which was intended mainly for development of strength grading, but produced also other strength data.

Comparison of test results with EN 338 values (MPa)

	Visual grade	Test result EN408	EN338	Alternative test result
$f_{c,0,k}$	C18	22,5	18	
	C24	26,2	21	
	C30	29,8	23	
	all	27,9		
$f_{c,90,k}$	all	2,2	2,6 (C27)	2,9 beam end, both edges loaded 3,7 middle support, both edges loaded 5,2 ASTM D 143
$f_{v,k}$	all	3,9	2,8 (C27)	7,2 I-beam

Suggestions for standardisation

Obtained results partly support existing standards, partly suggest a revision. Main results are compiled in thr following table.

Compression strength values parallel to grain were 30% higher than those given in EN338. It is suggested that higher values are considered when EN338 is revised. However, it has to be kept in mind that low visual grades were not included in compression test material.

Compression strength perpendicular to grain when tested according to EN408 was lower than values given in EN338. Comparison with ASTM shows an expected difference: same material which gives 5.2 MPa characteristic value in ASTM test, gives 2.2 MPa in EN408 test.

The beam tests support the suggested changes in calculation of effective area in compression perpendicular to grain calculation of Eurocode 5.

Shear results support the idea of having one shear strength value, not dependent on grade. This value could be taken as 3,8 MP as given as maximum value in EN 338.

41-6-3 H J Larsen, T A C M van der Put, A J M Leijten
The design rules in Eurocode 5 for compression perpendicular to the grain - continuous supported beams

Paper revised based on discussions at the ClB-W18 meeting. The main changes concern correction of an incorrect description of the rules in Eurocode 5, amendment EC5/Al.

Introduction

An important task for CEN TC 124 has been drafting standards for determination of the properties of timber. A point of discussion has been whether the standards should aim at getting well defined basic material properties or reflect typical uses. TC 124 has opted for the former (the scientific) approach assuming that it would then be possible to calculate the behaviour in practical use situations, whilst US and Australia has chosen the other (the technological).

This for example reflected in the European test method for shear (EN 408) where a test set-up as shown in Figure 1 has been chosen, while most countries use short test beams loaded in bending and shear (and compression perpendicular to the fibres).

For compression perpendicular to the grain the two options in Figure 2 were discussed. To the left is shown a test where a block of timber is loaded in uniform compression over the full surface. To the right is shown a situation where the test specimen (e.g. a sill) is loaded over part of the length corresponding to a rail on a sleeper. The latter method that is used in and Australia gives higher strength values than the block because the fibres adjacent to the loaded area contribute in taking the load, see Figure 3. The first method was chosen in Europe, an argument being that it reflected the situation in pre-stressed timber decks, and it was assumed that the rail test results could be derived from the block results.

To be able to report and compare test results it is for a load-deformation curve that is not linear necessary to define a failure criterion. The definition according to EN 408 for block compression is shown in Figure 4. The compression strength is defined by the intersection between the test stress strain curve and a line parallel to the initial curve at a distance of 0,01*h* where *h* is the specimen height.

Since the result of exceeding $f_{c,90}$ is not collapse but only large deformations, it has been proposed, e.g. by Thelandersson and Mårtensson to regard this situation as violation of a Serviceability limit state. But even by taking advantage of the lower partial safety factors for Serviceability instead of those for Ultimate limit states, many details in traditional timber structures, e.g. timber frame houses will no longer be acceptable.

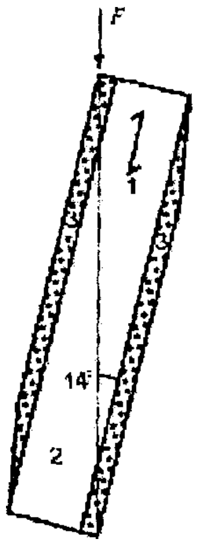


Figure 1. Shear test according to EN 408. The load is transferred through steel plates glued to the timber specimen that is loaded in shear at an angle to the grain.

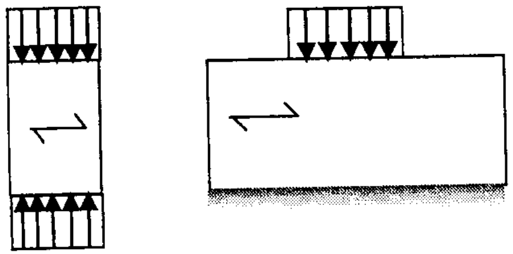
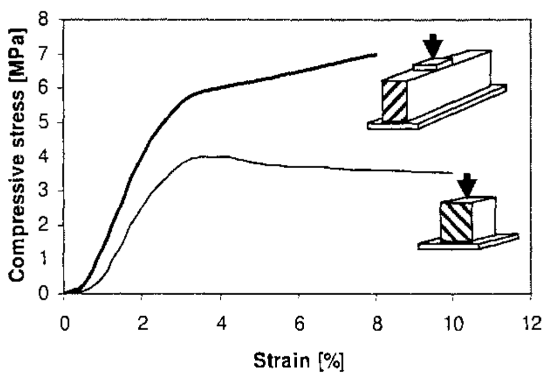


Figure 2. Pure block compression versus rail test



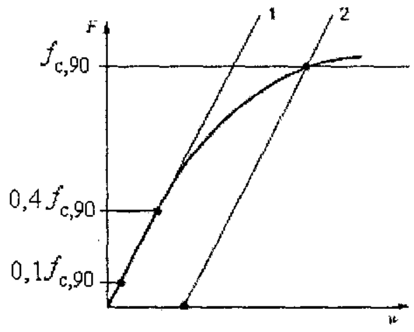


Figure 3. Load-deformation curves for uniform stress (lower curve) and for a rail situation (upper curve).

Figure 4. Load-deformation curve for compression perpendicular to the grain, definition of compression strength $f_{c,90}$ at 1 % off set.

Conclusion

On the bases of the evaluation of the model presented in the amendment of Eurocode 5/A1 and the model by Van der Put the following conclusions can be drawn:

– Van der Put's model is the more accurate and reliable in predicting the bearing strength perpendicular to the grain.
– The less reliable results of the Eurocode 5/Al model is due to the inability to take account of differences in beam height, load configuration and deformation. This causes sometimes unsafe and sometimes conservative results.
– It is suggested to replace Eurocode 5/A1 model by the model according to Van der Put

43-6-1 A Jorissen, B de Leijer, A Leijten
The Bearing Strength of Timber Beams on Discrete Supports

Abstract

The bearing strength capacity at discrete intermediate or/and end supports or areas of load introduction are the focus of this paper. Experimental research backed up by FEM and optical techniques have been used to assess and quantify the strength affecting parameters. It is demonstrated that the stress dispersion model by Van der Put is very well suited for strength predicting associated with 10% deformation.

Conclusion

Using both numerical and optical techniques, it has been shown that for non-continuous supported beams loaded perpendicular to grain the dispersion of bearing stresses are limited to a certain area. For coniferous wood (Spruce) the bearing area is restricted to 35% of the beam depth with a maximum of 140 mm.

2.2 CROSS LAMINATED TIMBER

37-6-5 P Fellmoser, H J Blass
Influence of the rolling-shear modulus on the strength and stiffness of structural bonded timber elements

Introduction
Rolling shear is defined as shear stress leading to shear strains in a plane perpendicular to the grain direction. Due to the very low rolling shear stiffness of timber significant shear deformations may occur. Fig. 1 shows a schematic representation of rolling shear stresses.

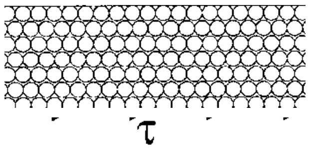


Fig. 1: Stress due to rolling shear

In prEN 1995-1 $G_{R,mean} / G_{mean}$ = 0,10 is defined for softwood. For the rolling shear strength a common characteristic value of $f_{R,k}$ = 1,0 N/mm^2 is given independent of the strength class.

Neuhaus determined the rolling shear modulus of spruce as 48 N/mm^2 for a moisture content of 9 % through torsion tests. Aicher et al. analysed the rolling shear modulus for different annual ring orientations in the cross-section using the Finite Element Method. Depending on the annual ring orientation he found values between about 50 N/mm^2 and 200 N/mm^2. Experiments by Aicher et al. resulted in a rolling shear modulus of 50 N/mm^2.

Summary
The stress distribution in and the deformation behaviour of solid wood panels with cross layers loaded perpendicular to the plane both depend on shear deformation. Due to the very low rolling shear modulus, shear deformation increases significantly depending on the thickness of the rolling shear layer.

The dynamic method of measuring the frequencies of a bending vibration was used for determining the rolling shear modulus of spruce. From the well known method for determining the modulus of elasticity parallel to the grain, a method to determine the modulus of elasticity perpendicular to the grain and the rolling shear modulus was derived. Rolling shear modulus and modulus of elasticity perpendicular to the grain both depend on annual ring orientation. Common values for the rolling shear modulus of spruce are between 40 N/mrn^2 and 80 N/mm^2.

The load bearing performance of solid wood panels with cross layers loaded perpendicular to the plane was analysed using the shear analogy method. Shear influence was determined for different build-ups of solid wood panels depending on the type of stress and span to depth ratio. For decreasing span to depth ratios, shear deformation increases significantly due to the very low rolling shear modulus. Significant shear influence was observed for span to depth ratios smaller than 30 for bending perpendicular to the plane and parallel to the grain direction of outer skins and for span to depth ratios smaller than 20 for bending perpendicular to the plane and perpendicular to the grain direction of outer skins.

39-12-4 R Jöbstl, T Moosbrugger, T Bogensperger, G Schickhofer
A contribution to the design and system effect of cross laminated timber (CLT)

Abstract
The design code of Germany DIN 1052:2004 is the first national code where the verification process for the engineered building product 'Cross Laminated Timber' (CLT) is defined. In this code the calculation of stresses for the layers of a bending stressed element has to be done for each single layer as a combination of the tensile stress in the centre line and the bending stress, as a difference between the centre line stress and the edge stress of the applied layer.

The present study has been done to evaluate an improved model of design for the homogenised product 'Cross Laminated Timber' (CLT). Starting from the base material 'boards', visual graded as S10 (according to the German standard DIN 4074-1) which should give a strength class of C24 (according to EN 338), tension tests have been made to get the characteristic properties (tensile strength and modulus). Four series of 5-layered CLT-elements of different widths have been produced and tested in bending. For comparison two small series with unidirectional build-up – quasi GLT – have been made.

The results indicate a clear affinity between CLT and GLT. On this findings a design concept for CLT based on the beam model for GLT, has been suggested. It includes the difference of homogenisation in the vertical direction (laminating effect) between CLT, and GLT by defining the factor $k_{cLT/GLT}$. Furthermore the so called 'system strength factor k_{sys}, given in EN 1995-1-1 (for 'equally spaced similar members) and EN 1995-2 (for timber deck systems working as parallel acting components) has been worked out for CLT to describe the load bearing behaviour in the horizontal direction.

Summary

In the presented project the base material 'boards' visual graded in the grading class S10 (strength class C24 respectively) has been tested to get the tensile properties.

The tensile MOE achieves the required value of about 11000 N/mm'. The tensile strength shows a low mean value ($f_{t,0,l,mean}$ = 26,1 N/mm^2) but a high COV (39,4 %), and therefore a low 5% fractile ($f_{t,0,l,05}$ = 12,5 N/rnm^2, lognormal distributed).

Based on the above mentioned starting material CLT elements have been produced in four series ('1c', `2c', '4c' and '8c', where the number in the notation stands for the number of the parallel arranged boards in the outer layers) as well as into two series of GLT elements ('1u' and '4u', where u stands for unidirectional). All specimens have been built-up as 5-layered elements and have been tested in bending in accordance to EN 408.

Three methods of calculation have been used: exact differential equation with Fourier discretisation, finite element analysis with plane stress discretisation, conventional beam model with rigid cross section and modified shear correction factor (r = 4,9 for a five layer element). All these methods have been calculated by use of three optimisation models, whereas the stiffnesses E_{90} and G_{90} have been fixed a two parametric model to optimise E_0 and $G_{90,90}$ (rolling shear), and two one parametric models with additional fixed $G_{90,90}$ (60,100) to optimise E_0. All calculation models leads to comparable results. For calculation the beam model with modified shear correction factor is proposed.

The bending test results suggest an affinity between the homogenised product CLT and GLT. Due to this situation it is comprehensible to use the same basis (beam model) for both products. The tests represent an increasing of the 5% fractile with increasing of width of the elements. Hence a so called system strength factor $k_{sys,CLT}$ = 1,1 could be defined, if four or more parallel arranged boards are given. Furthermore a factor k_h = 1,15 to consider the depth effect and the factor $k_{CLT/GLT}$ = 0,94 considering the difference in the homogenisation between GLT and CLT could be defined.

If all the three factors are multiplied a factor k_{CLT} = 1,2 ($k_{sys,\ CLT}\ k_h\ k_{cLT/GLT}$) is given for a 5-layered CLT element with a thickness of 150 mm and a width, where four or more parallel arranged boards in the outer layers are used.

New research works at the Institute of Timber Engineering and Wood Technology clearly showed that it is of importance to consider the COVt (tensile strength) for the boards and the COV_b (bending strength) of GLT when defining a new beam model.

The following model for GLT is expected (reference height h = 600 mm and reference width w = 150 mm) where a_{GLT} = 2,811 for COVt = 0,35 (normally given for visual graded material) and COVm =0,15:

$$f_{m,g,05} = a_{GLT} f_{t,0,l,05}^{0,82}$$

Based on this new beam model the following model for CLT can be expected (reference depth h = 150 mm, 5-layered element, four or more boards in the outer layers) with a_{GLT} = 1,76 + 5,0 · COVt (a_{cLT} = 3,5 for COVt = 0,35 (normally given for visual graded material) and COVm = 0,15):

$$f_{m,c,05} = a_{CLT} f_{t,0,l,05}^{0,8}$$

Based on the respected results and on the verification concept for GLT it is proposed to use a comparable concept for CLT. Instead of splitting the edge normal stress and strength respectively the following simple verification function can be used, when CLT is treated like a beam-like element (e.g. single-span beam).

$$\sigma_{m,c,d} / f_{m,c,d} \leq 1$$

The difference between the design concept according to DIN 1052:2004 and the presented concept in this paper is in the range of about 50% and more (depending on the used strength classes and given COVs), where the DIN-concept is on the conservative side.

40-12-3 R A Jöbstl, G Schickhofer
Comparative examination of creep of GTL and CLT-slabs in bending

Abstract

Cross laminated timber (CLT) is – comparable with plywood – build up of orthogonal layers. The base materials are boards or in further steps the lamella, which match the requirements of glued laminated timber (GLT). Concerning standardisation, CLT is only regulated nationally in DIN 1052 and SIA 265. The main current application lies in residential buildings. Because of high slenderness in case of ceilings – span to depth ratios of 25 to 35 – serviceability design is generally decisive.

As a result – beside of short-term-deformations – long-term-deformations and related creep of discussed structures has to be kept in attention. According the current European design requirements, long-time-deformation has to be derived by reducing stiffness characteristic which is regulated by deformation factor k_{def}. The k_{def} for CLT has still to be identified.

For that long-term four-point-bending-tests on five-layered CLT slab elements acc. EN 408 have been carried out at the Graz University of Technology. To reduce the sample size, parallel tests on GLT slab elements have been done with same testing configuration to enable direct comparison of test results of CLT with a well examined and known structure. By testing, loading (approx. 9 % and approx. 25 % of failure load) and climate (approx. SCI and SC2) have been varied.

The results for CLT lead to about 30 -40 % higher creep value k, compared to GLT, which nearly conforms the difference between unidirectional build up GLT and comparable to CLT orthogonal build up plywood.

Data gained from total deformation demonstrate smeared creep value $k_{c,CLT,5s}$, over the five-layered cross section. Furthermore, based on these results and by some additional assumptions creep value $k_{c,9090}$ for rolling shear G_{9090} has been derived. This has enabled calculation of CLT-elements with various number of layers, which demonstrate relative low difference to five-layered build up. By application of this method a practice relevant and smeared creep value for the whole structure – as possible for other layered wood-structures like plywood and OSB – can be derived by the engineer.

Summary

At Graz University of Technology comparable creep tests on five-layered cross laminated timber- and glued laminated timber-elements have been accomplished. After about one year of loading, increase of deformation of CLT compared to GLT reached around 39 % to 47 %. By assumption of similar creep-functions for both examined products over cross section smeared deformation factors $k_{def,CLT}$ for two climates have been derived. By application of simple engineering approach a deformation factor for layers perpendicular $k_{def,9090}$ has been re-calculated for the rolling shear modulus G_{9090} (the modulus of elasticity perpendicular to the grain E_{90} has been assumed with 0) and hence concluded for over cross section smeared deformation factors $k_{def,CLT}$ for alternative cross section build

Presented deformation factors for number of layers with 3 to 19 show converging function with increasing number of layers. In that way calculated deformation factors are comparable with plywood ace. EN 1995-1-1:2004. Based on these results it is proposed to consider the product CLT concerning long-term-behaviour within the group of plywood, whereby for number of layers lower or equal 7 the deformation factor has to be increased by 10 %.

41-12-3 R A Jöbstl, T Bogensperger, G Schickhofer
In-plane shear strength of cross laminated timber

Introduction

The mass product Cross Laminated Timber (CLT) – general of spruce (picea Abies karst.) – is build up of an uneven number of layers of boards. Each layer is oriented 90° to the two adjacent ones. All layers are connected stiff by adhesive. The boards of each layer can be positioned with or without gaps. If gaps are used, their clearance is up to w_{gap} = 6 mm. In case of no gaps, lateral adhesive at the narrow sides of the boards can afford additional stiffness.

CLT is a large-sized derived timber product. Due to transportation issues each element has a length of approx. 13 m and a width of approx. 3 m. Thickness of a CLT plate depends on the number of layers (3, 5, 7 or 9 but up to 21 layers for bridge decks) and the range of application. When dealing with CLT for wall elements, it starts usually with 60 mm and ends up with an overall thickness of 400 mm, when used as a CLT-slab element e.g. with 21 layer for a timber bridge.

Regardless the main focus of applications lies in building constructions, which can easily also be recognized by the typical size of a CLT plate, which has been already mentioned above. CLT plates act as wall, slab and roof elements, carrying loads in and out of plane (e.g. shear walls).

Summary
The determination of the shear strength for CLT in plane is not possible with the procedure acc. to ETA 03.04/06. The technical shear strength values supply significantly higher values than established in EN 338. A shear failure was detected in all 20 tests with an optimized test configuration. The mean value has been identified with 12,8 N/mm^2 and the COV value with 11,3 %. Depending on the statistical distribution function, a 5% quantile value between $f_{v,0,90,CLT,k}$ = 10,3 N/mm^2 and 10,6 N/mm^2 can de derived. Comparable shear strength values of softwood plywood plates are about 50% higher than the strength value on basis of these 20 tests. Geometric parameters like the *a/t* relation or the gap between two boards in one layer have to be further studied in future research.

42-12-2 C Sandhaas, J W G van de Kuilen, L Boukes, A Ceccotti, Analysis of X-LAM panel-to-panel connections under monotonic and cyclic loading

Abstract
Cross-laminated timber panels (X-lam) are more and more used as construction material for buildings. For a sound and reliable structural analysis in both static and cyclic loading situations, the behaviour of connections between wall panels has to be evaluated. As the X-lam panels are very rigid, these connections are determining significantly the mechanical performance of X-lam buildings.

A typical in-plane wall-to-wall connection makes use of an LVL strip that is mounted as an nlay along the edges of two notched panels and is connected to these two panels by single shear fasteners, for instance self-drilling screws. As a consequence, the static and dynamic behaviour of this connection type differs from panels that are directly connected to each other due to the LVL strip and the subsequent double row of single shear fasteners. The LVL-inlay joints have to be modelled as a serial system of single shear fastener springs. Therefore, monotonic and cyclic tests on such connections have been undertaken in order to get input parameters for subsequent design models. Bending angles of fasteners have been measured. A comparison of the test results with dynamic measurements and calculations on a 3-storey building has resulted in the conclusion that in-plane wall-to-wall panel connections are an important factor in the structural behaviour of X-lam buildings, especially under dynamic loading such as earthquakes. Design procedures dealing with EC5 and EC8 concerning this connection type are discussed and their applicability is evaluated.

Conclusions
The load-carrying capacity of this connection type loaded cyclically can be calculated with the Johansen equations defined in EC 5 and the total load-carrying capacity of a whole joint can be established with n_{ef} = 1. No difference could be observed between specimens with 3 or 5 layers. The energy dissipation capacity of the connection type is high as can be seen in Table 3 where the equivalent viscous damping is calculated. Energy dissipation capacity is an important concept for seismic design according to EC8. Furthermore, the
impairment of strength in different cycles at one load step is not higher than 20%.

As for the testing procedure, the problem of rotational movements when carrying out connection tests should be resolved and praxis oriented restraining scheme should be developed and applied by researchers in order to keep results comparable and applicable. Furthermore, the limit of 30mm of testing protocols may be not sufficient, especially for large diameter dowels. The tested connections could support a displacement of 45mm without rupture which allows good seismic design.

The system values in terms of load-carrying capacity and stiffness found by the tests and calculations can be used for further modelling purposes of this important connection type in seismic design.

42-12-4 R Steiger, A Gülzow
Validity of bending tests on strip-shaped specimens to derive bending strength and stiffness properties of cross-laminated solid timber

Abstract
Cross-laminated solid timber (CLT) is a panel-shaped engineered wood product, assembled of cross-wise oriented layers of lamellas (mostly softwood). In contrast to other panel-shaped engineered wood products, CLT is not only used as component of structural elements, but rather for load

bearing plates and shear panels itself. The application of CLT used as load-bearing plates requires information on strength properties, the design, however, is often governed by serviceability criterions like maximal deflection and vibration susceptibility. Hence, predicting the respective behaviour of such panels requires accurate information about their bending and shear strength as well as their elastic properties.

Regulations regarding the derivation of performance characteristics, evaluation of conformity and (CE-) marking of wood-based panels for use in construction are given in EN 13986. According to this standard bending strength and stiffness of CLT have to be assessed following the procedure in EN 789. The latter requires 4-point bending tests of strip-shaped specimens with a width of 300 mm, cut off the CLT panels. The span has to be taken as 300 mm + 32 *t*, *t* being the nominal thickness of the CLT panel.

By comparing results of bending tests on strip-shaped specimens and on full panels it is shown, that neither strength nor stiffness properties derived by testing strip-shaped panels are appropriate to assess the respective properties of the original panels. The analysis of the test data covers bending strength, bending MOE parallel and perpendicular to the grain direction of the face layers as well as shear moduli. Rolling shear failures which frequently occurred when testing strip-shaped specimens could not be observed in destructive tests of whole CLT panels. Additionally a verification by carrying out static bending tests (deflection measurements) under different loading situations showed, that the overall stiffness properties (elastic parameters of the stiffness matrix) can, alternatively to EN 789 tests or estimations with the compound theory, be derived directly by a modal analysis of full-size CLT panels.

Summary and conclusions

Strength properties can best be assigned to CLT by means of the compound theory. However, the mechanical properties (strength and stiffness) of the layers have to be known which means that the raw material has to be strength graded. Deriving stiffness properties of whole CLT panels with modal analysis is a good alternative to estimating them on base of the mechanical properties of the singly layers by means of the compound theory. Especially in cases where the raw material is not strength graded or its mechanical properties are not known with sufficient precision, the modal analysis can help in assigning correct stiffness properties to CLT. After having proven the correctness of the method by static proof loading, the panel properties were compared to bending MOE and shear moduli derived from tests on strip-shaped specimens cut off the CLT panels. One part of the tests additionally focused on bending strength and failure modes.

The following conclusions could be drawn:

- Bending strength and stiffness of CLT panels can vary quite strongly within one single panel. For both parameters differences between the strength and stiffness of stripshaped specimens cut off the panels of up to 100% have been found. Hence it is not possible to derive strength and stiffness properties of CLT panels from bending tests of few or single strip-shaped specimens.
- The accuracy of the test results when performing bending tests of strip-shaped specimens according to EN 789 is increased with increasing sample size. Mean values of at least 5 – 6 specimens better describe the actual bending stiffness of the panels. Average differences then amount to 10% (E11) and 6% (E22) but can still reach 20%. As asked for in EN 789, characteristic values of strength and stiffness properties of CLT have to be derived on samples which fulfil the criteria of EN 14358.
- The variation of the stiffness properties depends on the degree of homogenisation of the actual CLT panel product. The smaller the components (lamellas) are and the less the variation in mechanical properties is (which can be reached by adequate strength grading of the raw material), the better it can be concluded from tests on strip-shaped specimens to the bending strength and stiffness properties of the whole CLT panel.
- Compared to gross CLT panels, local non-homogeneities and faults (knots, pitch pockets, deviated grain, not adhesively bonded contacts, cuts, grooves, cracks) take more influence on the mechanical properties of the strip-shaped specimens. The smaller the width of such specimens is, the more their load-bearing behaviour is affected by local defects and non-homogeneities due to faults in the raw material or due to way of producing the panels.
- The distances between middle layer parts not adhesively bonded at their lateral sides and the number of cuts and grooves, which are aimed at reducing the deformations of the CLT panel in case of changing moisture, take a big influence on the shear moduli. When deriving respective values on base of testing strip-shaped specimens this possible variation has to be taken into account by using empirical relationships.
- When testing strip-shaped specimens, shear and rolling shear failures occur quite frequently, whereas such failure modes could not be ob-

served when testing whole CLT panels to failure. There bending failure was dominating. Punching, however, should be regarded, especially with thin panels and products with grooves and layers not adhesively bonded at their lateral sides.

- Single tests on strip-shaped specimens may serve as an instrument of production control especially regarding the quality of bonding. They should, however not be used to derive mechanical properties of CLT panels. In scientific studies testing of strip-shaped cut offs of CLT should only be carried out on big samples. Geometrical dimensions should not be taken smaller than asked by the standards and generalization of conclusions in most cases is not possible (e.g. type of failure).

2.3 ENVIRONMENTAL CONDITIONS

20-18-1 A Mårtensson, S Thelandersson
Wood materials under combined mechanical and hygral loading

Introduction

The influence of environment on the performance of wood or wood based materials is significant in many applications. The mechanical properties of wooden materials depend on moisture content and dimensional changes induced by moisture variation often lead to displacements which are greater than those caused by mechanical loading. Moreover, interaction of moisture changes with mechanical loading can lead to excessive deformation of wooden structures.

In composite structures like stressed skin panels, composite beams and layered wooden products, differential swelling or shrinkage produces internal stresses, which can cause degradation as well as distortion of shape. Such effects may accumulate as a result of repeated moisture cycling during the lifetime of the structure, and may contribute significantly to reduce its durability.

In codes for wooden structures, e. g. Eurocode 5, design values of strength and stiffness parameters are made dependent on moisture class and load-duration class. This means that the effect of moisture conditions on basic material properties during the lifetime of the structure is considered in a simple way suitable for practical design. Due to the complexity of the underlying physical problems, however, a rational basis for description and quantification of these effects is still lacking. Clearly, there is a need for further research in this area, in order to establish a more reliable knowledge base for code specifications.

In addition, environmental effects should enter structural design not only on the "capacity" side, but also on the "loading" side. Moisture and temperature induced deformations are referred to as indirect actions to be considered along with and in combination with other types of loading. It is not quite evident, though, how such actions should be considered in the analysis of wood structures. For instance, if stresses due to restrained shrinkage or swelling are calculated by normal engineering methods, unrealistically high values are usually obtained.

Consider, as a simple example, a wooden element which is drying with the shrinkage fully restrained, Fig. 1. According to linear theory of elasticity, the stress change $\Delta\sigma_i$ due to restrained shrinkage is given by

$$\Delta\sigma_i = E\Delta\varepsilon_s \tag{1}$$

where E is the modulus of elasticity and $\Delta\varepsilon_s$ is the shrinkage strain that would occur if the element were free to shrink.

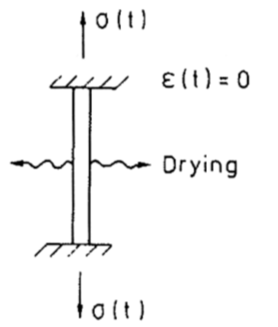


Fig. 1. Drying wood element under full restraint.

Using representative values for $\Delta\varepsilon_s$ in moisture class 1 and design values for E it is found that $\Delta\sigma_i$ calculated from Eq (1) may be as high as 50% of the design value of the tensile strength for wood (parallel to the grain) and wood based products. It should be noted that the design value of E is specified in codes under the assumption that a low value of E is unfavourable. When E is used to calculate restraint stresses the opposite is valid, implying that a consistent choice of design parameters would give still higher values of $\Delta\sigma_i$.

Stresses due to imposed deformations may be neglected when they contribute to 'ductile' types of failure as for wood in compression. But for 'brittle' types of failure as for wood or adhesive joints in tension and shear, such stresses can be expected to contribute significantly. Accordingly, it is of importance in many situations to be able to predict stresses induced by moisture variations in an accurate way. A main purpose with the present paper is to contribute to the solution of this problem.

Concluding remarks

The following main conclusions can be drawn from the present investigation:

1) Tests on hardboard show that the effect of moisture variations on tensile creep is surprisingly small, in view of the very significant mecanosorptive effects observed for other wooden materials.

2) The mechano-sorptive effects in the tested hardboard was found to be much greater in relaxation tests under moisture variations. Typically, the initial stress was reduced to 1/3 at the end of the first full moisture cycle.
3) The increase in tensile stress due to restrained shrinkage in wooden materials can be expected to be very moderate and only a fraction of that predicted by linear elastic theory.
4) When a full drying/wetting cycle has been completed, there will be a significant stress change in the opposite direction.
5) A simple constitutive model taking into account mechanosorptive behavior was found to give good agreement with creep type experiments when calibrated against these.
6) When the model was checked against relaxation type experiments (using the same material parameters), however, it showed poor agreement.

Based on published experimental evidence, Grossman has given a number of requirements for a model which exhibits mechano-sorptive behaviour. To this list can be added that the model shall be able to predict both relaxation and creep behaviour under varying moisture conditions. Tests of relaxation type are almost nonexistent in the literature, and it would be of great interest if experimentalists could perform such tests for different wooden materials and for different loading modes. For instance, prediction of drying stresses and cracking in timber requires such information for wood under tension in the radial and tangential directions.

22-11-1 K Erler
Corrosion and adaptation factors for chemically aggressive media with timber structures

Objective
The calculation of the load-bearing capacity of new timber structures or the check of that of old timber structures will be accomplished a few years hence in all European countries by adopting the limit states method. In this connection, the most important outside influences are being taken into consideration by applying adaptation factors, e.g. the kind and duration of the stress and strain (loading or action), the moisture of timber as well as adaptation factors for chemically aggressive media The last-mentioned influence is included hitherto only in a few regulations (specifications) like – for instance – in the Polish Timber Construction Code; however, only a limited number of media is being mentioned and no adaptation factors are being referred to.

The aim of the studies and investigations described hereinafter is to clarify corrosion processes and rates with timber components for the relevant aggressive media Conclusions may be drawn concerning reductions for the design. It is particularly important from the national-economic point of view to determine and check with which influences an insignificant corrosion is occurring.

The determination of the influence of chemically aggressive media on the load-bearing capacity of timber structures with long service-life periods and different conditions of utilization shall be supported by documentary evidence.

The objectives consist in
– enabling a statical check of old structures with regard to the corresponding degree of damage done to them, and
– taking into consideration in the case of new planning and design activities (projects) the reduction of the load-bearing capacity by means of scientifically ensured adaptation factors.

In this connection, relevant influences such as air humidity, kind of the acting medium and cross-sectional dimensions of the timber components concerned must be considered and applied when determining the adaptation factors.

Summary
The corrosion of timber components when exposed to a chemically aggressive influence is being described with the evaluation of new results and findings of studies, tests and investigations. In general, timber as building material is more resistant to the majority of chemical agents than steel and concrete. The significant parameters for the intensity of the corrosion with timber are indicated. The destroyed boundary zone of the width *d* which is assumed to be no longer able to support load is determined subject to the duration of action (period of occupation of the buildings concerned), to the kind of medium and to the air humidity. Based on the description, the depths of penetration of the agents and the reduced strengths can be determined. Subsequent to the testing of specimens sampled from the timber structure concerned – e.g. test cores – it is possible to exactly determine the boundary zone being no more able to support load or

to draw an approximate value for same from a graph (diagram). The residual load-bearing capacity of existing, structures or components can be checked and verified by means of the timber cross sections reduced by *d*.

In the draft of the new GDR Code "Timber Construction - Load-bearing Systems" adopting the method of limit states, for timber the most significant agents have been assigned to ranges (grades) of aggressivity, and stress degrees have been defined accordingly. Subject to the stress degree and to the cross-sectional dimension of the timber component concerned, adaptation factors for the influence of aggressive media have been fixed.

29-11-1 P Galimard, P Morlier
Load duration effect on structural beams under varying climate influence of size and shape

Abstract
The Duration of Load effect has been of increasing interest for both the understanding of wood or wood products in long term behaviour and for the determination of safety factors for engineering purposes. Many kinds of testing, mainly in bending, have been made but the large amount of configurations has always been reduced to few moisture contents and load histories. However, it is still not easy to compare all the results for the protocol of testing and presenting the results is not yet a standard.

Different attempts of evaluating the stress level have been done and their accuracy depends on what is wanted to be shown. The methods are applied on 2 meter LVL beams subjected to bending in natural environment. Forty beams are stepwise loaded in an open shed in the Laboratoire de Rhéologie du Bois de Bordeaux in order to have a failure of 80% of the beams within two months. The step levels are from 50% to 90% of the average short term strength. Two beam depths (100 & 150 mm) are tested. The beams can be sealed or unsealed. The curvature, the time-to-failure and the climatic conditions are monitored.

A ranking method is used to show the main trends of the duration of load results: the time-to-failure are not affected by daily or short time variations of hygrothermal conditions, but by the two month mean variation of the moisture content.

The long term strength reductions are evaluated and finally compared to standard (EC5) ones. This work is a part of a European AIR-Project which has been set up to get experimental data on DOL of LVL and glulam straight, notched and curved sized beams.

Conclusion

- We have shown, on straight LVL beams that opposite conclusions can appear from the same data depending on the way of presenting the results.
- However, the influence of the general changing of the climate was more important than the absolute moisture content.
- The strength reduction estimation has roughly given the same results from the two tested methods; so #50% beams can be accurately used.
- The short term strength has been influencing the results in a large scale, because of the moisture content reference and of the definition of the stress level.
- The duration of load effect is fairly depending on the tested shape: it is more important on straight beams than on notched beams.
- The EC5 krnod factor, for LVL beams, seems to be optimistic for EC5 Service Class 2 and conservative for EC5 Service Class 3.

A protocol of Duration of Load test and analysis is still needed because of the numerous testing configurations, the variability of the result analysis, and the influence of the analysis on the conclusions that can be contradictory.

30-11-1 R H Leicester, C H Wang, M N Nguyen, G C Foliente
Probabilistic design models for the durability of timber constructions

Introduction
The theory of structural engineering for timber construction is now sufficiently well defined that computational design procedures can be developed and even codified. By contrast, design for durability is still very much of an art, with design solutions varying from person to person. More usually, design for durability is omitted and the control of performance is undertaken by complying with 'good building practice'.

The key to developing a design procedure is first to have predictive models for durability. Because of the uncertainties associated with these models, they will of necessity have to be probabilistic models. Once such models are available computational design procedures can be developed and codified; optimised strategies for asset management involving design, maintenance and replacement can be derived; optimised procedures may be used to develop 'good building practice'. Perhaps most importantly, the availability of predictive models will enable innovative designs to be undertaken with confidence, and without the necessity of relying on past experience.

The purpose of this paper is to introduce a major investigation on timber durability being undertaken in Australia. It is a three year project, involving 5 research organisations at a cost of about one million dollars per year. An ambitious and key aspect of this project is to develop a probabilistic model for predicting the durability of any type of timber construction located anywhere in Australia.

The following describes the general concept of a probabilistic model, together with some of the practical aspects involved in developing such a model.

Concluding comment

The project to develop prediction models has had a slow start, largely because of unanticipated difficulties in formulating suitable attack mechanisms. However, once this has been accomplished, the next phase will be to develop design procedures and/or software that are user friendly. The complex effects of climate may need to be reduced to simple index parameters, or failing this, to regional parameters; similarly it may be effective to reduce the complex features of building construction to index parameters or at least to idealised building types. However, in general it will be desirable as far as possible, for all aspects of the prediction model to be globally applicable.

Furthermore, it would be expected that the predictions on durability would be associated with a high degree of variability. As a result, the use of refined models and accurate parameter estimates are not warranted. For this reason alone, within the Australian project the use of sophisticated methods will be avoided and emphasis will be placed on the use of simple methods both for analysis and for estimating the required model parameters.

33-12-1 J Jönsson, S Svensson
Internal stresses in the cross-grain direction of wood induced by climate variation

Abstract

A method to determine the internal stress state across the grain of glue laminated wood (glulam) is described in this paper. An experimental equipment for cutting specimens and measuring released deformation is used. The internal stress state can be calculated from the observed released deformations. For evaluation purpose a small and well-defined test series was carried out for specimens with different climate treatment prior to testing. Results for seasoned specimens show that internal stresses exist in glulam without the presence of moisture gradients. Results for specimens with induced moisture gradient show that the stresses become larger when moistening from a specific climate A to another climate B than when drying from B to A. The largest mean stress level found in the test was about 0,7 MPa, which is higher than the characteristic value of tensile strength perpendicular to the grain!

Conclusions

In the experimental part of this study a contact free measuring technique was used. It turned out to be a fast and easy way measure the deformations. The speed of measuring is important since small test specimens are used, a change of moisture content is not tolerable. The accuracy is acceptable (1/70 mm), but can be improved by using a camera with a higher resolution. A dynamic test method was used to determine the modulus of elasticity, mainly for two reasons, the speed of measuring and the fact that the slices were not suitable for a customary tensile test. The relationship between Es and Ed for different moisture content and varying annual ring pattern can be treated as linear.

The following conclusions can be drawn from this study.

- There is little difference between the values of modulus of elasticity for the four outermost slices. As expected the modulus of elasticity increases with increasing amount of radial wood.
- The influence of moisture content on the modulus of elasticity is most dominant on the slices in the middle part of the specimen.
- The modulus of elasticity for the middle part of the specimen is between 2-2,5 times the outermost part.

– In seasoned specimens i.e. uniform dried and moistened specimens, there were internal stresses!
– The maximum tension and compression stress for seasoned specimens was approximately 0,4 and 0,2 MPa.
– The stress level between the two groups of gradient specimens differs with approximately a factor 2.
– Results for specimens with induced moisture gradient shows that the stresses become larger for moistening from a specific climate RH 40% to another climate RH 80% than drying from RH 80% to RH 40%.
– The largest mean stress level was about 0,7 MPa, which is larger than the characteristic value of tensile strength perpendicular to the grain!

34-12-4 J Jönsson
Moisture induced stresses in glulam cross-sections

Abstract
Preliminary results from an experimental investigation to determine the internal stress state perpendicular to grain in glulam were presented at the CIB meeting in **Paper 33-12-1**. Further results from this research are presented in this paper together with improvements of the experimental methodology. The modifications concern digital camera equipment for strain measurements with better optics, fixtures for holding the camera, the method of releasing the deformation and the technique to moisture seal specimens. Altogether the changes in methodology has made the testing more rational and increased the reliability of the measurements. However, repetition of tests made' before these improvements have confirmed the results presented in the previous paper.

Three categories of tests are presented where strain and stress distribution in the cross grain direction are determined. The different categories are specimens seasoned in constant humidity, specimens exposed to an artificial single climate change and specimens exposed to natural climate outdoors under shelter. Results for seasoned specimens show that internal stresses exist in glulam without the presence of moisture gradients. Results for specimens with induced moisture gradient show that the stresses become larger when moistening from a specific climate A to another climate B than when drying from B to A. The largest stress level found in the tests was about 0,6 MPa. The last category of test (natural climate) shows a large variation in strains and stresses. The tensile stress level in the outer part of the glulam cross section exceeds the characteristic strength of 0,5 MPa during a period of approximately 80 days.

Conclusions
The stress values given below refer to average stresses along slices disregarding end effects. The maximum stress levels in the cross section are significantly higher. A preliminary estimate is that the stresses may be about 30 % larger than the values quoted below. The following conclusions can be drawn from the tests.
– Significant internal stresses were found in seasoned specimens with uniform moisture content.
– The maximum tension and compression stress for seasoned specimens is of the order 0,2 MPa.
– For specimens with induced moisture gradient, the stresses become larger for moistening from a specific climate RH 40% to another climate RH 80% than for drying from RH 80% to RH 40%.
– The tensile stresses are twice as high and the compressive stresses are three times
higher under moistening between RH 40 % and 80 % than under drying or vice versa.
– The largest tension stress level observed was about 0,6 MPa, which is larger than the characteristic tensile strength perpendicular to the grain.
– Specimens exposed to natural climate outdoors under shelter show a large fluctuation in released strains and internal stresses during a 160-day period from November to April.
– The stress level varied in the outer part of the cross section from -0,65 to 0,32 MPa and from 0 to 0,85 MPa in the middle part.
– The tensile stresses exceeded 0,5 MPa during a period of 80 days, out of a total testing period of 160 days.
– The main conclusion is that moisture induced stresses must always be taken into account in one way or another for failure modes involving tension perpendicular to grain.

36-11-1 R H Leicester, C H Wang, M N Nguyen, G C Foliente, C McKenzie
Structural durability of timber in ground contact

Abstract
This paper describes the development of a model to predict the strength of timber poles and rectangular sawn sections subjected to in-ground attack by decay fungi. The models are based on data obtained from extensive in-ground stake tests that were monitored over a period of 30 years and a limited number of full size pole and rectangular sawn sections. The model takes into account timber species, preservative treatment, maintenance practice and local climate. The model predictions are in quantified form, and hence can be used to make cost-optimised decisions for asset management purposes.

Concluding comments
Using the completed model, it is now possible to compute the deterioration over the years of the load capacity of structural elements constructed with timber in ground contact. The model is directly applicable to about 80 species of timber, CCA and creosote treatments and a variety of maintenance procedures. Through use of a climate index (based on rainfall and temperature parameters) the procedure is applicable to all locations in Australia, from temperate to hot tropical regions and from rainforest to desert areas. With slight modifications, it should be possible to apply this model to any structure, fabricated from any species and located anywhere in the world.

Current progress is related to calibrating and modifying the model according to the experience of experts. In addition, it is necessary to find a procedure for taking into account the deterioration due to mechanical degradation that occurs in the and regions of Australia.

Possibly the most important aspect of the engineering model described is that it provides a unified framework within which can be placed all types of knowledge, such as knowledge obtained from laboratory data, field data, expert opinion and accepted good practice. It can also be used to assess the monetary value of new knowledge.

38-11-1 R H Leicester, C-H Wang, M Nguyen, G C Foliente
Design specifications for the durability of timber

Abstract
In the development of prediction models for the durability of timber construction, data was obtained from several sources. These include basic physics and biology, field tests on small clear wood and steel specimens, field tests on full size structures, in-service structures and expert opinion. This paper provides a discussion on the role played by these various sources of information, and also procedures for drafting rules for engineering design codes.

Conclusion
There are several reasons as to why the formal application of Bayesian procedures is not suitable for application to the durability model. One reason is that the models are highly nonlinear and complex. Another reason is that the in-service data required for use in such a procedure is usually not completely defined.

The current procedure is to use an "evidence-based" approach, i.e. to ensure that the models comply with all the data sources mentioned including that of expert opinion. Another possibility is to develop models that focus on high risk performance rather than mean values.

The suitable choice of a reliability index β to be used in equation (2) is an interesting topic for investigation.

38-11-2 M Häglund, S Thelandersson
Consideration of moisture exposure of timber structures as an action

Introduction
Moisture exposure is a very significant factor for serviceability as well as load bearing capacity of structural timber elements and systems. Not only the moisture content level but also the variation of the moisture content is of great importance for the performance of timber structures and engineering, wood products. One critical factor is varying relative humidity in the ambient air, and thus non-uniform moisture content in wood cross sections. External and internal restraint of hygro-expansion will then create stresses mainly perpendicular to grain. Such stresses may cause cracks, reducing the load bearing capacity of the individual timber element and thus

the whole structure. Experiments performed on glulam subject to natural sheltered outdoor climate conditions have shown that climate variations can induce significantly high stresses (in addition to stresses from applied loads) in the range of two thirds of the characteristic strength value.

Determination of moisture content profiles due to natural moisture content variations is hence important in order to better understand and quantify how these variations affect timber. Today's design codes use service classes to account for moisture induced effects, but since the class selection is only based on anticipated equilibrium levels, the nature of timber exposed to moisture is not fully reflected. In order to improve design codes, it has been proposed that instead of using strength reduction factors, induced moisture stresses may be treated as an ordinary design load to be combined with effects from other loads (for example snow and wind load). For this purpose, a moisture exposure model that reflects the nature of the variations – the dynamics – of moisture in the ambient air is desired. The model should also include temperature, since this is needed for conversion of outdoor moisture levels to corresponding indoor levels. Moisture transport in wood can be modelled as a diffusion process to describe how penetration effects depend on temporal variation in relative humidity, RH. For example, Arfvidsson showed that the penetration depth (here defined as the depth where RH differs 1%) in the tangential direction for a semi-infinite solid spruce element exposed to diurnal, rectangular cycling between 50% and 95% RH is about 10 mm, whereas an annual cycling increased the penetration depth approximately ten times. The exact figures are of course highly dependent on what moisture transport model being used and its related parameters, but it demonstrates the transient properties of wood. In reality, however, the relative humidity varies irregularly with occasional large differences in magnitude from day to day.

In this paper, a general and possible approach to describe moisture exposure based on real recorded data is proposed. Specifically, the methodology is applied for climate data at two locations in Sweden, Stockholm and Sturup in southern Sweden. Moisture distributions in both time and space are calculated with a 2-D finite element program. Selected results on the response of timber subject to naturally varying climate at different climatic locations are presented. The obtained results may be used as a basis for code specification of moisture effects with targeted reliability levels.

Discussion and conclusions

The results presented in this paper are part of an ongoing research, which has the goal to characterize moisture as an action on timber structures. The results so far indicate that it is possible to make statistical estimates of moisture effects in terms of annual extremes, from given input of recorded meteorological data. This is consistent with modern safety concepts. A time series methodology has been employed, by which synthetic sequences of temperature and relative humidity can be generated. Simulated sequences may be used for probabilistic investigations of the response of timber structures to varying climate, for refined definitions of service classes and for calibration of climate related factors in timber design codes.

The following conclusions can be made from the research so far

- Extreme values of relative humidity indoors (service class 1) seem to be more severe than is usually anticipated, which means that the risk for adverse effects of moisture exposure can be significantly higher than normally conceived in design.
- The moisture content variation (difference between maximum and minimum MC, defined as 5 year return value) is of the same order of magnitude for indoor conditions and for outdoor conditions.
- Very low humidity levels with high shrinkage can cause problems from the point of view of serviceability, especially for wood products and panels with small cross dimensions. This problem is seldom recognized in design.
- The stochastic process model developed can be used to simulate climatic exposure as sequences of daily averages. The prediction from this model of extreme values with return periods of the order 5 years seems to be reliable. For lower probabilities the values produced by the process model are uncertain.
- Considering moisture exposure on timber structures with nomial dimensions, however, extreme values of daily average humidity will be damped out by the slow moisture transfer in wood. The stochastic process model can be expected to give reasonable results in such cases.
- For timber structures with larger dimensions, swift changes at the boundary are damped out fast and leave the inner parts of a cross section unaffected, whereas slow changes, as annual variations, affect the whole beam. Timber performs as a low-pass filter, allowing low frequencies to pass but filter out higher ones.

For handling of moisture effects in design of timber structures it is proposed that guidelines and principles are developed to determine consistent design values for

– Expected variation between maximum and minimum moisture content averaged over cross sections to be able to determine moisture movements with adequate reliability.
– Expected spatial variation of moisture content within cross sections to predict strength reduction due to moisture induced eigenstresses for failure modes perpendicular to grain.

43-20-1 H W Morris, S R Uma, K Gledhill, P Omenzetter, M Worth The long term instrumentation of a timber building in Nelson New Zealand - the need for standardisation

Introduction

High quality data on building performance is needed as timber is increasingly used for medium and large commercial, industrial and multi-residential structures in New Zealand and around the world. Timber buildings 6 to 9 storeys high have been constructed in Germany, England, Canada and Sweden. In New Zealand we expect a 3 storey post tensioned building in Nelson will be the trailblazer for taller structures here. Performance based standards require an understanding of building response at a range of serviceability levels as well as for life safety.

Large scale earthquake shake table experiments, such as those undertaken in the USA, Italy and Japan, provide very good initial response data for timber structures. Multi-storey timber structures in seismic regions are designed for structural ductility with the response analysis based on assumptions for damping which ill be different from the laboratory to fully fitted out buildings. It is important that monitoring is used to collect in-situ performance data, particularly in seismic areas, to provide a solid basis for standards for a range of fully finished structures.

For long span structures the timber deformation is a significant serviceability design parameter so the long term deformations need to be predictable. Monitoring provides understanding of the reliability of these design values.

Instrumentation of buildings takes considerable effort and standardisation is necessary to maximise the comparability of data. Timber has a smaller research base than the major structural materials so early standardisation will maximise the international usefulness of a performance database. Monitoring is discussed in reference to the GNS Science GeoNet network and the Nelson Marlborough Institute of Technology (NMIT) timber building in Nelson and is used to pose the challenge as to how to develop a standard approach for wider application.

Discussion and Conclusions

More data on real building responses to earthquake and strong wind events will mean better verification of design assumptions. The impacts, effects and costs of measured high level serviceability events would significantly contribute to better performance based design. This requires a number of buildings to be available to collect useful data. GNS Science have experience with modern installations and a comprehensive but modest plan for New Zealand wide installations. A compatible international programme would significantly strengthen such a database, particularly for timber structures 4 storeys or higher.

At the time of writing instrumentation is is only partly installed in the NMIT buildings. The timescale for design and installation has been very tight and has had to fit around the realities of construction project scheduling. The project has considerable complexity as well as design parameters to be determined. At the University of Auckland it became clear that a set of guidelines and standards would have made for better early decisions. In the longer term such standards would mean direct comparability between projects.

2.4 FRACTURE MECHANICS

19-6-3 K Wright and M Fonselius
Fracture toughness of wood - Mode I

Abstract
Mode I fracture toughness of wood can be detemined with CT-specimens, as proposed in the ASTM-standard. In the different orthotropic systems the value of fracture toughness K_{IC} was 200 - 360 kNm $^{-3/2}$ for pine and 200-340 kNm $^{-3/2}$ for spruce. The coefficients of variation were about 10 %. The values fit well with those presented in the literature.

Because of the great nonlinearity, for plywood), the K_{IC} value is not a valid material parameter. For the comparison, K_I -values were calculated of the maximum load to be 130 - 300 kNm$^{-3/2}$. The coefficient of variation was about 10 % for the TL-systems. Lathe checks weaken the TR and RT systems.

If the fracture toughness is considered to be proportional to the density, the correlation for pine and spruce is $R^2 = 0.6$. The numerical value of fracture toughness in kNm$^{-3/2}$ is roughly the same as the density in kg/m^3.

If fracture toughness is considered to be a linear combination of the square of density and other variables, somewhat better correlations are achieved: for pine and spruce $R^2 = 0.8$.

Discussion
The different emphasis in the Fonselius and Wrights works complicates the comparison of the values. However, because the density of spruce is lower than that of pine, the fracture toughness also is lower, as expected. Accordingly, the *K*-fracture toughness for plywood is about the same as for spruce, but the nonlinearity is larger: therefore plywood will show a larger *J*-value.

The CT-specimen seems to give reliable fracture toughness values. The isotropic polynomial solution is near the values calculated by the means of compliance measured in the *J*-test.

Compared with the results found in the literature (Wright and Leppavuori 1984), the following can be stated.

- The relations between the orthotropic fracture systems are not species independent. For Douglas Fir, the TR and RT systems are much stronger than for our materials, but the TL-system is somewhat weaker than found for pine. A possible reason for this is the different amount of rays in these species. The size of the tree used to make specimens out of, also may influence. In our cases the annual rings had a visible curvature. Douglas Fir stems are usually larger, so the specimens have likely been more ideal.
- Density is the most important parameter indicating the TL-RL fracture toughness, besides of the angle of the fracture plane. Roughly, the numerical value of-KIC fracture toughness in kNm-3/2 is the same as the density in kg/m3
- The effect of moisture is not satisfactory clarified. Our results point to an insensitivity to moisture, but sensitivity to moisture gradients. In the literature, no clear tendency is found.
- The influences of size and loading rate are negligible when common short-term loading is used.
- The PLV-results show clearly the difference between rotary-peeled and sawed plies; they give more uniform K Ic values for the different fracture systems.

19-6-4 K Wright
Fracture toughness of pine - Mode II

Abstract
Mode II fracture toughness has been studied by means of beam specimens. The material used was of the same lot of Pinus Silvestris as in the previous mode I tests. In short term tests, the value of K_{IIC} fracture toughness was found to be 1760 kNm$^{-3/2}$.

Fracture toughness is best predicted by density and the quarter of density. Moisture content, annual ring angle or other variables had no effect. Correlations were mostly poor, which may depend on the relatively large fracture toughness of the wood from northern Finland compared to its density.

In the long-term tests, the deflection growth was proportional to the bending stress and the 1/3th power of time. The 90 % stress level corresponded to a median life time of 5 h.

Conclusions
- The fracture toughness values calculated by different methods do not markedly deviate from each other. Stress intensity depends linearly on, the crack length.

- The short-term tests had a mean moisture content of 14 % and density of 420 kg/m3. Fracture toughness was 1760 kNm-3/2. This is 5.4 times mode I TL fracture toughness, which fits well the literature study.
- In regression analysis, significant variables were density ρ and ρ2, but the correlation was low, R = 0.54.
- The long-term tests gave fracture only by 90 % stress level; median life-time was 5 h. The stress level 45 % proved to be too low.
- Deflection increase rate was for the thin specimens twice that for thick specimens. The deflection increase rate was proportional to t1/3
- Crack growth could not be studied due to the rapid fracture process.

2.5 GLULAM STRENGTH

19-12-1 J Ehlbeck and F Colling
Strength of glued laminated timber

Introduction

Strength and stiffness values of glued laminated timber are dependent of the design of glulam beams, i.e. the properties of the outer and inner laminations as well as the quality of the finger joints. On principle, the outer laminations are of better or at least the same quality as the inner laminations. The extreme sixth of the depth or at least two laminations on either side of the beam are defined as outer laminations. It is necessary to differentiate in that way because the outer laminations decide essentially the bending strength of the glulam beam, whereas the tension perpendicular to grain and the shear strengths are in most cases controlled by the properties of the inner laminations.

19-12-4 F Zaupa
Time-dependent behaviour of glued-laminated beams

Abstract

The long-term deformation and failure of inflected glued-laminated spruce beams subjected to various levels of stress were studied over a three year period.

For the beams subjected to lower stress (30% of the short-term ultimate load), after three years the total viscous deformation was found to be equal to 55% of the initial elastic deformation; however, the phenomenon had not yet ceased. The beams subjected to greater stress (60% of the short-term ultimate load), failed after between 8 and 28 months. The time-dependent deformation developed up to the onset of failure was, after eight months, 30%, and after twenty-eight months, 55% of the initial elastic deformation.

The study revealed the dependence of the creep strain on the level and duration of loading. Observations on the mechanisms of the onset of the self-accelerating process of failure were formulated.

Conclusions

The results of the experiments carried out so far in this first, direct analysis make it possible to draw some conclusions.

The rheological behaviour of glued-laminated beam, subjected to long-term bending loads appears to fit the linear visco-elasticity of solids.

The time dependent delayed elastic deformation which, in the case of beam 4, was also measured in the unloaded and reloaded phases, was well evident: of the order of 17% of the initial elastic deformation, which represents about 65% of the total time dependent deformation. It reached completion in a period of some months.

There is, then, at least in the presence of high stresses, also an appreciable degree of irreversible creep, which indicates a viscoplastic deformation component. These values of delayed plastic strain are, however, of the order of 10% of the initial elastic deformation and 50% of the delayed elastic deformation.

In many ways, behaviour of this type appears comparable to that of concrete, and confirms the possibility of producing interesting mixed structures of glued laminated wood and reinforced concrete, even if the various initial elastic, delayed elastic and unrecoverable strain components have different impacts on the total deformation in these materials. The substantial differences seem to be reducible to the critical phases in the lifespan of the two materials. For concrete the first phase is critical because of the improvement with age of its rheological characteristics and its mechanical properties. With wood, because it has no analogous age dependent characteristics, later phases of its lifespan are critical: after a certain period of time a process of accelerating progressive failure may be manifested, if the permanent loads exceed about 50% of the short-term ultimate load.

The latter phenomenon comes about suddenly, without apparent premonitory signs, following a loss of resistance to tensile stress in the outermost fibres. The failure of the member, however, reaches completion after a period long enough to allow corrective measures, if the situation is recognized immediately. The site of the first fracturing corresponds to defects in the structural member, such as knots, weak joints, even slight damage to the fibres, and other types of local dishomogeneity, to which the wood proves to be rather sensitive.

Regarding this particular aspect, it is evident that, because of the limited degree of heterogeneity that characterizes accurately manufactured composite laminated elements, chosen on the basis of high quality control criteria, they may offer decisively more favourable conditions than the corresponding solid wood elements.

To the extraordinary ease of manufacture, transportation and installation of these elements there corresponds a high vulnerability, to errors and carelessness that should not be underestimated.

21-12-1 K Komatsu, N Kawamoto
Modulus of rupture of glulam. beam composed of arbitrary laminae

Abstract
An equation is derived for predicting the modulus of rupture (MOR) of glued laminated beam (glulam) composed of laminae with arbitrary grade, arbitrary size and arbitrary arrangement at the most critical section in the beam. Numerical experiments based on the Monte Carlo method are applied to compare the theory with the experimental results obtained on the full-size Douglas fir glulam beams. Comparisons between numerical experiments and full-size experiments indicate that the MORs of glulam beams can be predicted if the distributions of MTS (maximum tensile strength) and MOR of laminae as well as the co-relations between MTS and MOE(modulus of elasticity) and MOR and MOE of laminae are known. Size effect (depth effect) can be also explained well by the equation proposed in this study.

Conclusion
Modulus of rupture (maximum bending moment of beam/ section modulus of beam) of any glulam beam will be able to be predicted by applying the multi-layer composite beam concept as described in this report, if the MTS (maximum tensile strength) and MOR (flat-wise maximum bending strength) of laminae and other information for cross section of the beam are known.

In order to complete this approach more in general, research involving the longitudinal variation of material properties would be required.

21-12-3 J Ehlbeck, F Colling
The strength of glued laminated timber (glulam): influence of lamination qualities and strength of finger joints

Preface
In annex 2 of the draft of EUROCODE 5 "Common unified rules for timber structures" published by the Commission of the European Communities in October 1987 to seek comments by the member states a proposal for requirements of laminations and end-jointing for glued laminated timber grades is given with appertaining characteristic strength values for these grades. It is the intention of this paper to explain the background ideas which led to this proposal and to challenge any comments on this glulam strength system. This glulam grade system is mainly based on the bending strength whereas other strength properties, i.e. tensile and compressive strength perpendicular to grain as well as shear strength values, are estimated data based on experience at the present time.

23-12-1 J Ehlbeck, F Colling
Bending strength of glulam beams – a design proposal

The scope of a current research project' is the investigation of the bending strength of glulam beams aiming at the development of design proposals. The 'Karlsruhe calculation model" - a finite element model calculating the strength of glulam beams by means of Monte Carlo simulations - was thought to achieve this purpose.

The simulations showed, however, that the strength of glulam beams is a very complex field and that it is very difficult to describe the influence of one single parameter. The "Karlsruhe calculation model" takes into account every possible tendency, but the problem was to describe these tendencies mathematically.

Therefore, a statistical model (Colling 1990) was developed, which divides the totality of glulam beams into two groups: beams with wood failure (knots) and beams failing due to finger joints. On the basis of the "true" strength distributions of these two groups, it is possible to calculate the strength characteristics of the resultant glulam beams.

According to this model, the strength distribution of the final product glulam orientates itself very strongly by the lower of these two strength distributions and the characteristic bending strength of glulam beams is governed by the lower 5th-percentile of this group ("weaker material").

In fig. 1 the characteristic bending strength (5th-percentile) depending on KAR, ovendry density and MOE of the laminations is shown for beams with finger joint failure ($x^0_{5,fj}$) and wood failure ($x^0_{5,wood}$). The index "^{0}" indicates, that the strength values are valid for a standard beam with a

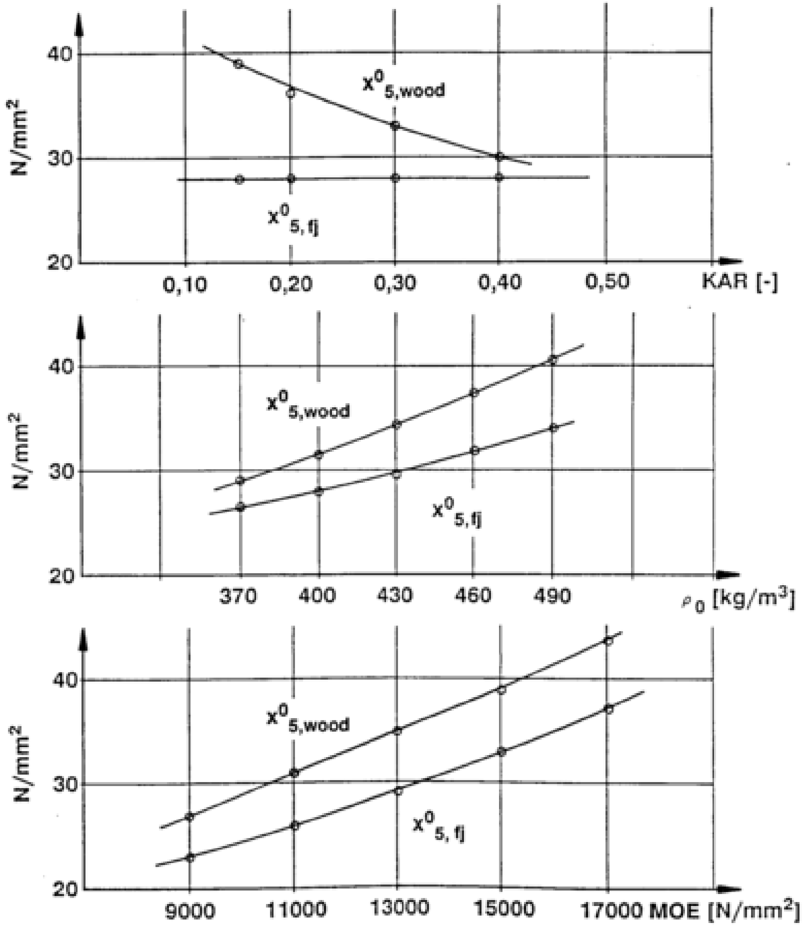


fig. 1: Characteristic bending strength of glulam beams with finger joint failure ($x^0_{5,fj}$) and wood failure ($x^0_{5,wood}$)

depth of 300 mm. Based on these calculation results, beams with finger joint failure were found to be the "weaker material" having the lower 5th-percentile. This tendency even increases with increasing beam dimensions, because size effects are more pronounced in case of beams with finger joint failure than in case of beams with wood failure. This may be explained by the higher variability of strength data in case of beams with failure due to finger joints.

Conclusion

Based on the characteristic bending strength $f^0_{m,k}$ of a standard beam under constant loading, the characteristic bending strength $f_{m,k}$ of any glulam beam may be calculated as

$$f_{m,k} = \left(\frac{L}{5400} \cdot \frac{H}{300} \right)^{-0,15} k_F f^0_{m,k}$$

where

L and H are length and depth of the beam in mm,

k_F is a load configuration factor (= 1 in case of a single span beam with uniformly distributed load).

The finger joints in the beam have to meet the following requirement:

$$f_{m,k,fj} \geq 1,15 f^0_{m,k}$$

It is essential to point out that in case of beams, systematically built up with laminations having significantly different MOE-values, the ultimate bending stress (in the outermost lamination) must be calculated according to the theory of transformed sections.

23-12-3 H Riberholt
Glulam beams. bending strength in relation to the bending strength of the finger joint

Scope

The scope of this paper is to illustrate the relation between the bending strength of glulam beams and that of the finger joints. It is written as a contribution similar to that in Ehlbeck & Coling (**Paper 23-12-1**)which also deals with this subject. But this paper employs another experimental source.

Conclusion
The relation between the bending strength of glulam and that of the finger joints seemsto depend on several factors. Among these are partly the size of the finger joints represented for example by the thickness of the lamination, partly the relation between the strength of the finger joint and that of the rest of the lamination. This could be explained by the fact that the weak finger joint is brittle and the surrounding wood is in the linear state so stress redistribution will not take place.

23-12-4 H Riberholt, J Ehlbeck, A Fewell
Contribution to the determination of the bending strength of glulam beams

Preface and explanation
In the draft it has been necessary to introduce 2 tables of strength classes,one for glulam with a homogeneous cross-section and one for glulam where different lamination grades have been combined in the cross-section. The last mentioned is frequently employed in practise and it could be preferable to standardise this set of strength classes.

Another possibility would be to have more classes so that homogeneous and combined grades could be included in the same table of classes. But this solution would give some problem with the setting of the strength values for axial tension and compression. These would have to be set at a rather low level in order to cater for the combined grade glulam with the low quality inner laminations. This would penalise homogeneous glulam.

Introduction
A strength class system enables combinations of grade and species having similar strength properties to be classified together with a common set of strength properties. Such a system simplifies the process of marketing structural timber by reducing the number of options at the specification/supply interface.

24-12-1 F Colling, J Ehlbeck, R Görlacher
Contribution to the determination of the bending strength of glulam beams

Introduction
This paper intends to summarize the results of the extensive research work done in Karlsruhe (Germany) on the bending strength of glulam beams. Above all it is the aim of this essay to develop design rules for glulam beams under bending.

The bending strength of glulam beams is primarily governed by two properties:
– the quality of the laminations used;
– the strength of the finger joints

This is repeatedly demonstrated and proved by numerous tests in different countries. In order to make the problem easier to understand how the bending strength of glulam beams is influenced by mixing these two properties, it seems useful to consider first of all both effects separately.

Conclusion
It can be summarized that for aiming at certain glulam strength classes the minimum requirements for the laminations and the finger-joints, as given in Table 8, may be used.

Table 8: Requirements to comply with some glulam strength classes

Glulam strength class	LH 25	LH 30	LH 35	LH 40
Strength class of the laminations	C 18	C 24	C 30	C 37
Requirements due to finger joints: $\rho >$ MOE >	 none none	 390 9000	 440 12000	 510 15000

ρ = mean density in kg/m^3 (moisture content u = 12%). MOE = mean lengthwise MOE in N/mm^2

25-12-1 E Gehri
Determination of characteristic bending values of glued laminated timber. EN-approach and reality

Introduction •
The loading models for bending of glulam are defined in EN TC 124.207. This model is based on a proposal of Riberholt/Ehlbeck/Fewell [1]. The characteristic bending moment of the beam is derived as:

$$f_{m,k,g} = (2.7 - 0.04 f_{t,0,k,1}) f_{t,0,k,1}$$

$$\frac{f_{m,k,g}}{f_{t,0,k,1}} = k_{lam}$$

for a beam with a reference depth of h = 300 mm. Moreover, it is established that:
the mechanical properties of glued laminated timber are derived by multiplying the laminate properties by a k_{lam} factor which make allowance for:

- a the test methods used for single laminations, which may be less constrained than when laminations are bonded together in a single member;
- b differences due to the dispersion of laminate low strength and low stiffness areas throughout the volume of a glued laminated member;
- c differences between the coefficients of variation of single laminates and laminated members.

The above statements are valid only for wood laminations without end joints. If end joints are present the bending strength for the splice must fulfill the following conditions:

$$f_{m,k,j} > f_{m,k,g}$$

The latter condition was adopted on the basis of a proposition of Riberholt. The more comprehensive and better founded investigations of Ehlbeck / Colling lead to more severe conditions concerning the splice strength. For the reference depth of h = 300 mm, the required condition is:

$$f_{m,k,j} > 1{,}15 f_{m,k,g}$$

The coefficient 1.15 is, in my opinion, a value that is too low. Based on the work of Colling / Ehlbeck / Görlacher a coefficient of 1.4 may be justified as will be explaned later.

Recommendations
The requirements in EN TC124.207 must be changed. It is recommended that:

a - Requirements concerning end joint strength (for beam depth h = 600 mm)
based on tensile strength: $f_{t,k,j} \geq 0.85 f_{m,k,g}$
based on bending strength: $f_{m,k,j} \geq 1.4 f_{m,k,g}$

b - Requirements concerning wood strength (for beam depth h = 600 mm) based on tensile strength: $f_{m,k,g} = 12 + f_{t,0,k,l}$ N/mm²

26-12-1 E Aasheim, K Solli
Norwegian bending tests with glued laminated beams-comparative calculations with the "Karlsruhe Calculation Model"

Introduction
In 1990 and 1991 extensive and systematic studies on the strength of glued laminated beams (glulam beams) have been carried out at the "Norwegian Institute of Wood Technology" in Oslo/Norway (Falk, Solli, Aas-heim 1992). It was aimed to obtain given strength values by variation of the properties of the laminations (density and modulus of elasticity).

The investigations described in this paper were performed to estimate and to predict the bending strengths of these glulam test beams with the "Karlsruhe calculation model" (Colling, Ehlbeck, Görlacher). The calculations were based on the informations made available and described in section 2. The test results (bending strength and modulus of elasticity) obtained in Oslo were unknown before finalizing the calculations and publishing the results.

Altogether, three different combinations of different built-up have been studied. In all cases the beam depth was 300 mm with nine laminations of 33,3 mm nominal thickness. The three beam combinations are shown in Fig. 1. The test set-up is illustrated in Fig. 2.

Summary
In 1990 and 1991 extensive and systematic studies on the strength of glued laminated beams have been carried out at the "Norwegian Institute of Wood Technology" in Oslo. For this purpose the mechanical properties of

the laminations were determined and classified according to CEN draft standards of that time (C30-12E and C37-14E). Glued laminated test beams following different strength classes (LH 35, LH40 and LC38) were constructed and tested in strength and stiffness.

At the same time strength and stiffness values of these beams were calculated independently by means of the Karlsruhe calculation model. Information about the properties of the laminations i.e. the statistical distribution of density and modulus of elasticity of the boards used for the three different combinations, about the finger-joints, and about the built-up of the beams were known from the Norwegian pre-tests. The results of the beam tests were kept secret until the predictive calculations were available.

The strength of all beam combinations (mean value and 5-percentile) were proved to be in very good agreement with the calculated values (within 4 % deviation). By this study it became once more evident that the Karlsruhe calculation model is suitable to predict the strength of glued laminated beams. It is on that account an appropriate aid for the evaluation of standards on strength classes for glulam based on the relevant properties assigned to the laminations and the finger-joints.

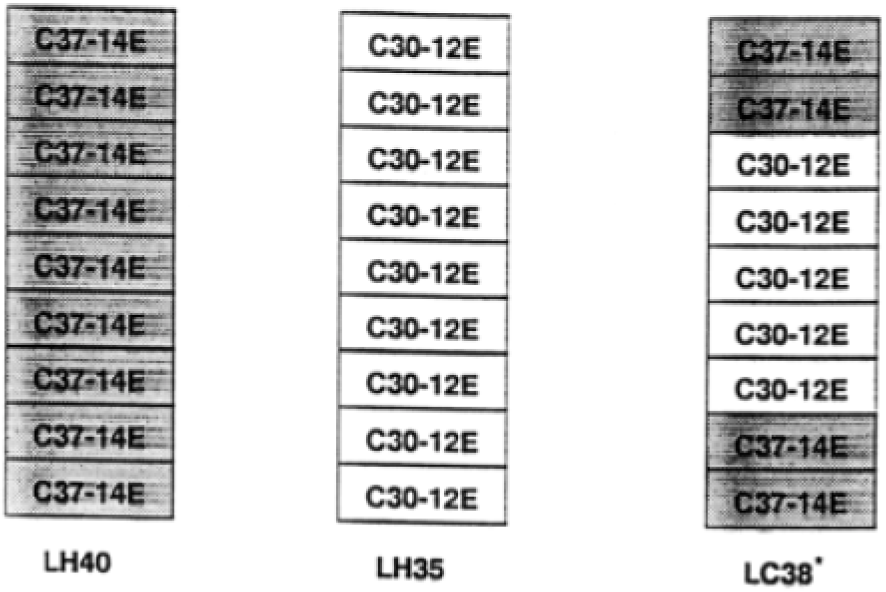


Fig. 1 Beam combinations tested.

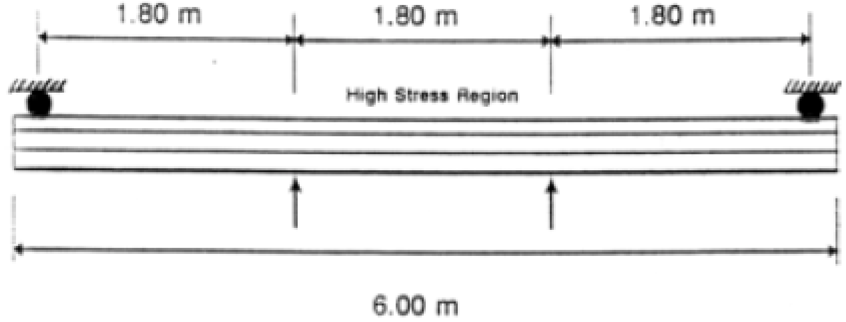


Fig. 2 Beam test configuration.

26-12-2 R Hernandez, R H Falk
Simulation analysis of Norwegian spruce glued-laminated timber

Abstract
A computer analysis model, referred to as PROLAM, was used to simulate the performance of glued-laminated (glulam) timber beams manufactured from Norwegian spruce lumber. Mechanical properties of tested lumber and finger joints were analyzed to determine the input properties required by the model, and Monte Carlo simulation procedures were used to compile and characterize bending strength and stiffness distributions of the glulam beams. Simulated glulam beam results compared reasonably well with actual results. Sensitivity analyses were also conducted to observe both the effects of redistribution of stresses within a glulam beam, and the influence of finger-joint tensile strength on glulam beam bending strength.

Conclusion
In summary, the lumber and finger-joint data from Falk et al. (1992) were analyzed to develop input properties required by a glulam beam simulation model developed by Hernandez et al. (1991). When the input lamination property values were used to simulate glulam beam performance, simulated results compared well with the test results.

Sensitivity analyses indicated that when the tensile strength of all the laminations were held constant and only the core laminations were reduced in stiffness, the decrease in bending strength was less than 4 percent. However, when the same stiffness configurations were modeled while considering the reduction in lamination tensile strength corresponding to the reduced stiffness of the core laminations, the reduction in glulam bending strength was approximately 15 percent. This implies that the difference between outer and core lamination stiffness be kept to a minimum of 15 percent to minimize the reduction in glulam bending strength to within 10 percent.

The second sensitivity analysis involved studying the influence of finger-joint tensile strength on the performance of the glulam beams in this study. Comparing simulated results without the influence of finger joints to simulated results with the influence of finger joints were only within 4 percent at both the 50th and 5th percentiles of glulam MOR. This suggested that for the glulam layups evaluated in this study, finger joints played a marginal role in the overall bending strength performance of the beams. This observation was supported by the actual results of the tested glulam combinations.

26-12-3 F Colling, R H Falk
Investigation of laminating effects in glued-laminated timber

Abstract
In this study, existing lamination and beam test results were analytically reviewed in an attempt to quantify the laminating effect for glued-laminated (glulam) timber. Laminating effect is defined as the increase in strength of lumber laminations when bonded in a glulam beam compared to their strength when tested by standard test procedures. In this study, fundamental concepts are presented to describe the laminating effect, estimates are made of the various physical factors that make up the effect, and a relationship is presented to quantify the magnitude of the effect.

Concluding Remarks
Although the analysis of research data showed a great deal of variability in measures of laminating factors, the following qualitative tendencies were apparent:

– Lamination effects were more pronounced at the characteristic strength level than at the mean strength level. This may be explained by the higher coefficient of variation of the lamination tensile strength compared with glulam bending strength data.
– Lamination effects decreased with increasing quality and strength of the laminations. This may be explained by a lower reinforcement effect (caused by smaller knots) and less influence of testing procedure (caused by more homogeneous material properties in a higher grade).
– Factors that contribute to the lamination effect were interrelated, making it difficult to accurately quantify them separately.

26-12-4 F Rouger
Comparing design results for glulam beams according to Eurocode 5 and to the French working stress design code (cb71)

Abstract
The publication of the EN TC124-207 standard draft about strength classes of glued-laminated timber gave rise to many questions from glued laminated timber design offices as from industrials. Those questions were especially about the characteristic values of the glued laminated timber issued from that of solid wood. However, we have to bear in mind that these characteristic strengths are used in the design methods via partial security coefficients defined as well as in the Eurocode 5 as in the CB 71 regulation. In order to have a significant idea of the changes due to the European regulation, it is necessary to compare practical cases of design. The aim of this paper is to explain Eurocode 5 options and to compare them to CB 71 regulation, and also to quantify the differences between those two regulations. An example of bending under self weight + snow load will be studied. The case of solid wood will also be investigated as a bass to that of glued-laminated timber.

Conclusion
In the case of solid wood, Eurocode 5 design results are between CB 71 normal actions and extreme actions design results, which is in agreement with soft calibration concepts. In the case of glulam, Eurocode 5 design results are always more penalizing than CB 71 design results. For this reason, the French glulam industry is concerned about its future and raised the question during the CEN 124 meetings. A possible answer could be to enable the use of Annex A as a normative Annex and to increase the values by 15%, which would place the glulam in the same position as solid wood. Of course, this would imply an increase of quality control in the industry. This option was already discussed in previous CEN and CIB meetings. It needs to be a compromise between a scientific reality and an economic reality.

27-12-3 E Serrano, H J Larsen
Influence of weak zones on stress distribution in glulam beams

Background

When analyzing a beam with non-homogenous cross-section, a commonly made assumption is that, plane sections normal to the beam axis remain plane and normal when the beam is loaded. This assumption according to traditional beam-theory leads to the well known fact, of piecewise linear stress distribution over the cross-section of a glulam beam of lamellas with different modulus of elasticity. A zone with lower stiffness would, according to these assumptions, be subjected to stresses corresponding to the reduction of stiffness. It is often claimed that a, "weak" zone (i.e. a knot or finger joint) with lower stiffness and strength than adjacent material would be subjected to stresses of less magnitude and therefore not have such severe effect, on global beam strength. The present analysis shows that a low-stiffness zone not, necessarily is relaxed in the above described way.

Conclusions

The analysis suggests that the simple assumption that a local and proportional reduction of stiffness and strength has minor influence on beam strength is not valid for small zones, i.e. knots and finger joints. The stress reduction in a small zone is far from proportional to the stiffness reduction and therefore the stress is closer to the strength of the material in a small weak zone, than one would expect by intuition.

28-12-1 E Gehri
Determination of characteristic bending strength of glued laminated timber

Introduction

The situation up to 1992 has been described by Gehri in **Paper 25-12-1** where also the requirements for the finger-joint based on the tensile strength are given. Meanwhile new research by Falk / Solli / Aasheim (1992) and Coiling (1994) allow a better understanding of the behaviour of glued laminated beams in reference to the lamination strength. The last draft of prEN 1194 (September 1994) takes partly in account the new findings.

It has to be recalled here that the strength model for bending is now based on a depth of h = 600 mm and a beam length of 16 to 18 times the depth.

The width of the beam had not been considered in the bending strength model till now, although in all "European" strength models the tensile strength of the laminations has to be referred to a width of t = 150 mm, using adjusting factor of $(150/b)^{0,2}$. This seems not logical.

The strength model used in the USA takes in account for both, for tensile strength of the lamination and for bending strength of the glularn beam, a width factor (see AITC 117 -93: Design Standard Specifications for Structural Glued Laminated Timber). This has to be reminded when analysing and comparing the results of different research works.

Conclusions

Based on the approach given in section 4, the requirement in prEN 1194 should be changed. It is recommended the following:

(a) Requirements concerning wood strength (for homogeneous sections and depth h = 600 mm) based on tensile strength of lamination:
$f_{m,g,k} = 4 + 0.75\, f_{t,0,l,50}$ N/mm^2
for machine graded laminations with coefficient of variation for the tensile strength smaller than 0.3.

(b) Requirements concerning end joint strength (for homogeneous sections and depth h = 600 mm) based on tensile strength of finger-joint:
$f_{m,g,k} = 1.18\, f_{t,j,k}$ N/mm^2

The above requirements are valid for beam and lamination width of b = 150 mm. For other widths use an adjusting factor of $(150/b)^{0\text{-}1}$.

For combined sections, with outer laminations of depth h/6 and modulus of elasticity E_2 , the following adjusting factor has to be used on both requirements:

$$\left[1-\left(\frac{2}{3}\right)^3\left(1-\frac{E_2}{E_1}\right)\right]$$

29-12-1 G Schickhofer
Development of efficient glued laminated timber

Abstract
Within most of the European countries the grading of timber as well as its classification results from visual obvious characteristics of quality. The inaccuracy of such visual grading, as far as strength and stiffness of timber are concerned, causes an insufficient utilisation of the material timber itself. Special mechanical grading methods working by means of accurate grading parameters to determine stiffness are a basic requirement to classify the respective strength of boards used for the production of Glulam timber. We give you a survey of the results from mechanical grading relating to the particular grading classes and point out which characteristics constitute mechanically graded boards. Finally we define the efficiency of mechanically graded boards within the field of Glulam members, considering also the quality of finger joints respectively interdependent grading scales and qualities of products and their effects on Glulam timber quality.

34-12-1 B Yeh, T G Williamson
High-strength I-joist compatible glulam manufactured with LVL tension laminations

Abstract
In recent years, the growing popularity of I-joists in residential construction has spawned strong demands for high-strength structural glued laminated timber (glulam) with I-joist compatible (IJC) depths in North America. Using the model prescribed in ASTM D3737, Standard practice for establishing stresses for structural glued laminated timber, APA -The Engineered Wood Association has developed glulam layup combinations using full-length (without end joints) laminated veneer lumber (LVL) as tension laminations to satisfy the market needs. These high-strength IJC glulam products have a characteristic flexural strength ($f_{m,g,k}$) of 43 MPa (6300 psi) and a mean modulus of elasticity ($E_{0,g,mean}$) of 14.5 GPa (2.1 x 10^6 psi), which represent the highest performance level that has ever achieved by the commodity glulam used in North America.

This paper describes the details of the lay-up combinations and the results of full-scale glulam beam confirmation tests. For quality assurance purposes, the required control values for the LVL tension laminations are established and reported. These lay-up combinations are being recognized by the evaluation service agencies of the major building codes in the United States.

Results obtained from this study suggested that relationship between the characteristic tensile strength of the LVL tension laminations ($f_{t,0,l,k}$ and the characteristic flexural strength of the glulam beams ($f_{m,g,k}$) is likely to depend upon not only the LVL, but also the glulam manufacturers. It was noticed that the relationship between $f_{t,0,l,\mathrm{k}}$ and $f_{m,g,k}$ did not necessarily follow the American National Standards Institute (ANSI) A190.1, American National Standard for Wood Products – Structural Glued Laminated Timber. Therefore, the required $f_{t,0,l,}$ value for QA purposes should be confirmed by LVL tension and full-scale glulam beam tests. Without the confirmation data, the $f_{t,0,l,\mathrm{k}}$ should be assigned the same value as $\mathrm{f_{m,g,k}}$.

Conclusions
The following conclusions are supported by the results obtained from this study:
- LVL could be used as tension laminations for high-strength glulams up to, but not limited to, an $f_{m,g,k}$ value of 43 MPa (6300 psi) and $E_{0,\mathbf{g},\mathrm{mean}}$ value of 14.5 GPa (2.1 x 10^6 psi).
- The lay-up combinations could meet the values given above provided that the LVL tension laminations are properly quality-controlled.
- It is also important to recognize that relationship between $f_{t,0,l,k}$ and $f_{m,g,k}$ is likely to depend upon not only the LVL, but also the glulam manufacturers. Therefore, the required $f_{t,0,l,k}$ value for QA purposes should be confirmed by LVL tension and full-scale glulam beam tests. Without the confirmation data, $f_{t,0,l,k}$ should be assigned the same value as $f_{\mathrm{m,g,k}}$

34-12-2 B Yeh, T G Williamson
Evaluation of glulam shear strength using a full-size four-point test method

Abstract
The shear strengths of structural glued laminated timber (glulam) have been traditionally evaluated in the United States based on the procedures set forth in ASTM Standards D3737 and D2555 using small block shear values of clear wood specimens. For most glulam products, the design shear stresses so derived are conservative. In recent years, the demand to optimize the design shear stress has been increased due to a higher design

shear stress offered by competing structural wood composites, such as laminated veneer lumber (LVL) and parallel strand lumber (PSL).

Since 1997, APA has conducted a series of full-size shear tests on glulam manufactured with Douglas fir, Southern pine, and Spruce-Pine-Fir. A four-point load method with a clear distance between the edge of the reaction bearing plate to the edge of the nearest curved load bearing block of at least 2 times the specimen depth was used to test all specimens. Based on this experience, the full-size shear test method has been adopted in ASTM D3737 as a standard test method for determining the horizontal shear strength of glulam. This paper provides detailed descriptions of the test methods, experimental results, and data analyses.

The test results obtained from this study indicate that the characteristic shear strength values based on full-size shear tests are approximately 70% of the values determined from small block shear tests. However, the allowable horizontal shear stress could be increased by a factor of at least L25, including a 10% reduction to allow for occasional seasoning checks. This increase can be attributed in part to the difference in the procedures used to derive the design value between the full-size and small block shear tests.

Limitations on use of results

It is very important to realize that these new allowable shear values obtained from this study are intended to be limited to prismatic glulam members subjected to typical dead, live, snow, wind, and earthquake loads only. The allowable shear stresses for impact or cyclic loading, such as may occur in bridges or crane rail applications, have not been evaluated. Neither have the effects of these higher shear stresses been accounted for in the design of non-prismatic members which are typically subjected to an interaction of shear stresses with other stresses. For these applications, the previously published shear values, which have been proven adequate through years of experience, should be retained for design use.

Conclusions

The following conclusions are based on the results obtained from this study:

- The setup used in this study can be used to evaluate the shear strength of full-size glulam. ASTM D3737 adopted this test method in October 2000 as an alternative standard test method for determining the horizontal shear strength of glulam. Since then, this test method has been used by other researchers in the United States for evaluating glulam shear strength of ponderosa pine with equally satisfactory results.
- The width effect on the characteristic shear strength was determined to be negligible
  for DF and SP glulam members and assumed to be negligible for all other species.
- The characteristic shear strength and allowable shear stress for the species other than DF, SP, and SPF can be established by multiplying the small block shear value by the procedural calibration factor of 0.70 and 1.25, respectively. The allowable shear stress derived using this procedural calibration factor includes a 10% allowance for checking.
- Results obtained from this study have been accepted by the major building code agencies in the U.S. and the new allowable shear values are being used by the wood engineering community.

34-12-6 G Schickhofer
Determination of shear strength values for GLT using visual and machine graded spruce laminations

Abstract

The aim of this research project was to conduct tests to determine load-based shear strength values for GLT components subject to bending load. In order to achieve this objective it was necessary to develop a suitable test set-up for glued laminated timber. Although extensive research of the literature revealed that such tests have been conducted for solid timber, the lack of standardised test set-ups is equally a problem in this area. Recent analyses have been performed by B. Yeh, T.G. Williamson and G. Schickhofer and B. Obermayr on glued laminated timber. On the basis of preliminary tests on 24 glulam beams (I-cross section, rectangular cross section with reinforced edge zones, h = 320 mm, h = 608 mm) and finite element analyses aimed to optimise the load introduction area and the cross-sectional form, further tests were based on an I-cross section. Visually and machine graded laminations were used to build the cross-section of the test pieces (S10, S13, MS10, MS13 and MS17 in accordance with ON DIN 4074). A total of 75 test glulam pieces were analysed (5 series with different glulam strength classes) taking into account the optimised three-point loading test set-up with central load introduction (single-span, three-point loading test setup), 5 pieces taking into account an overlap on both sides of

500 mm, and 10 pieces taking into account a two-span, five-point loading test setup. With the aid of the results it was not possible to confirm the formal correlation between shear strength of glued laminated timber $f_{v,g}$ and the tensile strength of the laminations $f_{t,0,l}$ in accordance with EN 1194/1999. Rather, in contrast to the normative rising shear strength values with increasing glulam strength classes, a reduction in shear strengths was displayed. It would seem appropriate to discuss the EN correlation.

36-12-1 H Bier
Problems with shear and bearing strength of LVL in highly loaded structures

Abstract
In most timber engineering design, neither shear nor bearing strength governs the design of bending elements such as beams and joists. However, when the structure carries very high loads over short spans, code values for shear begin to limit the design. Field experience suggests that for these structures, characteristic stresses based on codified test methods and evaluation of bearing and shear are simply, not realistic. The low variability of LVL means these effects can be studied in detail.

A test programme was undertaken to evaluate shear strength of LVL in single span and two span tests, on beams with and without bolt holes. As expected, shear failures were only observed in the very strongest LVL. The very high deformations observed showed that for concrete formwork elements, bearing deformation in the LVL support structure could become critical before shear, and standard engineering shear formulae may give misleading results.

An alternative bearing test was developed and used to assess bearing for beams where deformation control is critical. This test is based on identifying the onset of visible permanent set after repetitive loading, and limiting the design bearing stress accordingly. The information can be used to assist in the preparation of span tables for radiata pine LVL used to support concrete in formwork structures, and the concept could be used for other species.

Conclusions
Shear
The shear strength of LVL can be determined for the very highest grades from a single short span bending shear test. Many specimens will not fail in shear, but a sufficient number can be obtained for derivation of a characteristic value.

For lower grades of LVL, bending failures are more likely to occur, because the material has insufficient bending capacity to mobilise a shear failure. Use the high grade strength.

A two span bending test programme showed that large deformations rendered standard formulae invalid and bearing performance could govern formwork design.

Bearing
Very limited data suggest that it may be possible to derive a more rigorous value for the onset of permanent bearing deformation to cover a range of typical bearing widths used in concrete formwork practice.

Currently used stress of 12 MPa for bearing in formwork is very close to the values obtained for a 50 mm wide bearing.

The total deformation of bearing members is significant relative to the 2 mm and 3 mm limits imposed in the design of formwork structures. Total deformation includes surface indentation and compression through the depth of the beam.

Further study and cyclic testing of formwork models is recommended to ensure serviceable formwork components in engineered wood.

39-12-3 B Yeh, T G Williamson, Z A Martin
Effect of checking and non-glued edge joints on the shear strength of structural glued laminated timber beams

Abstract
Shear strength of structural glued laminated timber (glulam) beams may be affected by the in-service conditions or manufacturing processes. While glulam is typically manufactured with kiln-dry lumber and therefore less susceptible to checking and splitting, glulam beams still check or split, usually at the first or second glue-line and at the beam ends, as they gain or lose moisture in response to direct exposure to water, changing relative humidity and temperature in the surrounding environment.

Literature is available for determining the effect of checks or splits on the horizontal shear strength of glulam beams based on conservative assumptions. It is not uncommon for the architect, builder or homeowner to

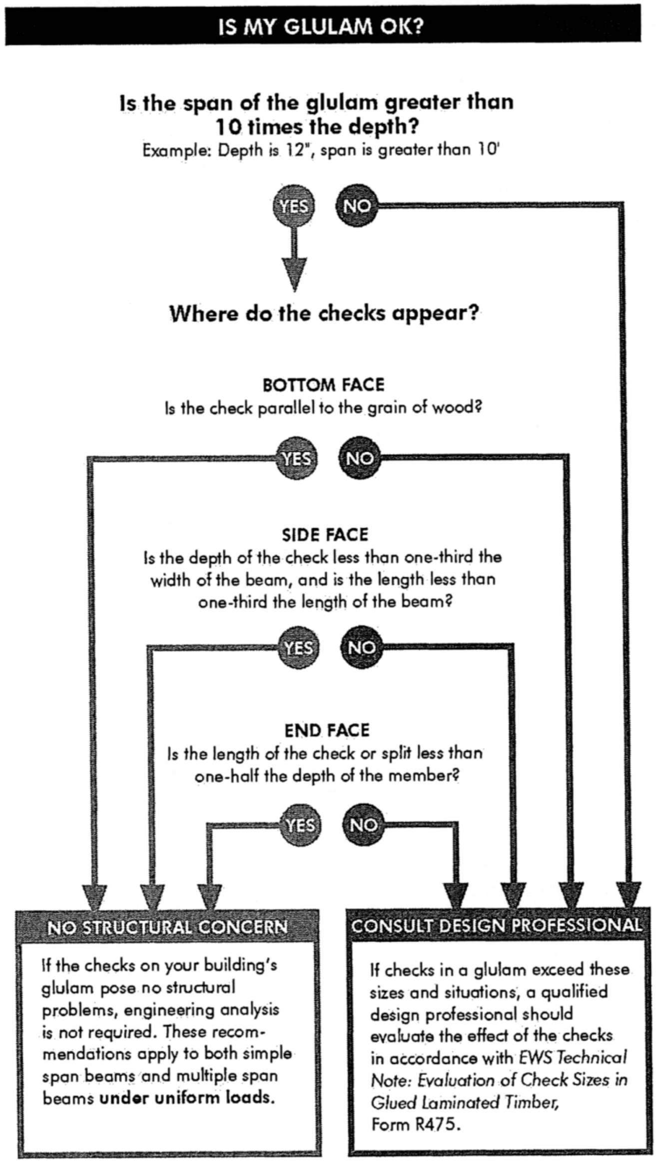
be alarmed when a significant check or split is found on a glulam beam. In many instances, however, the check or split may have limited influence on the horizontal shear strength of a glulam beam and the structural integrity of the glulam beam is not compromised. In order to define the boundaries of checks or splits, upon which the influence of such checks or splits can be safely ignored, APA - The Engineered Wood Association (APA) conducted a series of full-scale glulam beam tests based on the most common configurations and locations of checks or splits.

The glulam manufacturing processes may also affect the horizontal shear strength of a glulam beam. For example, the U.S. design code reduces the horizontal shear strength of a glulam beam when manufactured with non-glued edge joints using multiple pieces of side-by-side lumber and loaded in the direction parallel to the wide face of the laminations (*y-y* axis). The shear strength reduction is not required when such a glulam beam is loaded in the direction perpendicular to the wide face of the laminations (*x-x* axis). However, there is only limited data available to substantiate these cases. In support of a revision to the Japanese Agricultural Standard (JAS) for Structural Glued Laminated Timber, APA conducted a series of full-scale glulam beam tests to evaluate the effect of non-glued edge joints in multiple-piece layups on the horizontal shear strength of glulam beams. This paper describes the test results and findings from the checking and non-glued edge joint studies.

Conclusion

Based on the test results, the presence of unbonded areas between laminations, as could be caused by seasoning checks, does not affect the performance of glulam beams. A new publication on this subject, Owner's Guide to Understanding Checks in Glued Laminated Timber was developed and released by APA in March 2006. Figure 9 shows a flow chart included in the Guide, which provides simple guidance for evaluating glulam checks. Note that an engineering analysis is still required when a check exceeds the limitations specified in the Guide.

For the non-glued edge joints, the glulam shear strength is not affected by the non-glued edge joints in the x-x orientation based on the test results reported above. The effect of non-glued edge joints on the glulam shear strength in the y-y orientation can be conservatively estimated in accordance with the methodology currently adopted by the glulam industry in the U.S. These results have been used to support the revision of the JAS glulam standard to permit the use of non-edge glued joints in the core and inner laminations of JAS glulam beams. Further research on a more realistic strength reduction factor due to non-glued edge joints is recommended.



40-12-1 F Lam, N Mohadevan
Development of new constructions of glulam beams in Canada

Abstract
In Canada the manufacturing of Glulam beams and the grading rules of laminating stock are stipulated under Canadian Standards CSA 0122. This paper reports on procedures undertaken to develop new constructions of Canadian Glulam beams that can fit well with the characteristics of the wood resource. Detailed study of the lam-stock was first conducted via a series of grading analysis and testing. The lam-stock was testing non-destructively for modulus of elasticity and then destructively for tension strength. Grading rules were modified to establish new grades of lam-stock. Subsequently, a detailed knot database was developed to study the performance of these grades based on the US-GAP program. Tension testing was also conducted to establish the strength of finger joints for various new grades of lam-stock. A stochastic finite element program was used to evaluate different prototype construction of glulam beams. An experimental program was then conducted to test full size glulam beams in bending and shear to verify program predictions including size effects issues. Excellent agreement between program predictions and test results was observed.

Conclusions
The characteristics of Douglas fir lamina were studied based on visual grading, defect mapping, MOE and tensile testing, and finger joint testing. Based on these investigations an optimized set of five Douglas fir lamina grades T1, Cc, B, C and D were developed. Their strength properties assessed and the grade turnout was established.

Various combination of the beam lay-ups built with the new lamina grades was analyzed using the computer models ULAG and GAP to develop new lay up of 24*f* Dogulas fir glulam beams. ULAG tend to give more conservative results compared to GAP. Subsequently the performance of the new beam construction was demonstrated through full size testing of two sets of 0.30 in deep and 0.61 m deep glulam beam in bending. The target specified strength and MOE requirements for the 24*f* glulam beams were confirmed and excellent accuracy of ULAG in simulating the flexural strength distribution of glulam was demonstrated. The enhanced ULAG also predicts the shear capacities of glulam with good agreement explaining the observed influenced of stress volume on the shear strength of glulam.

The procedures established from this study demonstrate a new method for glulam beam lay-up design and assessment by using ULAG to predict the flexural capacity of Glulam beams as well as using the tensile strength and the corresponding MOE values of the lamina and the tensile strength of the finger joints as input.

40-12-2 R Brandner, E Gehri, T Bogensperger, G Schickhofer
Determination of modulus of shear and elasticity of glued laminated timber and related examination

Abstract
This paper deals with the determination of the modulus of elasticity of boards, finger joints in tension and hence builds up glued laminated timber (GLT). Emphasize has been taken on determination of G-modulus of GLT by execution of both standardized test methods ace. EN 408, by torsion tests and application of shear-fields in constant transverse force areas during destructive four-point bending tests ace. EN 408. Comprehensive evaluations of relationships between G- and E-module and related mechanical properties have been carried out. Furthermore product of (E · G) has been examined as relevant value for lateral torsional buckling based on test results and simulations. A proposal for further standardization for determination of G-modulus of GLT according to EN 408 will be presented. Simulations evaluate the influence of selected parameters on expectable G-modulus. The data states as bases for further regulations concerning lateral torsional buckling of GLT in dependence of size in comparison to solid wood beams and should be considered for standardization.

Conclusion
Emphasize of presented research project has been to examine various static test configurations for the determination of G of glued laminated timber GLT GL24h. Both test configurations given in EN 408 – 'single span method' and 'variable span method' – have been applied and compared to torsion tests and GSF derived from measurements of shear deflections in 'shear fields'. As discussed, both methods of EN 408 cannot be proposed for further application due to lack of robustness and by consideration of economic aspects.

Measurement of shear deflections during standardized 4p-bending tests acc. EN 408 enables enlarged knowledge of tested beam and may reflect robustness in case of consideration of constraints concerning size and material inhomogeneities. The torsion method, in contrast, enables simple, robust and cost efficient determination of G-values, relevant for stability considerations. Both methods torsion test and measurement of shear deflections in shear fields are proposed for implementation in EN 408, torsion in regard to ASTM D 198 and shear field method in regard to standard 4p.-bending test of EN 408 by consideration of discussed constraints!

Additional accomplished simulations of *E, G* and (*E·G*) for the design process concerning lateral torsional buckling reflect perfect agreement with data of practical tests and enables an advanced further regulation and derivation of $E_{0,g,05}$ and $G_{g,05}$ in EN 1194 (see Tab. 10).

Tab. 10: Proposed models for $E_{0,g,mean}$, $E_{0,g,05}$, $G_{g,mean}$ and $G_{g,05}$ for the regulation of glued laminated timber in EN 1194

Modulus of elasticity	$E_{t,0,g,mean}$	$= E_{t,0,l,mean}$
	$E_{t,0,g,05}$	$= E_{0,g,mean} \cdot \min\begin{cases} \frac{1}{60}\cdot(n-1)+0.67 \\ 0.9 \end{cases}$
Shear modulus	$G_{g,mean}$	$= 650\ \text{N/mm}^2$
	$G_{g,05}$	$= G_{g,mean} \cdot \min\begin{cases} \frac{1}{60}\cdot(n-1)+0.67 \\ 0.9 \end{cases}$

n … quantity of laminations

In regard to $G_{tor,mean} = 620$ N/mm^2 and GSF = 690 N/mm^2, gained from static tests, and by consideration of minor variation of G_{mean} compared to $E_{0,mean}$, (see **Paper 27-10-1**) a constant value of $G_{g,mean} = 650$ N/mm^2 can be proposed for all strength classes of homogeneous and heterogeneous GLT out of softwood. This confirms a proposal of Blass 2005. Based on data gained from simulations a higher $k_{05} = 0.9$ for GLT with $d_{GLT} > 600$ mm and /or $n > 15$ it can be proposed in contrast to current $k_{05} = 0.81$ acc. EN 1194. This leads to an increase of + 11 % of l_{ef} in the design of lateral torsional buckling $\sqrt{E_{0,g,mean} 0{,}81 \cdot I_z G_{g,mean} 0{,}81 \cdot I_{tor}} = 0{,}81 \cdot \sqrt{E_{0,g,mean} I_z G_{g,mean} I_{tor}}$ vers. $\sqrt{E_{0,g,mean} 0{,}09 \cdot I_z G_{g.mean} 0{,}90 \cdot I_{tor}} = 0{,}90 \cdot \sqrt{E_{0,g,mean} I_z G_{g.mean} I_{tor}}$ → **increase of l_{ef} = 11.1 %** and in contrast to structural timber to an increase of +34 % based on $k_{0,5} = 0.67$. For example in case of $d_{GLT} = 300$ mm and $n = 8$, $k_{0,5}$ has to be calculated with $k_{0,5} = 1/60 \cdot (n-1) + 0.67 = 1/60 \cdot (8-1) + 0.67 = 0.78$.

40-12-4 T G Williamson, B Yeh
Standard practice for the derivation of design properties of structural glued laminated timber in the United States

Abstract
Structural glued laminated timber (Glulam) has been used in North America for more than 70 years. The design properties for glulam when manufactured with a recognized manufacturing standard, such as the American National Standard for Structural Glued Laminated Timber, ANSI A190. 1, are typically derived in accordance with ASTM D 3737, Standard Practice for Establishing Allowable Properties for Structural Glued Laminated Timber.

ASTM D 3737 itself is an analytical model based on the "I_K/I_G" model established in 1954 through extensive research conducted by the US Forest Products Laboratory in Madison, Wisconsin. In addition to the bending strength, other design properties of glulam, such as modulus of elasticity, shear, compression parallel to grain, and compression perpendicular to grain, can be calculated using ASTM D 3737 when the glulam layup (grade combination) is defined. As compared to European practice, ASTM D 3737 represents a different perspective in assigning glulam design values, which is well supported by years of practical experience in North America and thousands of confirming full-scale beam tests. One of the differences is the fact that ASTM D 3737 and ANSI A190.1 are based on using the strength reducing characteristics of the laminating grades as the basis for assigning design properties and then requiring the use of an equivalent end joint strength to support these properties.

In addition, the ASTM D07 Committee on Wood is currently balloting a new standard that will permit the establishment of design bending stress and stiffness based on full-scale flexural tests. This paper describes the standard practice for the derivation of design properties of glulam in the United States based on both analytical and empirical approaches. This information provides alternative methods to the existing European practice in assigning glulam design values. Understanding the differences between the European and US practice could help facilitate the development and

harmonization of glulam standards that are being developed in countries such as China and Taiwan as well as for ISO standards currently under development.

Conclusions

With a large number of projects underway in ISO/TC165, Timber Structures, to develop ISO standards for glued laminated timber, it is important that countries involved in the process understand various national standards and how the provisions of those national standards need to be accommodated in the ISO process. This is also true when countries such as China and Taiwan move toward developing their own national standards for products such as glulam and must consider how other national standards have been developed. In virtually all cases some compromises are required by the various countries involved to be able to work towards harmonization of glulam standards through the ISO process while acknowledging national differences.

40-12-6 M Frese, H J Blass
Bending strength of combined beech-spruce glulam

Abstract

This paper is the third part of an easily understandable series dealing with the bending strength of beech glulam published in CIB-W18 since 2005. It describes how to handle combined beech-spruce glulam with bottom and top lamellae of beech and middle lamellae of spruce.

The main topics of the paper are particularly stress distribution in the cross section, characteristic bending strength, MOE, beam lay-up and demands on both the strength grading of the lamellae and the characteristic finger joint bending strength.

In comparison with combined beech glulam it is to be expected that substituting middle spruce lamellae for beech decreases the cost of materials due to the higher price of beech. But the lower MOE of spruce compared to beech has to be considered causing an increase in tensile or compression edge stress, respectively, if the cross section is to be subjected to the same bending moment as a combined beech glulam beam. The use of the calculation model for the characteristic bending strength which is originally related to combined beech glulam does, therefore, necessitate some further considerations to define strength classes for combined beech-spruce glulam.

Conclusions

Substituting beech by spruce in the middle zone of a combined beech glulam beam causes an increase in edge stress in the beech-spruce cross section of less than 3% in comparison with the combined beech cross section. The important thing about it is that the portion of the middle zone must not exceed 60 % of the beam height. By defining marginally higher demands on the characteristic finger joint bending strength than in case of combined beech glulam it is possible to compensate for the decrease of about 3% related to the characteristic bending strength. Hence, the same strength classes are possible as in case of combined beech glulam. Table 5 gives a survey of the results and the demands on both strength grading and finger joints.

Table 5 Strength and stiffness values and requirements; reference beam height 600 mm

	GL28hyb	GL32hyb	GL36hyb	GL40hyb	GL44hyb	GL48hyb
strength values (N/mm^2)						
$f_{m,k}$	28	32	36	40	44	48
stiffness values (N/mm^2)						
E_0,*mean*	13200	13200	13200	14000	14700	14700
$E_{0,05}$	12400	12400	12400	13300	14200	14200
beech: requirements outer zone (>*h*/5)						
DEB[2]	<0,33	<0,20	<0,042	<0,20'	<0,20	<0,042
E_{dyn}	-	-	-	>14000'	>15000	>15000
$f_{t,l,k}$	22	27	32	36	40	48
$f_{m,j,k}$	>48	>54	>59	>64	>71	>72
spruce: requirements middle zone (<3/5·*h*)						
[3]	S10	S10	S10	S10	S10	S10

[1] This grading model was in addition developed after publishing Table 5 in [1]
[2] Quantifies the single knot according to DIN 4074-5
[3] Visual grading according to DIN 4074-1

41-12-2 M Frese, H J Blass
Bending strength of spruce glulam: new models for the characteristic bending strength

Abstract
A comprehensive research project regarding the bending strength of beech glulam showed: Combined visual and mechanical strength grading of boards is very competitive and a strength model, in which the glulam bending strength depends both, on the board tensile strength and the finger joint tensile strength, is a completely transparent model being particularly suitable to determine requirements for the board and for the finger joint tensile strength.

This paper describes the application of these principles to an alternative and new strength model for spruce glulam. For that the effect of the board and finger joint strength on the glulam bending strength is numerically determined by means of simulated glulam beams. According to the findings, described in this paper, current requirements for boards and finger joints are insufficient to ensure the nominal strength values of GL24 to GL36 according to EN 1194.

Conclusions
38 bending tests on full size spruce glulam beams were performed. Since the strength values were obviously too low compared with EN 1194, this investigation was motivated. It was the aim to explain the low bending strength values by developing new strength models for the characteristic glulam bending strength.

By means of a computer model, suitable to simulate different strength grading methods, the mechanical properties of glulam beams were calculated and bending tests on those beams were numerically performed. The simulation results and the board tensile strength, belonging to the grading methods, forma database which was used to perform a regression analysis in order to derive new strength models. Two models, being of importance to calculate the characteristic glulam bending strength, were determined. They are particularly suitable to predict the bending strength of glulam manufactured from visually and mechanically graded boards, respectively.

There is good agreement between the strength models and Colling's results from the 1990s. They can also be verified by bending tests published in the literature. The all-over ratio of predicted values to test results in the literature on average amounts to 1.02. In this ratio 451 bending strength values of glulam are considered.

One can infer from the strength models, that the current requirements for board tensile and/or finger joint strength are not sufficient to ensure the characteristic glulam bending strength values assigned to the strength classes GL24 to GL36. This finding is to be explained for homogeneous GL32: Whereas the current standard EN 1194 stipulates 22 N/mm^2 and 38.8 N/mm^2 for characteristic values of board tensile and finger joint bending strength, respectively, the new strength models lead to demands of about 27 N/mm^2 and 43 N/mm^2 . For combined beams even higher values are required. Against the background of knowledge the poor characteristic bending strength values of the 38 test beams were caused by too low requirements for boards and finger joints.

For further standardization, the paper contains a basic proposal for the characteristic glulam bending strength in the well known format of EN 1194. In addition, it contains an elaborated proposal for prEN 14080 for the chapter "Strength and stiffness properties of glued laminated timber".

43-12-3 M Frese, H J Blass
System effects in glued laminated timber in tension and bending

See 3.14 System effects

43-12-4 C Faye, F Rouger, P Garcia
Experimental investigations on mechanical behaviour of glued solid timber

See 3.14 System effects

2.6 LIMIT STATE DESIGN

19-1-1 R O Foschi, Z C Yao
Duration of load effects and reliability based design (single member)

See 2.7 Load duration

23-1-1 J Kuipers
Some remarks about the safety of timber structures

Preface
The purpose of this paper is to contribute to the discussion on safety, in particular of timber structures. The author is aware of the fact that much more advanced theories were developed with respect to the statistical treatment of loads and strength properties and the probability of failure. The way in which we have to deal with damage due to long duration of load however is less clear. For the design of competitive timber structures a better view on this problem is necessary; may this paper help to the development of such a view.

Introduction
In Heron 5, 1968 a rather simple method for the determination of structural safety was developed and used in particular to try to find out if structures, made of different materials and designed according to then existing standards would show comparable probability of failure.

For timber it was assumed that the effect of duration of load according to Wood, the Madison-curve, could be used in a slightly simplified way and furthermore that every load of a certain duration would cause a strength reduction or damage proportional to the ratio of that load to the one, causing failure after that particular lapse of time.

Although several attempts have been made to demonstrate that the Madison curve has not a universal validity, until now there is not an accepted way to deal with the problem and to combine the different results.

In the following firstly some attention is paid to the interpretation of duration-of-load-effects. Secondly it is tried to give – in the same simplified manner as before, i.e. using normal distributions for loads and for strength – a general reasoning for the determination of the probability of failure if certain values for $\gamma_q \gamma_m k_{mod}$ are prescribed.

Conclusions
- The probability of forces or stresses in a structure to exceed certain long-duration strength can as a first approximation be calculated in the same way as has been done before in Heron 5
- The conditions of Eurocode 5 give β-values of about 2.5 to 3.5, which is of the same magnitude as what was calculated before.
- It seems not true therefore that the safety in timber structures is significant higher than was estimated by earlier calculations.
- These β -values however were calculated at the end of the lifetime of the structure, say after 100 years. The damage due to the load will probably only occur at the end of this period. During far the greatest part of the lifetime of timber structures the reliability is much higher than the existing calculations show.
- The β -values increase – of course – if a lower variability in the strength properties may be assumed;
- For the extreme load combinations of short duration a higher β so a higher safety is found than for the permanent loads.
- For glulam a higher safety is found than for sawn wood, even with the lower γ-value in Eurocode 5. Such a special rule for less variable material seems well justified.
- If the Madison-factor of 0.55 could be increased to e.g. 0.65 the γ-values will be significant higher or, with the same β -values the values of the other variables could be chosen more competitive.

N.B. With other, more sophisticated methods, better estimations of the reliability of structures can be made. It may be expected that higher values of β (3.5 to 4) will be found, using the same Eurocode 5 proposals. It is highly recommended that such reliability studies for timber structures should be made. The way in which the influence of long and short duration of loading has to be taken into account should be given very careful attention.

23-1-2 F Rouger, N Lheritier, P Racher, M Fogli
Reliability of wood structural elements: a probabilistic method to Eurocode 5 calibration

This communication reports some preliminary results of a research program which has been initiated in France on "Reliability of Wood Structures". The Centre Technique du Bois et de I'Ameublement and the University of Clermont-Ferrand have initiated this program for design codes calibrations and structural (systems) design purposes.

The Eurocode 5 requires probabilistic analysis further than simple conversions from working stress design codes. This approach has been already employed for other materials -(steel and concrete) in the European Design Codes but also for wood in North America . The safety levels are usually given for single components. The systems design has been recently more and more investigated.

Based on mechanical models, probabilistic design of wood systems might be of great interest at least for three reasons:

– get some benefit of material variability, at least for redundant structures in which the between members variability gives load sharing effects.
– effects of semi-rigid connections on structural behavior increase the global safety, compared with single components design.
– wood structures have a good behavior under seismic or wind actions. A probabilistic analysis based on stochastic processes should give practical consequences for design.

This paper describes only some results dealing with single components analysis. The systems analysis will be investigated in the next two years.

Abstract.
The first part of this paper describes the reliability theory and methods available to compute probabilities of failure. In order to formulate further design equations, we need to know the distributions for both material behavior and actions.

Random variables have been fitted to wood properties (MOE, MOR ...) using a French Database which covers different species. This database is the result of a large research program whose objective was to qualify French species through their physical and mechanical properties, in relation with the sylvicultural modes. Modelling has been done for different species, different grades and different cross-sections.

The actions have been modelled using information available in the existing Limit State Design Codes (CSA086, CEB ....). Extreme values distributions have been used to model wind, snow, occupancy loads. The permanent loads have been modelled by normal distributions.

The first reliability analysis has been done for the basic case of a solid wood beam in bending. This approach is similar to the other design codes, but changes in climate, materials or return periods might slightly change the partial coefficients. The target safety indexes might also be different with respect to different constructions.

The second reliability analysis reported in this paper focused on bolted joints. A simplified fracture model has been used together with design codes recommendations in order to formulate limit state functions. The influence of humidity was also investigated as a modification factor to partial coefficients.

Conclusion
Single elements calibrations have shown that the current partial coefficients give safety levels comparable to other design codes.

The bending study has shown that the serviceability limit states give much lower safety levels than ultimate limit states. The geometrical uncertainties have a slight influence on the safety levels, which might be omitted in design codes, at least for a given humidity. The bending problem will have to be completed by other simple problems, as shear for example.

For bolted connections, it seems difficult to adopt only one value for γ because of a strong influence of joint geometries. The humidity effect is slightly higher than predicted by the Eurocode 5. The study has been done only for ultimate limit states, based on a theory of perfect plasticity. Further investigation should be done, taking into account elasto-plastic behaviors and looking at the deformations, which probably influence the safety of the connections.

In a near future, these simple investigations will give a basis for systems analysis in which interactions between elements influence the global safety of the structures.

31-1-1 C J Mettem, R Bainbridge, J A Gordon
A limit states design approach to timber framed walls

Abstract
Lightweight platform framed walls have become established on the basis of quasi-empirical design methods, allowing interpolation, but not extrapolation. Both medium-rise building aims, and desired wall construction improvements, place limitations on furthering this approach.

This paper describes investigations carried out under a research and development project, the principal purpose of which is to produce an effi-

cient limit-states design method for timber frame walls that is compatible with EC 5 Part 1-1 & suitable for both low-rise and medium-rise buildings.

Introduction

The subject of the behaviour of timber framed wall panels is not as simple as might at first be imagined, and has a long history of research and development. Although most research has led to valuable information regarding the general behaviour, there is a certain degree of conflict, particularly in relation to test methods. This results in apparent discrepancies in racking capacity when designing to different design codes.

The subject is addressed in Section 5.4.3 of EC5, but this currently provides only one design principle, accompanied by a number of application rules based on a cantilever model. This constitutes only a very minor section of EC5 and is massively disproportionate to the parallel guidance provided in the current UK national code, whereby an entire section of BS 5268 is devoted to the design of domestic type timber frame walls, and a companion section to address non-domestic design, testing, fabrication and erection is under development. Whilst it is recognised that EC5 draws upon other supporting standards in some of these respects, there currently remains an area of strong divergence from the main body of EC5 in the UK NAD.

The continued adoption of the construction methods currently employed in the UK for timber frame walls is supported by their long established history of use, which has led to commercial confidence in the timber frame product. The BS 5268 method is in compliance with EC5 C15.4.3 P(1) and there is no reason why the racking resistance assessment procedure in BS 5268: Part 6.1 cannot form foundation for conversion to limit states, hence enabling full compatibility within the broader scope of the structural Eurocodes. Conversion however is not a straightforward exercise, the largest hurdle being the need to separate ultimate strength considerations from serviceability conditions.

The work reported herein is therefore composed of two key aspects:

a) The development of a limit states based method in the spirit of existing BS 5268 practice
b) Experimental work, focused upon serviceability limit states aspects and gauging the influence of boundary test conditions upon the behaviour pattern of simple wall panels of modern constructional form, thus providing pointers towards translation of component to system behaviour.

32-1-1 H J Larsen, S Svensson, S Thelandersson
Determination of partial coefficients and modification factors

General

In most codes the safety requirements are expressed in symbolic form as

$$S(\gamma_G G_k + \gamma_{Q,1} Q_{k,1} + \gamma_{Q,2} Q_{k,2} \dots) < k_{mod} R_k / \gamma_m \qquad (1)$$

where

S	Action effect
G	Permanent action
Q_1, Q_2	Variable actions
R	Capacity
γ	Partial coefficient (safety element) on G, Q and R
k_{mod}	Factor that takes into account the difference between the resistance in the structure under the actual conditions and the resistance determined in a standard test. For most materials other than wood, k_{mod} is equal to unity. For wood, k_{mod} depends on the moisture and load history.

Index k denotes characteristic value, i.e. a chosen or prescribed fractile in the distribution.

Table 1 - Examples of γ-values.

		Eurocodes[1)]	Nordic recommendations[2)]	New Danish Code[3)]
γ_G		1.35	1.0	1.0
γ_Q	imposed actions [4)]	1.5	1.3	1.3
	natural actions [5)]	1.5	1.3	1.5
γ_m				
	timber	1.3	1.5	1.64
	glulam	1.3	1.35	1.50
$\gamma_Q \gamma_m$	(timber)	1.95	1.95	2.13-2.46

1) [ENV 1991-1, 1994] and [ENV 1995-1-1, 1993]. - 2) e.g. [DS 409, 1982] and [DS 413, 1982]. - 3) [DS 409, 1998] and [DS 413, 1998]. Until the end of 1998 the Danish values were identical to the Nordic ones. - 4) Floor loads etc. - 5) Wind and snow loads etc.

By setting $\gamma_G = \gamma_{Q1} = \gamma_{Q2} = \ldots = 1$, expression (1) corresponds to the permissible stress system, the safety factor being γ_m/k_{mod}. In the now common partial safety system, γ_G is different from at least one γ_{Q1}. It should be noted that a factor can be moved freely from the action side to the resistance side, i.e. the safety level is described by γ_m/γ_G, $\gamma_m/\gamma_{Q,1}$, …

Examples of γ-values are given in table 1.

32-1-2 V Enjily, L Whale
Design by testing of structural timber components

Abstract

This paper is concerned with the conversion of component test results such as trussed rafters, wall panels, ply-web beams, etc. to design values in accordance with ENV 1995-1- 1. Since the publication of ENV 1995-1-1: 1994. the lack of guidance for converting component test results into design values and the importance of number of tests required for practical and economical reasons have been repeatedly highlighted.

Annex A of ENV 1995-1-1: 1994) (although informative) deals with the determination of 5-percentile characteristic values from test results and acceptance criteria for a sample. Although, this part of ENV 1995-1-1 may be adequate for determining the properties of materials or small component tests, such as joints, it is totally unacceptable for component tests by the industry, as the minimum number of tests given in this section of ENV 1995-1-1 is 30. This is not only uneconomical, it is also impractical for component tests such as trussed rafters, wall panels, ply-web beams, glulam beams, etc.

A method is proposed for converting component test results to design values using suggested modification factors and the acceptance of the results for consideration by CIB/W 18, TC250/SC5 and its Project Teams for inclusion in ENV 1995-1-1.

Conclusions and recommendations

- The lack of guidance for converting component test results into design values and the importance of the number of tests required for practical and economical reasons have been highlighted.
- The assessment of the conversion factor from test to design is strongly dependent on the type of test and the type of material. No further guidance is given in ENV 1991-1: 1994. ENV 1991-1: 1994 admits that the guidance given for Design by Testing is incomplete since it must also depend on the type of test and materials involved.
- The current ENV 1995-1-1 and ENV 1991-1 methods of deriving design values by testing is not appropriate for timber structures / components.
- Although, Annex A of ENV 1995-1-1: 1994 may be adequate for determining the properties of materials or small component tests, such as joints, it is totally unacceptable for component tests by the industry, as the minimum number of tests given in this section of ENV 1995-1-1 is 30. This is not only cumbersome and uneconomical; it is also impractical for component tests such as trussed rafters, wall panels, ply-web beams, glulam beams, etc.
- It is recommended that the proposed simplified Design by testing methods for Ultimate and Serviceability limit state designs given in this paper should be adopted for the revision of ENV 1995-1-1: 1994. The modification factors recommended for the number of component tests in this paper are based on the assumption that
  - the coefficient of variation of the components tested is 8% (this is an important factor, as when the coefficient of variation increases, so does floe number of replicates to be tested).
  - experience of behaviour of components in service.
  - proof of their safe use in the past few decades.
- The factor η has three values, one for timber, plywood and glue laminated timber (0.894) and another two for board materials other than plywood related to service classes 1 and 2 (0.93 and 1.182 respectively).
- Tables in Annex includes factors of safety obtained using the proposed method. The factors for permanent loading would not be used in practice as it is unlikely that a structure or component would have a design load consisting of permanent load only (e.g. self weight).

33-1-1 S Svensson, S Thelandersson
Aspects on reliability calibration of safety factors for timber structures

Introduction

From time to time national structural codes are revised. The safety coefficients for actions as well as strengths for different materials are then often calibrated in reliability studies and different structural materials are benchmarked against each other.

In structural reliability analysis an important aspect is how to describe the random variables governing the problem. It is often forgotten that the target safety index β_{target} and the required probability of failure were once determined under certain circumstances. In the Scandinavian countries target safety indices were determined more than 25 years ago by calibration against existing simple structures with proven performance. For the sake of simplicity, it was then assumed that loads were normal distributed and that parameters related to the load carrying capacity were log-normal distributed. However, for time variable actions such as snow and wind, Gumbel or Gamma distributions give better fitting to data. In recent studies it is, therefore, not surprising that evaluation of structural reliability is made using Gumbel distributed variable load, together with the safety index, β_{target} given in building codes. The problem is, however, that the values of β_{target} are based on calibration to earlier proved practice under other assumptions, as mentioned above. A reliability based design will then give larger dimensions or higher required strength for load bearing elements compared to earlier practice. If the evaluations are used for code calibration it will lead to proposals to increase the partial coefficients, γ, to ensure structural reliability. The change of variable load description, from a normal to a Gumbel distribution, with fixed β_{target} affects structures with low portion of permanent load (dead weight) such as timber structures more severely than heavy structures. The reason for this is misinterpretation of safety indices, and failure probability, as absolute values of reliability.

The effects of using Gumbel distributions instead of normal distribution for variable load in code calibration of material partial coefficients for timber will be demonstrated. The calibration of target safety index is repeated as it was done when probabilistic theory was introduced in the building codes, but with new assumptions of load description, and it is shown that the target safety index is not an absolute value but should depend on the assumptions used in the calibration. The direct consequences of this approach for code calibrations related to design of timber structures are illustrated.

Conclusions

The main aim of this study is to demonstrate the consequences of changing the statistical description of random parameters in reliability based code format without considering how the rules in the code once were derived. These consequences were not discovered in this study, they were already pointed out when the concept of invariant safety index was first introduced. Results presented herein, however, are thought to give clear picture of the consequences for timber structures. The study discusses the common issue of changing the statistical description for natural (snow) loads from Normal to Gumbel distribution. Similar consequences will result if the statistical description of any other random parameter is changed. The message of this study can be concluded as:

– Target safety levels, β, presented in codes are not universal values. They will vary depending on the circumstances they were derived.
– Changing the statistical description of a random parameter without considering the origin of the safety levels given in codes will lead to wrong conclusions.
– Light weight structures, such as timber structures, are more affected by a incorrect change of variable load description from normal to Gumbel distribution, i.e. when the change of description is done without considering the origin of the safety levels.
– The definition of characteristic value must be consistent with the statistical description used in the reliability analysis.

33-1-2 A Ranta-Maunus, M Fonselius, J Kurkela, T Toratti Sensitivity studies on the reliability of timber structures

Introduction

Reliability analysis has been used to assess structural safety for about half a century. In the early works the distributions of variables were limited to only Normal distributions to enable closed form solutions. Development of the methods was active in 1970's when modern computers enabled wider range of methods to be used and textbooks were written on the subject. The role of reliability analysis was discussed also in the CIB-W18 meetings. A comprehensive study on the reliability based design of wood structures was published by Foschi (1989).

Numerical methods enable today fairly easy computation of structural safety parameters for comparison or calibration of safety coefficients of different materials and loads. These kind of studies have been recently made related to Eurocode preparations

The failure probability P_f or safety index β, is sensitive to the load and strength distributions selected. Eurocode (Draft prEN 1990, Basis of design, Annex C) gives advice on how to make the structural safety assessment. For instance, extreme distributions have been named for imposed and natural loads and Lognormal or Weibull distributions for material strength. Indicative values are given also for target safety level as shown in Table 1.

Table 1. Target reliability index β for normal structural members (prEN 1990).

Limit state	Target reliability index	
	1 year	50 year
Ultimate	4.7	3.8
Fatigue		1.5 to 3 .8[2)]
Serviceability (irreversible)	2.9	1.5

[2)] Depends on degree of inspectability, reparability and damage tolerance.

The objective of this paper is to analyse, how sensitive the calculated safety parameters are to the selection of the types of distributions and to the values of coefficient of variation COV). Concerning standard distributions, similar studies have been made earlier. Now a numerical procedure is used which allows us to analyse the effect of any numerically given distribution.

Conclusions

These simple calculations indicate that the present level of safety coefficients in Eurocode 5 (γ_m = 1.3 and γ_Q = 1.5 for variable loads: total safety factor γ = 1.95) results in safety level β = 4.2 in case of Gumbel load (COV = 0.4), and (β = 4.8 in case of Normal load (COV 0.4). The result with Gumbel load suggests that instructions given in Eurocode, Basis of Design, Annex C are inconsistent: the use of Gumbel distribution for variable loads together with target safety level of $P_f = 10^{-6}$ or) β = 4.75 leads to requirement of so high safety coefficients which cannot be supported by practical experience. In the history of timber structures such failures can hardly be identified, which should be counteracted by the rise of general safety level in design.

Sensitivity analysis reveals that quite different conclusions can be made depending on the load distribution type used:

1 in case of Gumbel distribution the structural safety seems not to depend much on deviation of strength values, especially if lognormal distribution is used for strength. Also truncation of the lower tail by good quality control gives only a small advantage in terms of safety coefficient.

2 in case of Normal distribution the safety level depends more strongly on COV of material strength: Lognormal distribution with COV = 0.2 needs 16 % higher safety coefficient than material with COV = 0.1 to achieve the same safety. In case of Gumbel load, the same difference is only 1 %.

38-102-1 A Asiz, I Smith
New generation of timber design practices and code provisions linking system and connection design

Introduction

During the last several years the Canadian timber design code committee (Canadian Standard Association 086 Technical Committee) has been laying the groundwork for a major overhaul of the national model code CSA Standard 086-01 'Engineered Design in Wood' (CSA 2001 a). Key issues are:

I. Need for consistency in the objectives, philosophy and technical details that underpin provisions of different sections of the code. For example, if the section dealing with seismic design of braced frames requires use of ductile connections, then the section dealing with detailed design of connections must provide guidance on how to achieve such mechanistic behaviour.

II. Explicit consideration of the relationships between design provision applicable to individual components (members and connections) and the performance of major structural subsystems and complete structural systems. As in other countries, structural codes in Canada are premised on sequential design of individual components. This premise is coupled with an expectation that if every component 'strong and stiff|' the complete structure will have adequate behaviour. Such an approach leads to great uncertainty about system behaviour at failure. Also, although arguably the current approach is conservative, solutions tend to be uneconomic.

III. Development of partial coefficients (load and resistance factors) in design equations that reflect 'true' rather than 'nominal' reliability levels. To date partial coefficients in the Canadian timber design code are calibrated to yield essentially the same solutions as were achieved from past Allowable Stress Design (ASD) codes. Structural reliability methods have been used, especially in connection with setting design resistances for small dimension lumber members. However, as the calibration point is always achieving safety indexes under partial coefficient design that maintain parity with ASD within selected calibration problems, it is simply a very fancy way to do so-called 'soft code conversion'. For genuine progress under item II it is essential to deal with real reliability levels, i.e. not bother about parity with ASD solutions.

IV. Modernise the connection design provisions many of which have their origins in unrecorded committee decisions made in the order of 50 years ago, based on US studies for military purposes at around the time of the Second World War.

V. Rapid integration of new products, typically proprietary, into the market place. This requires the establishment of a framework and methods that ensures consistency in how design properties are assigned to old and new products.

There have and will continue to be also major changes in provisions of the National Building Code of Canada (NRC 2005), i.e. the model document that lays down general - construction type independent - design requirement, and specifies the magnitudes of environmental loads and load factors. The 2005 edition (to be published in September) provides, for the first time, explicit statements of mandatory 'design objectives' and functional statements. The role of the timber code, and other material specific codes, is to provide a basis from which designers can produce solutions that satisfy various objectives listed in the building code. Implicit in items I to V listed above is recognition the potential that changes in the building code releases will only be fully harnessed by the timber construction sector if accompanied by major change in the timber design code.

The last major overhaul of code provisions in Canada that relate to timber connections occurred in the early 1980's. But what was done then was mostly concerned with reformatting, information as the code as a whole was transformed from an ASD to a partial coefficients format. Here the need to revise connection design methods is employed as the tool for tying the sections of the national timber design code together, just as actual connections tie timber structures together. This touches on all of the issues I to V.

Extensive use is made below of working documents produced by the University of New Brunswick's Timber Engineering Group under the leadership of Professor Dr. Ian Smith. Specific consideration is given in those documents, and this one, to promoting design practices and provisions that:

- Make it transparent to designers what mode of failure governs the strength of particular connections, and how that relates to system behaviour.
- Embody probabilistic Load and Resistance Factor Design (LRFD) concepts.
- Guide designers toward an appropriate choice of structural systems to resist given sets of loading combinations.
- Guide designers toward appropriate selection of wood and other structural materials.
- Integrate design provisions specific to connections with those pertaining to the overall system design. In the context of CSA Standard 086-01 this amounts to integration of connection and general design requirements.
- Maximize possibilities for technical harmonization of Canadian and international practice.

What is outlined here is intended to be consistent with activities by other Canadian experts, especially those working on issues related to system behaviour. Although as yet so inconsistencies remain, work reported in the paper "Framework for lateral load design provisions for engineered wood structures in Canada" (Popovski and Karacabeyli 2005) is convergent with what is discussed here. The next three sections discuss the logic for new connection code provisions, general design code provisions that interrelate with design of connections, and current activities.

41-1-1 J Köhler, A Frangi, R Steiger
On the role of stiffness properties for ultimate limit state design of slender columns

Introduction

In the daily practice the engineering codes and regulations form the premises for the use of timber as a structural material. Code regulations in North America, Australia and Europe are based on the limit states design (LSD) approach which is put into practice as load and resistance factor design (LRFD) formats. Initially, LRFD methods where converted as so called "soft conversions" of allowable stress design (ASD), the design method which was commonly used in code regulations before LRFD was introduced and which is usually based to a major part on experience, tradition and judgment. In the last decades this situation has changed; structural reliability concepts have been developed and provide a rational basis for the reliability based calibration of LRFD formats.

Typically, reliability based code calibration takes basis in the assessment of rather simplified design situations, i.e. bending-, tension- or compression components sustaining some typical load combinations. The herewith calibrated partial safety factors are, strictly, only valid for these simple design situations. For the well known reasons of applicability and clarity - beside reliability two major objectives of codes and standards - the application of the same partial safety factors for different design situations is common in present structural design formats.

Strength related timber material properties are generally considered for ultimate limit states, whereas stiffness related timber material properties are of interest when serviceability limit states are considered. Both, strength and stiffness related timber material properties have to be considered for ultimate limit states where the stresses, i.e. the load bearing capacity of the structure, are directly dependent on the deformation of the structure. An example for this is the design of slender columns against axial loading. Within the present paper two European code formats, EN 1995-1-1 and DIN 1052, for the design of slender columns is considered and the role of the timber stiffness property is analysed.

Conclusions

Load and resistance factor design (LRFD) formats comprise simplified limit state design equations together with factored design values for loads and resistances. In general, the factorization takes in so-called characteristic values. Characteristic values correspond to fractile values of the underlying probability distributions of the load and resistance variables. For resistance variables the lower 5 % fractile value is used in general, for load variables the 50 % or the 98 % fractile value is used, whereas variable loads are generally represented as the distribution of their annual extreme value.

Load and resistance factor design (LRFD) formats are also called semi-probabilistic because characteristic values are by definition understood as a simplified representation of the corresponding underlying random phenomena. Load and resistance factors could be understood as 'set screws' that are calibrated to facilitate for consistently safe and efficient design solutions over the wide range of different structural systems to be designed in our build environment.

In Europe the partial safety factor for solid timber resistance $\gamma_M = 1.3$ is such a 'set screw' factoring 5%-fractile values of timber strength related timber material properties. This factor has proven to be efficient for different simple design situations as bending or tension under different combination of loads.

However, for more complex design situations or for different material properties involved in the limit state design equation, it has to be carefully assessed whether the factorisation by γ_m provides safe and efficient design solutions.

In the present paper it has been discussed how γ, is used to factor the modulus of elasticity (MOE) in a 2nd order design formulation for slender axially loaded columns in EN 1995-1-1 (mean value of MOE factored by $1/\gamma_M$) and in DIN 1052 (5 % fractile value of MOE factored by $1/\gamma_M$. The reliability of different design solutions, according to EN 1995-1-1 and to DIN 1052 and for different slenderness, has been assessed. The reliability assessment indicated that design solutions are either too safe (DIN 1052) or show reliability indices which are on the limit of acceptance.

As a fast track solution an alternative factorisation of MOE has been assessed that showed consistent and acceptable reliability indices for both, compact and slender columns. However, this alternative factorisation should not be understood as a solution, but more as an indication for further research and proximate code review.

The significant sensitivity of reliability estimates to the representation of stiffness in the design equation suggests also that the creep effect caused by the load and climate history during the lifetime of a structure is a factor of utmost importance for column design. This aspect, entirely masked out in this study, should be taken into account in further investigations with greatest care.

43-101-1 T Poutanen
Dependent versus independent loads in structural design

Abstract

In current structural codes loads are sometimes assumed correlated and sometimes non-correlated. Occasionally, when the load includes more than two loads, both assumptions may be applied in the same load combination.

Permanent loads G (e.g. permanent loads of multi storey buildings) are always assumed correlated in the serviceability and in the failure design.
A permanent load and a variable load are always assumed non-correlated in the failure design but correlated in the serviceability design,
Variable loads Q are assumed correlated except in the case of two variable loads where a constant combination factor Vo is applied. This denotes an approximate non-correlated combination as the combination factor Vo is variable.
The current hypothesis is that all loads are non-correlated and combined independently. However, the code literature includes no explanation as to why the loads are often assumed correlated.

This paper establishes that two loads may be correlated or non-correlated. The loads may be proportions of a third load and therefore correlated or correlated for some other reason. On the other hand, most loads are non-correlated.

This paper illustrates that loads are combined dependently in structural design if the strength or deflection constraint is considered and independently if these constraints are not considered.

The author has set out in a previous paper [4] how correlated and non-correlated loads are combined independently. However, the conclusions of the paper [4] are wrongly based on an assumption that all loads are correlated.

A method of combining non-correlated and correlated loads dependently has not previously been published. This paper includes results of a Monte Carlo calculation and concludes that the independent combination seems to be a good and safe approximation to combine dependent loads.

Conclusion

The current practice in combining loads is inconsistent as it does not differentiate between correlated and non-correlated loads and assumes all loads to be non-correlated.

Some loads, e.g. the permanent and live loads of multi storey buildings, are correlated and these loads should be combined without a combination factor ψ_0. Live loads are combined currently with a combination factor i.e. wrongly according to this study, but permanent loads are combined without a combination factor i.e. correctly.

Loads may be combined dependently or independently. Independent combination results in a bigger load and therefore this method should be used in structural design.

If the eurocode is changed to apply correlated and independent load combination i.e. $\psi_0 = 1$ and at the same time subsequently changed, load safety factors are deleted, i.e. $\gamma_G = \gamma_Q = 1$, material safety factors are made variable, design point values of variable loads are made variable, the overall accuracy i.e. the maximum excess reliability, would be less than in the current code with less design work.

2.7 LOAD DURATION

6-9-3 T Feldborg, M Johansen
Deflection of trussed rafters under alternating loading during a year

See 3.19 Trussed rafters

19-1-1 R O Foschi, Z C Yao
Duration of load effects and reliability based design (single member)

Introduction
The introduction of reliability-based design in wood structures requires consideration of the effects that load duration has on degradation of strength over time. Normally, when no duration of load effects are present (as in the case of steel structures), the estimation of reliability over the service life requires only probabilistic information on the material's short-term strength and on the maximum load which the structure would receive over the life period considered.

When strength degradation is present, the reliability estimation is complicated by the fact that load history must be taken into account, considering not only load magnitudes but also the duration of each load period. Of course, central to the solution of the problem is the availability of degradation or "duration of load" model which would allow the estimation of the degradation effect for any load sequence. The authors, in a separate paper, discuss different duration of load models, their relative advantages and shortcomings, and their ability to represent experimental trends for constant loads (**Paper 19-9-1**).

A second consideration in reliability estimation is the procedure chosen for the calculations. Simulation is a straightforward procedure which can account for the variability in the material as well as the possible loading sequences. This procedure, however, is normally tedious and expensive. Calculation of reliability indices based on algorithms like Rackwitz-Fiessler's should be preferred as they are much faster and, in general, equally accurate. However, application of such algorithm presents special problems in the case of interaction between strength and load sequence.

Finally, reliability calculations for wood structures must take into account the behaviour of the structural system rather than only that of a single member. In fact, the usually high variability in single member properties is compensated by the action of a redundant system, and consideration of such action is the only way in which a realistic reliability assessment can be made for structural applications of wood.

An example of reliability estimations for timber systems has been presented by Foschi. Nevertheless, many codes are based on "single member" design, with "load sharing" factors applied to take into account the added reliability of a system. In this context, reliability of single members is a first step in the consideration of the more general problem.

The objective of this paper is the discussion of single member reliability in bending, using the Canadian damage accumulation model, and the development of techniques based on the Rackwitz-Fiessler algorithm for the reliability index.

19-6-1 J Kuipers
Effect of age and/or load on timber strength

Introduction
During many years much attention has been paid at many places in the world on the effects of long-duration-loading on timber. Most of the investigations on the subject dealt with the measurement of time-to-failure of test specimens under constant loads of relatively high levels. Such investigations suggested that a long-duration-strength of about 55 % of the short-duration-strength should be used to determine safe working stresses. Such working stresses for permanent loading on structures, built for an intended lifetime of 50 to 100 years turn out to be on a level of, very roughly, 25 % (to 35 % for joints) of the short-duration-strength values. The question arises if such low load levels do have a comparable damaging effect on the initial strength or that they, being so low, do not have any strength decreasing effects at all.

J Vermeyden comes to the conclusion that not much research had been carried out to study the effect of long-duration of loading on the remaining strength of timber structural elements. The available information deals with the effect of repeated loading and of relative short duration of constant loads, sometimes to high load levels, but not with the effect of long-duration-loading to about the allowable levels. Nevertheless Vermeyden comes to the expectation that such preloading very probably will not have a significant strength reducing effect.

In the following some information, available at the Stevin Laboratory, has been put together as a first attempt to come to an answer to this question. The available date was not the result of a systematic approach because the investigations served other goals; nevertheless it is thought that they are of sufficient interest and a stimulant for further research.

Conclusions.
It is not possible to base firm conclusions on the foregoing data, which was expected. Nevertheless there is not any indication that the mechanical properties of the old and used material were inferior to those of nowadays wood, due to relatively long service periods in structures. In this connection three notes are added:

– the 52 years old glulam beams showed low EI-values, probably due to partly delamination. This means that the real stresses in the structure must have been higher than the calculated ones;
– although the bulkhead planks were "only" for 18 and 15 years in use one should bear in mind that in that case the loads were acting permanently – but of course to an unknown level;
– the original properties of the wood are of course unknown; a quality difference of the bulkhead planks was mentioned, leading to slightly higher bending strength of the older material. This however is not the same as saying that the intrinsic wood properties (small clear) were different.

So it is not proven that the relatively high strength values of the used wood could not be the result of originally much higher strength material with reduced properties due to long time use.

19-7-5 T Feldborg, M Johansen
Slip in joints under long-term loading

Introduction
For timber trussed structures with mechanical fasteners the main part of the deflection of the structure is due to slip in the joints. In chords, continuously over two or more spans, the stiffness of the joints has considerable influence on the bending moment distribution to spans and supports at the nodes.

Full scale experiments with timber trussed rafters, type W, mounted as roof structure of an open house were carried out in the years 1974-1978 at SBI. The structure was long-term loaded, and measurements were taken of the deflections of the trussed rafters and deformations of the splice joint at midspan of the lower chord. The moisture content of the timber varied in accordance with the outdoor climate.

The deformations varied very much and were bigger than expected. It was therefore decided to perform long-term tests on splice joints under controlled climate conditions in order to study how the deformations and the strength are influenced by alternating moisture content. One of the aims is to get realistic design values for slip in joints when calculating internal forces and deflections of timber trussed structures.

The tests are being carried out at the laboratory of the Danish Building Research Institute (SBI). The loading was applied in the autumn of 1984 and the main preliminary results of the slip measurements that will be continued for half a year are reported below.

Conclusion
The absolute slip values are rather high for the joint types other than the joint with nail plates and for these types the slip will often be decisive for the design of the joint.

From the values of relative slip it appears that stiffness of joints based on slip measurement from short-term loading does not give sufficient information about the development of the slip due to long-term loading. The relative slip varies considerably for the different types of joints.

Further tests are needed to clarify the influence of other load levels and other humidity histories.

19-9-1 R O Foschi, Z C Yao
Another look at three duration of load models

Introduction
The problem of load duration effects in timber and wood-based products has attracted considerable attention during the last decade. In particular, the research effort has concentrated on determining the differences between the behaviour of material in structural sizes and that of small, clear specimens. Results reported by Madsen and Barrett (1976) first showed that Douglas fir lumber in bending did not follow the trend of the "Madi-

son curve", derived from small clears and traditionally used to quantify duration of load effects for all structural applications of wood. Tests were subsequently started at the Western Forest Products Laboratory in Vancouver using Western hemlock lumber in bending. The results have been reported by Foschi and Barrett (1982), and they not only confirmed the experimental trend observed by Madsen but the conclusions were reinforced by a substantially larger sample size.

In the U.S., tests were started at the Madison Laboratory by Gerhards (1977, 1986), using a sample of 2 x 4 lumber particularly selected to provide low short-term strength material. As a result of this activity, a joint Canada-U.S. project was begun between Forintek Canada Corp. and the Madison laboratory. The planning included testing of spruce lumber in Canada and Douglas fir in the U.S., in two sizes, two lumber qualities, in bending, tension and fully-restrained compression. The Canadian part of the project is well underway, with the bending testing almost complete for 2 x 8's and continuing for 2 x 4's. Testing in tension and compression is equally advanced. Preliminary results from the bending tests were presented during a symposium on load duration organized by Forintek Canada Corp. (Foschi and Barrett, 1985).

Experimental programs are also underway in Europe, particularly in the U.K, Denmark and West Germany. The interpretation of the experimental results has also received substantial attention. The development of "duration of load models" is required to link the conclusions from the tests, performed under constant loads, to the more general design situation of loads varying over time.

Essentially two approaches have been used in the development of load duration models: 1) accumulation of damage and 2) fracture mechanics or crack propagation. The work of Gerhards and Foschi and Barrett uses the concept of damage accumulation, while Nielsen's is based on crack propagation in a material with viscoelastic properties.

In the first approach, "damage" is seen as a state variable ranging from 0 at the beginning of load application to 1 at failure. Several types of damage accumulation laws could be postulated, depending on whether the accumulation rate is assumed to depend only on the stress level or also on the previously accumulated damage. Election of a particular form for the damage law must rely on how well it will match the experimental results when the corresponding test load histories are entered. Although "damage" cannot be measured directly, one can think of it as being implicitly related to more "physical variables". For example, an admissible definition of damage would be the ratio between the current crack length and the crack length at failure, since it satisfies the range conditions (0-1). Simple damage accumulation laws, depending only on stress level, have long been used in the study of fatigue in metals: Miner's rule of linear damage accumulation is a well known example.

The fracture mechanics approach postulates a law for the speed at which a crack will grow under stress. In linear, elastic fracture mechanics the law commonly used is an empirical relationship between speed and level of the stress intensity factor, derived from tests. Since the stress intensity factor depends on the crack length, this law implies that the crack speed is controlled both by stress level and the current length of the crack. Nielsen has proposed a similar model but, instead of making the assumptions of linear, elastic materials, he considers a material which behaves viscoelastic around the crack tip. Thus, upon load application, the material around the tip deforms (without the crack propagating) until a critical deformation is achieved. At this time, the crack advances and the process repeats itself until the crack achieves a critical length at which very rapid failure follows.

One objective of this paper is to discuss briefly these three models and their ability to represent data from an experiment on Western hemlock lumber in bending (Foschi and Barrett, 1982). A second objective is to discuss the problems which each of these models present when they are used in the development of reliability-based design procedures.

Conclusions

Three duration of load models have been considered. Two of them, the fracture mechanics approach of Nielsen and the damage accumulation model by Foschi were found suitable to represent accurately the experimental trends in Western hemlock lumber in bending. The model by Gerhards was found to be lacking in the flexibility needed to follow the experimental trends. The fracture mechanics model is not easy to use, since it requires numerical integration to obtain the time-to-failure in all cases when the applied stress varies with time. In this context, applications of these models to reliability calculations will be easier using damage accumulation. A general calibration program has been written, but fitting must be done carefully and with a degree of judgment.

19-9-2 P Hoffmeyer
Duration of load effects for spruce timber with special reference to moisture influence - a status report

Introduction
The present paper reports on a project which is still incomplete in terms of testing and analysis. The paper is an interim report and presents the limited results of analysis carried out to date.

The research project is aimed in particular at an investigation of the possible influence of moisture content on the effect of duration of load on spruce timber. Such an effect was earlier found for clear wood.

The project is part of a joint programme between Princes Risborough Laboratory (PRL) and The Technical University of Denmark. The Swedish Institute for Wood Technology Research (Träteknik Centrum) has supplied the test samples and carried out initial strength- and NDE-tests to ensure matching of the British and Danish samples.

Conclusions
Definite conclusions are not to be drawn on the basis of these preliminary results. However, some general trends may be seen:

The curve for the higher stressed sample is above that for the lower stressed sample. For moist timber the effect seems to be there even for a long period of time, whilst for dry timber the effect seems to vanish after a short time. This suggests that low quality timber may be more severely weakened by load duration than high quality timber. This is opposite to the effect found by Madsen and Barrett.

Although the difference may well be within experimental variability, it is suggested that the difference is real, and based on the fact that tension perpendicular to grain failure – which is the predominant failure mode for timber subjected to bending – is known to cause shorter time to failure than tension and compression parallel to grain. Gerhards and Link in a recent paper also found a duration of load effect for timber in bending in excess of that predicted by the Madison curve.

If curves for higher stressed samples are above curves for lower stressed samples, it may be concluded, that the present investigation shows a duration of load effect which already during the first few months of loading is of the same order as that predicted by the Madison curve.

The effect of moisture content on the duration of load for timber has been shown to be no less than that found for clear wood. The curves are shifted towards longer times, but the curves for timber runs parallel to the curves for clear wood, and the log-time difference between dry and moist states is of the same order of magnitude.

19-9-3 T A C M van der Put
A model of deformation and damage processes based on the reaction kinetics of bond exchange

Introduction
Long term creep tests during 10 to 23 years on Joints, where the wood is determining for the strength, indicate that deformation- and damage behaviour of wood is non-linear even at low loading levels and can be explained by the theory of reaction kinetics of deformation. This is confirmed by tests on clear wood, done as part of an E.E.C.-project. The basic concept of this theory of deformation kinetics is to regard plastic flow as a chemical reaction of molecular bond breaking and bond reformation. For wood this approach is complicated by the complex structure, and only an estimate of the involved processes or molecular parameters will be possible.

As long as the molecular structure and interaction is not entirely understood, every approach will have a phenomenological character and it is at first necessary to state here a phenomenological model in a physical right form and to derive the possible simplifications to be able to find the main determining processes and to explain the behaviour. The predictions of the theory will be compared with general experimental tendencies of the behaviour of wand known from literature.

Conclusion
It is shown that the derived model is able to explain experimental phenomena, although a simplified analysis and simplified properties of the parameters where used to be able to obtain solutions in a functional form.

19-9-4 U Korin
Non-linear creep superposition

Preface:

A method is presented for the representation of the behavior of viscoelastic materials under complex load histories. The method was applied during a study of the creep of glass reinforced polymers for the prediction of the creep of reinforced specimens. It was also applied to the transformation between various viscoelastic functions, like creep to relaxation, creep to stress-strain. The method is based on the basic creep superposition principle, and it was modified to treat non-linear viscoelastic materials.

It is the author's feeling that the method may be used also for analysis of creep of timber and creep superposition of timber. The paper presents the theoretical background of the proposed method and the development of the proposed superposition model.

The proposed method is programmed in computer language and may be easily applied for the analysis of creep data and for the analysis and prediction of creep superposition behavior.

19-9-5 R Kliger
Determination of creep data for the component parts of stressed-skin panels

Abstract

The main aim of this investigation has been to predict the creep characteristics of stressed-skin panels consisting of wood-based materials in the web and compression flange and a steel sheet in the tension flange. This paper describes studies of the long-term bending and shear characteristics of wood beams, the compression characteristics of particle board and the shear properties of glued joints between
(a) wood and particle board and
(b) wood and steel.

Two different rheological models are used to fit the creep data, which corresponds to the behaviour of a linear visco-elastic material, which creeps under sustained loading. It is shown that the selection of the time for the initial measurement (after application of the load) is of great importance.

19-9-6 P Glos
Creep and lifetime of timber loaded in tension and compression

Summary

As a supplement to existing investigations on small, clear specimens and to full-size bending tests, long term tension and compression tests on full-size European spruce specimens (picea abies) were carried out for the first time. The material tested consisted of glulam laminates, collected as representative samples from German glulam plants. The tension and compression specimens had cross-sections of 30x120 mm and 30x180 mm and a gauge length of 1.65 m. In 10 test series a total of 212 specimens was subjected to constant load tests (stress level 14 N/mm^2 and 21 N/mm^2; loading time 6 months) and to ramp load tests (time to failure 10 sec, 1hr and 240 or 368 hrs). All tests were carried out under constant climate conditions at 20 C and 65 % r.h.

Both under tension and compression tests a pronounced duration of load effect was noticed which was clearly stronger in the lower range of strength distribution than reported in earlier bending tests carried out in Canada. The duration of load effect seems to be equally pronounced in both tension and compression despite differences in creep behaviour. Its magnitude corresponds approximately to that of the Madison curve. The test results support the assumption that unless the applied stress surpasses a threshold of roughly 50 % of the respective short term strength of timber apparently no damage or strength degradation over time is caused. A modification factor which would take into account the duration of load effect in structural timber design codes would depend not only on load history but also on strength distribution and consequently on the timber grading system.

Conclusions

All test results presented here are based on a very limited number of tests on European spruce (picea abies). With this reservation the following conclusions can be drawn:

1. Structural timber exhibits a significant duration of load effect in tension and in compression under constant climate conditions. The test results indicate that this effect is more pronounced in the lower range of strength distribution than was the case for bending tests carried out in Canada. Nor were Canadian results supported by new ramp load bending tests with

American Douglas-fir 2 by 4 timber. In these tests it was also found that the rate of loading affects the total distribution of bending strength. Whether these differences are partly due to differences between wood species or grading rules remains to be determined in current investigations in various North American and European countries.
2. The test results indicate that the duration of load effect seems to be equally strong in both tension and compression despite differences in creep behaviour. In the particularly interesting lower range of strength distribution the magnitude of the duration of load effect both for ramp load and constant load tests corresponds approximately to that of the Madison curve. A tri-linear curve as found in Canadian tests with full-size bending specimens could not be confirmed in the present tests.
3. Several independent investigations indicate that unless the applied stress surpasses a threshold of roughly 50 % of the respective short term strength of the timber apparently no strength degradation over time is caused. From this follows that strength of timber will only be affected if its short term strength is less than twice the maximum applied stress. This is the case for a small portion of the total timber population only, namely for low strength timber around and below the population's 5th percentile. According to the present study it has to be assumed that low strength timber is also subjected to a significant duration of load effect. Therefore this effect has to be taken into account in structural timber design codes.

Only load effects which surpass 50 % of the short term strength of the weakest component will contribute to the damage and strength reduction in structural timber. The amount and magnitude of critical stresses in that range are stochastic values and depend on the total code system, i.e. the underlying partial safety factors for load and material, load assumptions and the grading system. Therefore a modification factor which would take into account the duration of load effect in structural timber design codes cannot be derived directly from a stress ratio - time to failure diagram as done in the past. It has to be determined by means of probability analyses which include all important parameters. Studies at hand indicate that depending on partial safety factors, load assumptions and grading systems an adequate value may range from as much as 0.7 to 1.0.

20-7-2 T Feldborg, M Johansen
Slip in joints under long-term loading

Introduction
A paper on timber joints under long-term loading was presented in **Paper 19-7-5** The paper to which reference is made described the test programme, the specimens, the test performance and the slip development during approx. 600 days of loading and 7 cycles of alternating relative humidity (RH).

The tests have been continued until 784 days (2,15 years) of loading and 9,5 cycles of alternating RH. After the unloading the stiffness and maximum strength of all joints were determined in a standard short-term test. The experimental work was finished in the spring of 1987 and the main results are reported below.

22-9-1 M Badstube, W Rug, W Schöne
Long-term tests with glued laminated timber girders

Introduction
The present report is dealing with the preparation and implementation of long-term investigations and tests with glued laminated timber girders subjected to bending. The layers of the "BSH 1V13"-type glued laminated timber have been sorted mechanically to obtain strength grades. The tests are being carried out under an outdoor roofed storage facility and will be continued for a minimum period of 10 years. In this connection, the girders are being exposed to a variable long-term loading.

The objective of the tests and investigations is the measurement of time-dependent deformations and the determination of the residual load-bearing capacity after a loading period of at least ten years.

From the results and findings, the creep factor k_{creep} and the modification factor as to "load action period" k_{mod1} will be obtained.

Summary
Long-term tests and investigations are being carried out using glued laminated timber girders subjected to bending with the action of a variable lonrg-term, load at a climate (environment) of "outdoors under a roof".

Initial results and findings of the measurements are providing information on the magnitude of the creep factors to be expected.

24-9-1 T Toratti
Long term bending creep of wood

Summary
Long term creep and recovery test results of wood in a bending load of 10 MPa stress and subjected to relative humidity cycling are analysed. A mechanosorptive model that fits the test results is proposed. Simulated values of creep at ten years of loading are presented using the model. According to the model, the for bending deflection can be about doubled to account of the creep of ten years loading with a cyclic load of 10^{-3} MPa and subjected to a natural outdoor relative humidity.

Conclusions
Creep test results of very long load duration and subjected to a high number of relative humidity cycles are analysed. The results do not seem to support the existence of a mechano-sorptive creep limit. Recovery of deformation does not seem to be complete but is to a certain extent of plastic nature. The irrecoverable deformation increases as mechano-sorptive creep increases. A mechanosorptive model is presented based on the above observations.

The simulation of the bending creep of wood in natural outdoor environment conditions, using the model presented, results in the following relative creep and k_{creep} values:

Definitions: $e(t) = (1 + k_{creep})\ \sigma / E$

Relative creep: Total deformation per elastic deformation.

	Loading	Relative ceep	k_{creep}
Solid wood	Constant 10 MPa	2.75	1.75
44 x 94 mm^2	Cyclic 10^{-3} MPa	2.20	1.20
Glulam	Constant 10 MPa	2.40	1.40
190 x 1460 mm^2	Cyclic 10^{-3} MPa	2.00	1.00

24-9-2 A Ranta-Maunus
Collection of creep data of timber

Creep data
A collection of existing creep data of timber is made in order to assist code writers. Only experiments with direct relevance to structures are included: structural size, allowable stress level and minimum duration of load 6 months.

Data is given in tables, and the values are expressed in terms of k_{creep} defined by

$$E = \left(1 + k_{creep}\right)\sigma / E$$

Data is divided into 3 climatic groups:
– artificially controlled in order to keep constant humidity
– naturally changing humidity
– artificially controlled to have strong cyclic variation

In some cases values for 50 years are calculated by the models given in the articles. For joints only data concerning nail-plate connections is collected.

Ufortunately, very little creep data related to service class 3 (Eurocode 5) has been available.

24-9-3 I R Kliger
Deformation modification factors for calculating built-up wood-based structures

The main purpose of this paper is to discuss the use of creep factors k_{creep} when calculating deflection in built-up structures.

Conclusion
It is obviously very difficult for the code writers to have general rules and recommendations and to cover all the possible and "impossible" design cases at the same time. However, most timber members are built into a structure in one way or another. Most structural elements with various material and joint combinations can be designed with high accuracy at the initial stage. In this case, the differences in the calculated and actual initial deflection caused by high scatter in the modulus of elasticity in timber and timber-based materials are not the subject of this paper. When the effects

of moisture and creep are added, a normal design procedure for structures with a long life expectancy, more expensive material combinations or connections will probably make the design much too conservative.

When a steel sheet for example is rigidly connected with timber beams, it would perhaps be reasonable to use smaller reduction values of the modulus of elasticity (k_{creep}) due to creep for these beams than the values obtained from the codes. Another solution could be to use some sort of reduction factor for the calculated deflection of built-up structures in Serviceability Limit State Design which takes account of various material combinations and durable connections.

In order to promote the use of built-up or composite wood-based structures with more rigid and durable connections (which are often more expensive) in future, we should find a way of producing designs on the conservative side and of taking account of various material combinations and various connections at the same time.

26-9-1 S Thelandersson, J Nordh, T Nordh, S Sandahl
Long term deformations in wood based panels under natural climate conditions. a comparative study

Introduction
Most long term studies of the behaviour of wood based panel products has been performed under controlled moisture conditions, mainly with constant relative humidity. In practice, the relative humidity is always more or less variable. For this reason, the relative ranking in design of the materials with respect to creep factors and moisture sensitivity might not reflect the performance in practice in an adequate way. The objective of the investigation reported here was to study the relative performance of some wood based materials under rather humid and variable conditions. To this end, comparative long term tests were performed for a number of panel products exposed to the same natural conditions. The design codes considered in the analysis of results given in this paper are Eurocode 5 and the Swedish building code.

Summary and conclusions
The following main conclusions can be drawn from the investigation so far:

1. The relative ranking between the tested materials used in Eurocode 5 and in the Swedish building code is reasonably correct with regard to creep factors.
2. The creep factors specified in the codes seem to be somewhat underestimated for all structurally classified materials considered in the investigation.
3. The rate of creep deflection for materials with a high degree of processing such as hardboard, MDF and particleboard is markedly higher during wet periods than under dry periods.
4. For wood the rate of creep is largest during dry (or drying) periods. During wetting periods the wood beams exhibit a spring back i. e. the beams rise against the load.
5. Plywood and OSB exhibit similar behaviour as wood, but the rate of creep is less dependent on humidity changes.

The tests will continue for at least another one year period. This may give further experience, which could modify the above preliminary conclusions.

28-9-1 R Gupta, R Shen
Evaluation of creep behavior of structural lumber in natural environment

Abstract
In order to describe creep behavior of structural lumber in natural environment, a bending test with twenty Douglas-Fir beams subjected to a constant load was conducted under an open shed in the Forest Research Laboratory at Oregon State University. Deflections of the beams were measured along with daily fluctuations in temperature and relative humidity. An existing five-element creep model was used to fit the experimental data. The five-element model did not describe creep behavior of structural lumber in natural environment. The general observations show that stiffness of the beams has strong influence on magnitude of creep strain, and the creep strain closely follows the fluctuations of air temperature. A four-element model, including the stiffness and air temperature effects, has been developed. The model fits the experimental data very well.

Summary

Based on the analysis of the data from the creep experiment over six months, it may be concluded that creep deformation can be much larger than elastic deformation, and it depends upon the stiffness of the beams.

The Fridley's five-element creep model does not predict creep behavior of structural lumber in natural environment. Due to the little change in moisture content in the specimens, the mechano-sorptive element in the five-element model does not contribute creep strain of structural lumber subjected to the natural cyclic environmental conditions.

It was observed that the fluctuations of creep strain follow the variations of air temperature. Temperature fluctuations and edge-wise MOE are considered to be two major effects in modelling. A four-element model was developed to predict the creep strain of structural lumber in natural environment. The four-element model has fitted the experimental data very well. The four-element model has verified that MOE of the beams does affect the creep strain.

Literature cited

30-9-1 S Aicher, G Dill-Langer
DOL effect in tension perpendicular to the grain of glulam depending on service classes and volume

Abstract

This paper reveals the concept and today's results of an extensive experimental and theoretical research work on the DOL effect of glulam in tension perpendicular to the grain with special respect to different climates and volumes. In the different test series with about 180 large scale specimens the primarily examined parameters were the influence of climate conditions – service class I and two different service class 2 environments – and the influence of volume (0,01 resp. 0,03 m^3) comprising two cross-sectional widths of 90 resp. 140 mm. The investigated glulam volumes, built-up from machine graded laminations, conformed acc. to characteristic densities to high grade European glulam strength classes (GL 32 resp. 36). Due to time ranges of applied stepwise loading regime the empiric results apply primarily for short and medium-term duration of load classes. It is shown by means of damage model considerations that the stepwise loading regime has almost no influence on the DOL median in constant climate. The empiric results reveal that todays modification factors for short and medium term duration of load classes, specified in Eurocode 5 for service class 2 conditions, are highly non-conservative in case of tension strength perpendicular to the grain.

Conclusions

The results of the presented research work prove a significant and extreme difference of the DOL effect in service class 1 and 2 in case of tension perpendicular to the grain of glulam. Consequently the empiric k_{mod} values for primarily design relevant short and medium term duration of load classes are well below those assumed today in Eurocode 5 for service class 2 conditions.

Finished and still ongoing DOL tests with the larger volumes (V = 0,03 m^3) indicate that the volume effect, conforming to a Weibull shape factor of about 5 in ramp load tests, might be slightly less severe in long term loading.

The investigations were conducted with glulam of high strength classes; it should be checked whether the results apply in equal rigidity to lower quality material.

30-9-2 G Dill-Langer, S Aicher
Damage modelling of glulam in tension perpendicular to grain in variable climate

Introduction

So far, the so-called Canadian damage model has been applied primarily to model the damage evolutions of North American lumber subjected to long term bending loading. Hereby load history results solely from gravity loads and climate is perceived as a parameter implicitly included in the damage state variable. A few attempts are known for an explicit recognition of the effect of transient moisture history in a damage model, either based on the US-model or on the Canadian model); in all cases bending was regarded.

Table 3 contains a compilation of the individual short term Weibull stresses and the empiric altogether with the calculated times to failure for all eight failed specimens. It may be seen that damage calculations based on transient Weibull stress level yield in general a good qualitative prediction of times to failure. The quantitative agreement is satisfactory for most specimens, except specimens where the damage evolution curve exhibits a plateau which leads to major deviations in failure time due to minimal variations of damage parameters or load history.

Conclusions

The Canadian damage model was applied to DOL tests of glulam loaded in tension perpendicular to the grain at constant and changing climate conditions.

The calibration of the model parameters was performed on the basis of empiric data obtained from constant climate tests with stepwise increased loading and from related results of ramp load tests. Hereby, two different sets of parameter start values, representing two qualitatively different shapes of damage accumulation functions, turned out to yield equally good fits of times to failure.

The idea of the presented damage modelling approach in case of varying climate conditions with resulting high eigenstress terms was to keep the basic Canadian model. In an extension the stress level should be expressed by an adequately derived damage relevant Weibull stress level. The stress history of DOL tests at cyclic varying humidity was derived through transient diffusion and mechanical FEM-calculations including mechano-sorptive creep and the structural effect of polar orthotropy due to annual ring curvature. The resulting transient stress field was integrated to a damage relevant Weibull stress which was related to short term Weibull strength by ranking method. The derived Weibull stress levels were used as load history input for the Canadian damage model. The results of the damage calculations yielded a qualitatively good prediction of times to failure.

The presented new approach to incorporate changing climate influences into damage calculations through an apt stress scalar (Weibull stress level) seems to be promising for all cases where moisture gradient dependant eigenstresses are important.

31-9-1 A Ranta-Maunus
Duration of load effect in tension perpendicular to grain in curved glulam

Introduction

The strength of timber depends on its moisture content and the duration of load, which has been commonly known for a long time. Strength reduction is accelerated by moisture variation as has been observed under bending load as well as under tension perpendicular to grain. In the latter case, moisture gradients tend to cause local swelling or shrinkage, which changes the stress distribution and has an impact on load bearing capacity.

This paper concentrates on cases where a failure of curved glued laminated timber beam is caused by tensile stresses perpendicular to grain. The theoretical focus of this research is on stress distribution through thickness. During this research period the focus has been shifting: at the beginning it was believed that creep, especially mechano-sorptive creep, will change the stress distribution and consequently impair the load bearing capacity. This being still true, another factor seems to be more important: moisture induced stresses in internal parts of the beam during wetting can easily exceed all other effects. Accordingly, duration of load effect is partly being replaced by "maximum moisture gradient during the life time-effect".

The reasons why the observed strength under long-term loading is lower than that obtained in short term testing at constant moisture content are defined in this paper as follows:

- material is weakened by accumulating damage due to a constantly high stress level and duration of the stress. The factor to take this reduction into account is denoted by kj.
- stress distribution: load bearing capacity of the member is lowered by changes in stress distribution caused by creep and hygro expansion. The factor used to take this reduction into account is denoted by ka.

This paper summaries the experimental and calculation results. The total effect of load duration combined with moisture effects is calculated by $k_{DoL} = k_\alpha k_t$.

Firstly, k_t is calculated based on experiments carried out under constant humidity conditions, and finally k_α can be determined from cyclic humidity tests.

Conclusions

Tensile stresses perpendicular to grain in curved and tapered beams, and in tension specimens cut from glulam, are found to be higher in the middle section, in the plane where the pith is located, and much lower over the rest of the cross-section. The same is true also during moisture cycling in long term loading. Thus, failure will start when stresses in the middle exceed the strength. In practical situations, maximal stresses are obtained when a dry period is followed by a humid period, and both periods are long.

When the effect of moisture changes is compared to the effect of mechanical loading, we can conclude that the moisture load caused by the analysed cycles corresponds to an extra load of 0.15 to 0.35 MPa when acting simultaneously with a mechanical load causing stress of 0.2 MPa, when the beam is not surface coated. A good coating will decrease the moisture load by 70 %. A single fast change from 65 % RH to 90 % RH seems to be more effective than the test cycles analysed.

While moisture gradients proved to be more important than realised in advance, some other factors appeared to be less important: creep after several moisture cycles seems not to change the level of stresses from that of the first cycle. Accordingly, for the analysis of the duration of load behaviour under tensile stress perpendicular to grain, it is of great importance to consider the largest moisture cycle or change. All other duration of load effects are of much less importance and can well be estimated on the basis of the traditional stress ratio vs. log time to failure graph based on ideal constant humidity experiments. Under ideal constant conditions duration of load effect in tension perpendicular to grain is slightly less severe than suggested by the Madison curve.

Volume effect during long term loading is found to be of the same order than adopted in Eurocode 5. The strength of a curved beam in comparison to a tensile specimen is higher than expected justifying the use of a higher k_{dis} value, 1.85 for curved beams. On the other hand k_{mod} factors should be lower, about 0.5 for medium and long term loads, or an additional moisture load should be calculated, depending on the surface coating or impregnation of curved beams.

32-9-1 P Becker, K Rautenstrauch
Bending-stress-redistribution in tension and compression and resulting DOL-effect

Introduction

Creep is generally assumed to be larger for compression than for tension to some degree. This results in a change of stress-distribution. The stress for the tension edge will increase and decrease for the compression edge. An analytical solution of this behaviour can be easily derived with the assumption of creep limits for both impacts.

The resulting stress-distribution will lead to a decreasing computational bending strength value, because the stress for the tension edge, which will be finally responsible for failure, has already increased during lifetime of a structural element. The exact difference can be determined by simulation. For initial and resulting stress-distribution load increments are applied until tension strength is reached for the tension edge. Plasticating ability of wood subjected to compression is considered.

Conclusion

Different creep in tension and compression was observed by many researchers. The exact difference can hardly be quantified because it strongly depends on climatic and loading conditions and also on the quality of the material.

It generally leads to changing edge stresses, which under the assumption of linear viscoelasticity can be easily determined using creep factors. If certain factors are given for compression and tension creep a bending creep factor, which is slightly lower than the mean of tension and compression creep, can be determined also.

Because of the strong ability of the material to plasticise under compression, the influence of an increasing tension stress on the duration-of-load phenomenon turns out to be quite low, almost negligible for normal and high quality material. Taking timber of low quality, the influence becomes larger; a big difference in tension an compression creep can rather be not expected for this kind of material though.

32-9-2 R Grantham, V Enjily
The long term performance of ply-web beams

Abstract

Ply-web beams have been used for a number of years in floor and flat roof constructions, such as swimming pools, that require medium-span structural beams. Many of the advantages associated with ply-web beams stem from their structural efficiency, utilising structural timber flanges and plywood webs. This produces lightweight components that are easily installed on site and reduce the dead weight of construction when compared to solid timber beams. The design of ply-web beams has been thoroughly investigated with the exception of their long-term performance. Very few studies have covered this aspect of design, which may be critical in service condi-

tions producing fluctuating moisture contents in the beam.

This paper reports the results of long-term performance tests on Ply-web T beams over a period of 6 years when loaded to a variety of utilisation ratios in uncontrolled conditions. Both Finish Birch and Canadian Douglas-fir plywood were used for the web of beams, which were monitored for relative creep under conditions of naturally fluctuating relative humidity and moisture content. Comparisons made with the guidance given in EC5 at both the Serviceability and Ultimate Limit State have highlighted some significant differences between design predictions and the actual performance. Firstly, strength tests conducted on ply-web beams with Finnish Birch webs showed an insufficient factor of safety for EC5 designs. At the Serviceability Limit State, the deflection of ply-web beams with both web types was in excess of EC5 predictions. Recommendations have been made for kdf factors to control the creep deflection of ply-web beams.

Conclusions

The indications from the strength tests are that:

– The strength of ply-web beams is not adversely affected by prolonged loading or large deflections.
– Safety factors of more than 2.5 exist between failure loads and permissible design loads using BS5268. Additional pond loading due to creep deflections may erode this safety factor in practice.
– Similar safety factors have been determined for designs using EC5 for Canadian Douglas fir ply-web beams.
– Beams designed to EC5 using Finnish Birch ply-webs had reduced factors of safety beyond that recommended in the UK NAD to EC5.
– Material and construction quality have a more marked effect than load history on the failure load of ply-web beams.

The following may be concluded from creep test measurements, creep modelling and the derivation of creep deflection factors:

– The continued creep of all the ply-web beams tested, indicated that cessation
of creep was not expected within the long-term duration category of EC5.
– Power models used to predict relative creep proved a good prediction for the long-term creep but may overestimate kdef factors for permanent load duration.
– Test data showed that a non-linear relationship exists between the load duration, utilisation ratio and relative creep of ply-web beams. The relationship was markedly different for beams constructed with different types of plywood web.
– The current kdef factors in EC5 grossly underestimate the creep of ply-web beams exposed to fluctuating environmental conditions for all load duration categories.
– The proposed kdef factors control creep deflections within limiting values for a variety load combinations and durations.

36-9-1 A J M Leijten, B Jansson
Load duration factors for instantaneous loads

Introduction

There area number of load cases such as earthquakes and single blasts where timber is exposed to substantially higher loading rates than in the standard short duration test. Examples of single blast loads are explosions but also impact by vehicles against a timber guardrail with loading times much less than one second sometimes up to of a few thousands of a second. Old test data shows that timber is well able to withstand impact loads. However, the validity of the old test data is questioned as more elaborate research indicates conflicting results.

Conclusions and proposal for change of modification factor for instantaneous loads

On the bases of the research evaluated the following conclusions can be drawn:

– In design of timber structures that are exposed to instantaneous loads the inertia effects cannot always be ignored. Design based on static equivalent methods can be very deceptive in reflecting the actual stress situation. For this reason a proper stress evaluation method like computer simulation methods is preferred.
– The strength modification factor for instantaneous loads currently in Eurocode 5, (kmod > 1) is based on impact tests of small clear specimens where the effect of inertia is disregarded.
– The impact bending strength is grade, wood species and loading rate dependent.

– Jansson and Leijten both demonstrated a general tendency that with increasing loading rate the bending strength decreases.
– Commercial grades with knots tend to show a higher degradation of the bending strength than clear free material. The strength degradation is substantial for heat-treated timber.
– Based on the test results evaluated it is difficult to propose instantaneous DOL factors as the shorter the time to failure the more the bending strength drops. However, for structural softwoods in Table 3 a proposal is given to change the duration of load factor for instantaneous load with high impact rates to be closer to reality than current code values.

Table 3. Proposed DOL factor k_{mod} for instantaneous loads for Eurocode 5, Service class 1 and 2 (moisture content 12-20 %.

	Present factor k_{mod}	Proposed factor k_{mod}
Solid timber	1,1	0,85
Heat treated solid timber		0,6

39-9-1 M Fragiacomo, A Ceccotti
Simplified approach for the long-term behaviour of timber-concrete composite beams according to the Eurocode 5 provisions

Introduction
The timber-concrete composite beam (TCC) is a construction technique extensively used for both upgrading of existing floors and new buildings. Many advantages can be achieved by connecting a lower timber beam with an upper concrete slab, including increase in strength and stiffness, better seismic performance, larger thermal mass and fire resistance, better acoustic separation, and the possibility to maintain the timber floor when restoring existing buildings.

The design of the TCC must satisfy both ultimate (ULS) and serviceability (SLS) limit states (CEN 2003). The former are controlled by evaluating the maximum stresses in the component materials (concrete, timber and connection system) using an elastic analysis. Such approaches generally lead to solutions sufficiently accurate provided that realistic properties of the connection system are employed.

The SLS are checked by evaluating the maximum deflection both in the short- and long-term. Timber, concrete and connection systems, in fact, all show time-dependent behaviour. Concrete is characterised by creep, shrinkage and thermal strains; timber exhibits creep, mechano-sorptive creep, which is the increase in delayed strains due to cycles of moisture content, shrinkage/swelling, and thermal strains; the connection system itself is also characterised by creep and mechano-sorptive creep. This results in the long-term behaviour being affected also by the environmental conditions. Some numerical models have been proposed in order to evaluate rigorous solutions. The former FE model was compared against a number of long-term experimental tests showing the possibility to predict accurate results. The software accounts for all of the aforementioned time-dependent phenomena using the Toratti's rheological model for the creep and mechano-sorptive creep of timber and connections system, and the CEB-FIP Model Code 1990 formulas for the creep and shrinkage of concrete. The actual distribution of moisture content, which was proved to be highly variable over the timber cross-section, is computed by solving the diffusion problem for a given history of environmental relative humidity. The temperature distribution, much less variable, is instead considered as constant over the cross-section but variable in time.

A simplified analytical approach was also proposed, however further research highlighted that some phenomena ignored in such an approach such as concrete shrinkage and inelastic strains due to environmental conditions may lead to significant errors and make the simplified solution not conservative. Some more accurate methods for the long-term behaviour have then been proposed. Kühlmann & Schäntzlein proposed the use of effective values of creep and shrinkage to take into account the different trends in time of those coefficients in the component materials (timber, concrete and connection system). Fragiacomo proposed an improvement of the Ceccotti's approach in order to account for the mechano-sorptive creep, concrete shrinkage, and inelastic strains/stresses due to environmental temperature and relative humidity variations.

The aim of this paper is to propose a simplified yet accurate solution for the long-term behaviour of TCC's. The approach will allow the designer to account for the whole loading history of the structure including concrete shrinkage, effect of props during construction, and inelastic strains due to environmental variations. The accuracy will be assessed by comparison with numerical solutions on a number of TCC's of technical interest. Finally the influence of different environmental conditions such as outdoor and heated indoor conditions on the long-term performance will be discussed.

Conclusions

The paper investigates the long-term behaviour of timber-concrete composite beams (TCC´s). A simplified yet accurate approach based on simple closed form solutions has been proposed for the prediction of all relevant quantities. The approach is an extension of the method suggested by Ceccotti to account for the mechano-sorptive effect, concrete shrinkage, shrinkage/swelling of timber and concrete due to environmental variations, and modality of construction (propped construction). The proposed approach has been compared with the current approximate method and with the numerical solution carried out using a rigorous Finite Element program. The influence of different environmental conditions (outdoor, heated indoor conditions) on the behaviour of the TCC has also been investigated. The primary observations of this research are reported herein after.

1. the Toratti rheological model, which accounts for the mechano-sorptive effect, is hardly dependent, in the long-term, upon the yearly moisture content variation Au for values larger than 1.65%.
2. the creep coefficient of connection, assumed by the EC5 twice as large as the creep coefficient of timber, seems to be overestimated according to the outcomes of some tests.
3. the use of the effective modulus method for evaluating the effect on the TCC beam of creep and mechano-sorptive creep leads to accurate results.
4. the effect of concrete shrinkage can be precisely calculated using the rigorous elastic formulas for TCC's with flexible connection, with the inelastic strain due to shrinkage being measured from the time of concrete curing. The creep is taken into account using the effective modulus method, and the influence of the props is negligible.
5. the shrinkage/swelling due to environmental variations can be taken into account using the rigorous elastic formulas for TCC's with flexible connection. The temperature variations in the concrete and timber beams are assumed equal to the environmental one. The moisture content variation in the timber beam is assumed constant in each point and equal to the average value over the cross-section.
6. the numerical-analytical comparison points out that the proposed approach leads to accurate results in terms of deflections and stresses. Conversely, the use of the current approach which neglects concrete shrinkage and shrinkage/swelling due to environmental variations leads to non-conservative and, therefore, unacceptable approximations.
7. the change of environment from outdoor to indoor heated conditions, characterised by one-third of temperature variations and the same or half moisture content variations, leads to minor reductions in deflections and stresses. Some major reductions take place only if no moisture content variations occur at all. However, this case seems more a pure theoretical limit than a real possibility. According to these outcomes, which are based on the use of the Toratti rheological model, the type of environment (outdoor, or heated indoor) does not seem to play an important role on the performance of the TCC beam. However, since this outcome is strongly dependent on the type of the rheological model used for timber, a confirmation should be searched using other types of rheological models.

42-6-2 R Widmann, R Steiger
Impact loaded structural timber elements made from Swiss grown Norway spruce

Introduction

The consideration of an impact load is rather a scarce scenario when designing structural timber members. Vehicle impact on timber guardrails or on unprotected bridge columns as well as impact of stones and/or snow on respective wooden barriers are common examples.

The unintentional impact of falling/jumping persons on wooden scaffold boards or planks and its sometimes fatal consequences was the motivation for research project carried out at EMPA. No design codes exist in Switzerland and Europe to cover this kind of loading and literature only delivers few data for impact bending strength of structural timber elements. In addition the application of e.g. the existing k_{mod} factor according to Eurocode 5 for such cases is questionable and in literature it is also discussed if the published factor is correct for impact loadings in general.

Hence, a test program was initiated in order to obtain basic data about impact bending behaviour of large sized members in order to support the development of adequate regulations in regulations and codes.

Summary and Conclusions

The impact bending and static bending tests with square cut beams and boards from Swiss grown Norway Spruce and a related research showed that:

- The used methods and technologies for the determination of impact energy in form of impact work and loss of kinetic energy worked well and the respective data correlated well. However, the determination of the point/time of rupture on base of the data was not easy.
- Impact bending strength – determined with a standard approach without consideration of inertia effects – was higher than static bending strength and impact work was higher than static work. High slenderness ratios lead to higher bending strength but this could not be observed for impact work.
- On base of the obtained data no conclusion in regard of load duration classes according to EC5 can be drawn.
- Data analysis is still ongoing in order to evaluate the influence of other parameters on the impact bending behaviour.
- Data from different sources are difficult to compare as tailor made equipments was used for the respective tests. The results of our tests only partly match published results. One of the reasons for this could be that inertia effects were not (yet) considered in our analysis.
- If data of impact tests should be used for the determination of load duration factors like kmod a minimum of standardization of test method(s), equipment as well as of analyze-tools should be available. This would also be helpful for further research activities and could be a research topic itself at the same time.

2.8 STATISTICS

13-17-1 W G Warren
On testing whether a prescribed exclusion limit is attained

Introduction
The problem addressed is that of testing whether a prescribed exclusion limit (fifth percentile) is attained in a well-defined population of plywood panels. This population can be envisaged as, and may in actuality be, a shipment. It is assumed that sampling of the shipment is effectively at random, and that the sample size is a negligible fraction of the population (shipment).

Several strategies will be considered:

(1) Fixed sample size, normality assumed
(2) Sequential sampling, normality assumed
(3) Fixed sample size, distribution free
(4) Sequential sampling, distribution free.

For (1) and (2) a study of the robustness against departures from normality will also be undertaken.
Sequential sampling with normality assumed was considered by the writer in a Supplemental Report (May, 1977). In this report the development of the operating-characteristic curves is not correct; an approximation of the unconditional distribution of the statistic x - k_s was used instead of the conditional distribution given that a decision had not been reached at a previous stage. The implementation of the scheme is not affected but the operating characteristics are in error, although it is difficult to ascertain the magnitude of the error. Indeed, the curves obtained may well be reasonable approximations.

A sequential strategy, based on the normality assumption, thus needs redevelopment. Mathematically this is not an easy task. Since operational sample sizes are urgently required, a fixed-sample size strategy will be considered, firstly, with sequential methods to be discussed in a subsequent note.

16-17-1 P Glos
Notes on sampling and strength prediction of stress graded structural timber

Abstract
There are various factors that influence the quality of estimating characteristic stress values. Among these is the definition of the specific population about which inferences are to be made, decisions about the sampling method and the sample size as well as the statistical definition of the characteristic stress value and the choice of the statistical model used for its computation.

For many of the decisions mentioned above the statistical theory does not offer satisfactory decision rules. Hence in some cases the engineer must decide intuitively. Regarding the harmonization of codes and standards it would be helpful if such decisions were standardized internationally.

As a contribution to the discussion of this topic this paper summarizes the potential errors that may arise when estimating the 5 percent exclusion limit and tries to assess their magnitude as a function of the sample size and of the underlying statistical assumptions.

16-17-2 B Norén
Sampling to predict by testing the capacity of joints, components and structures

Background
During the last ten years the working group CIB-W18 has concentrated on an European model code for timber structures: CIB Structural Timber Design Code. Design methods are dominating in this code in terms of formulas for calculating deformation and strength of structures and structural components. Introductionary are also presented strength classes for structural timber and glued laminated timber with the correspondent profiles of characteristic strength at different kind of stressing.

Additionally, CIB-W18 has dealt with methods of verifying characteristic values of stiffness and strength of material, joints, components and structures of wood. This work had been carried out in subgroups set up by CIB and RILEM rather informally. The results are passed as proposals to

ISO, generally through the technical committee TC 165. It is essentially a matter of testing standards, but complicated by the introduction of authoritative requirements on testing conditions based on divergent opinions and code philosophies in different countries. Thus, the selection of material for the testing, as well as methods for deriving and transforming of characteristic values from the test results, have been discussed in detail. A proposal for selection of wood for the purpose of testing mechanical joints was accepted as a CIB/RILEM Timber Standard already in 1976. This standard has been applied in Sweden since then.

Otherwise, the matter of sampling for testing of components and structures has not been carried to a satisfactory solution. Possibly, an international agreement will have to be restricted to general principles.

17-17-1 I Smith, L R J Whale
Sampling of wood for joint tests on the basis of density

Introduction

In this paper are described two methods used by TRADA to sample wood for mechanical joint specimens on the basis of density. These methods are employed in the following circumstances:

Method 1

It is required to produce a large number of sets of nominally identical specimens matched on the basis of density and it is not possible to cut a replicate for each set from a common piece of wood.

Method 2

It is required to produce sets of different types of joints matched on the basis of density and it is not possible to cut a replicate for each set from a common piece of wood or the sets have different numbers of replicates.

It is possible to use method 2 to select wood for specimens in a single set to match a pre-specified density distribution.

The objective of any density matching procedure is to produce identical distributions of specimen density for each set. It is not sufficient to merely select specimens so that sets have identical means and standard deviations for specimen density. As an illustration Table 1 shows an example of incorrectly matched sets of specimens. In what follows it is shown how matched density distributions between sets have been attained.

Incorrectly matched density distributions. TABLE 1.

Density values (kg/m^3)

Replicate	Set 1	Set 2	Set 3
1	375.0	395.4	414.6
2	425.0	395.4	414.6
3	475.0	500.0	414.6
4	525.0	500.0	585.4
5	575.0	604.6	585.4
6	625.0	604.6	585.4
Mean	500.0	500.0	500.0
S.D.	85.4	85.4	85.4

18-17-1 P Glos
Sampling of timber in structural sizes

Scope

The mechanical properties of structural timber strongly depend upon natural growth characteristics and manufacturing practices. Under otherwise same conditions (species, grade, size, moisture content etc.) timber may exhibit a substantial variation in properties. From this follows that test results are strongly influenced by the respective sampling procedure and that the method of choosing a sample is an all-important factor in determining what use can be made of it. If a generalization is to be made from the results of an empirical investigation to a specific population, it is a prerequisite that the test material sampled is representative for that population.

In the field of timber engineering various questions can arise, that presume different test material selection procedures. Among these are e.g.:

- Evaluation of characteristic mechanical properties related to a population defined in space and time. This requires sampling of specimens that are representative for the defined population.
- Evaluation of the effect of specific treatments on characteristic mechanical properties.This may require so-called matched sub samples, that are selected from a given sample so that they have the same distribution of mechanical properties.
- Evaluation of mechanical properties of timber structures and of joints made with mechanical fasteners.

In general this requires selecting timber whose growth characteristics vary only in predetermined narrow limits.

In the following, sampling or selection procedures are described for the aforementioned points under discussion.

These sampling procedures are generally applicable. They do not depend upon the planned sample size nor on whether all elements of the sample or, as in the so-called In-Grade Testing Programs, only a part of the sample will be destructively tested.

Decisions on sample size and number of elements tested destructively are governed by the financial scope of the program, the chosen degree of precision with which the properties are to be estimated and the chosen test and data analysis procedures. Some relations between sample size and the precision of the statistical inference are shown as general information.

19-17-1 R H Leicester
Load factors for proof and prototype testing

Summary

Load factors for proof and prototype testing are derived by calibration with existing structural design codes. This calibration is done on an element by element basis, even though the testing is in general applied to multiple member structures.

The load factors take into account uncertainties of loads and strength, duration of load effects and the test configuration.

Concluding comment

The procedure developed herein is based on selecting a test load in the format given by equation (1).

To do this, each structural element in turn is examined and the appropriate load factors k_c, k_D and k_U derived; the largest multiple factor k_C, k_D, k_U obtained is the appropriate one to be used in equation (1). For structural acceptance, the test load must be carried without distress by a proof tested structure or by every tested structure in a sample of prototypes.

It is outside the scope of this paper to discuss the use of proof and prototype testing as a basis for acceptance. However it may be worth mentioning that in general the use of a pure prototype testing procedure is inefficient when applied to timber structures. This is because the high variability of some timber elements, together with the small sample sizes tested means that a large margin of safety must be used to allow for the fact that the structures tested may be stronger than average.

Usually a prototype test of a timber structure is undertaken when there is some uncertainty concerning a particular failure mode, such as that associated with a complex joint. If this is the case, then it is acceptable to consider only the load factors relevant to that failure mode or to reinforce the structure in such a way that failure can occur only in the structural mode under investigation; in such an event, the acceptance of the remainder of the structure must be based on conventional design computation procedures.

19-6-2 R H Leicester
Confidence in estimates of characteristic values

Abstract

This paper describes three algorithms that may be used to estimate five percentile values with 75 percent confidence. A method for assessing the effectiveness of estimating algorithms is described. The suitability of using a 75 percent confidence level is discussed.

Conclusions

A method has been proposed for comparing the effectiveness of estimating algorithms, and has been applied to three such algorithms. The use of a very simple estimator algorithm was found to give reasonable results and hence in practice the use of more complex algorithms may not be warranted.

The significance of the choice of confidence level in sampling was examined through the application of simple reliability concepts; this indicated that the use of a 75 percent confidence level rather than the mean value is to be recommended.

21-6-1 R H Leicester
Draft Australian Standard: Methods for evaluation of strength and stiffness of graded timber

Preface

The following is based on a draft Australian Standard, but modified in terms of presentation and with the emphasis on the assessment of characteristic values for limit states codes. Over the past five years the draft Standard has been used to evaluate the design properties of a variety of timber products including sawn timber (both hardwoods and softwoods), laminated veneer lumber, plywood and scrimber. In all cases the results obtained have been considered by the timber industry to be reasonable and acceptable.

21-6-2 A R Fewell, P Glos
The determination of characteristic strength values for stress grades of structural timber.

Introduction

Background

With the introduction of the timber design code EC5, which will have authority throughout the European Economic Community and is aimed at reducing barriers to trade, it is important that international agreement can be reached on a standardised method for determining characteristic strength values for stress graded timber. Indeed the authors would argue that it is more important than producing an international standard on methods of stress grading; something which CEN is already seeking to ensure. The reason for the authors view is that EC5 uses a strength class system which could incorporate any number of national grades. Different countries have developed their own grading systems which suit their species, market conditions and structural demands and it is unlikely that one grading system would find favour with all of them. The aim should therefore be to assign the various grading systems to the strength classes, and for this a standardised method for deriving characteristic values and a standard for quality control of graded timber are essential. Characteristic strength value is defined in the Eurocode as a population fifth percentile value. These values are dependent on sampling, test methods and analytical procedures.

This paper is in two parts. Part 1 discusses the necessary procedural steps and points out some of the effects resulting from the various decisions that can be reached. Part 2 proposes set of rules that could form the basis of a standard.

The authors have endeavoured to ensure that as much existing test data as possible will be acceptable and that minor species are not neglected by the need for large and expensive test programmes. It is also important that timber is not made less competitive as a structural material by adopting conservative approaches at each step in the procedure that would accumulate to an unacceptable degree.

Whilst various grade/species combinations with different strength properties can be incorporated in a strength class system without complicating the code, other values given by the code such as factors for moisture content adjustments, size effects and duration of load need to remain constant for grades and species to enable interchangeability within a strength class.

Special considerations with respect to machine graded timber,

It is well known that machine graded timber has higher yields and lower variability between the fifth percentile values for different samples, than visually graded timber. This last effect gives rise to the lower bending stress characteristic values and partial factor for machine grades in EC5 and is illustrated in Figure 1 which is taken from Fewell. The figure shows the distributions of sample fifth percentiles for 20 sub-samples of 100, 200 and 300 pieces randomly selected from a parent sample of 652 pieces of European redwood/whitewood. The distributions are shown for both visual grading and machine grading with the modulus of elasticity limit selected to give approximately the same yield as the visual grade.

In addition there is a fundamental difference between the two grading systems that affects the approach for deriving characteristic values. For a visual grade you can sample the graded timber, follow procedures for testing and analysis and determine a characteristic stress. For machine grades (in a 'machine controlled' system such as used in Europe) you start with a characteristic stress, (and it makes sense to use the strength class values) and determine the appropriate machine settings. This is necessary because the settings need to vary for grade, species and size and so a mathematical model relating all these variables to strength needs to be determined. The test samples therefore need to be larger and ungraded. Where the North American 'output controlled' machine grading system is used, the test samples can be smaller and the model simpler because the settings are refined during production. That system is more appropriate for sawmills operating long production runs for a very limited number of grades and sizes. You can of course test samples of machine graded timber to verify the settings in a 'machine controlled' system. Whilst this paper gives guidance on the requirements of a machine grading system it is intended that the topics of the approval, control and setting of grading machines should be the subject of another paper, owing to the size of the subject.

Conclusions
With a strength class system, as in Eurocode 5, in which many grading systems can be incorporated, it is more important to have a standard method of deriving characteristic values than to have an international standard for stress grading.

This paper discusses the criteria and conditions that affect the determination of characteristic values. Proposals are made for procedures which are intended to serve as the basis for an international standard.

24-17-1 R H Leicester, F G Young
Use of small samples for in-service strength measurement

Introduction
In-grade test measurements intended to assess in-service strengths are used to define reliable structural strength. For this reason the availability of these in-grade measurements are an important component of effective timber engineering design. Unfortunately however, these in-grade measurements can be prohibitively expensive to undertake. For strength measurements large samples are often needed because the aim is to evaluate the five percentile in-service strength with a certain degree of confidence, usually a 75 per cent confidence. Even within a single species there are many grades and sizes to evaluate for each property; particularly in the case of large timber sizes, the cost of both the material and laboratory tests can be very costly.

For these reasons, it can often be desirable to attempt a compromise in which the testing of limited samples is used to provide a conservative but useful estimate of the in-service characteristic values. The following describes a method for this which is particularly suitable for use with softwood timbers; here it is assumed that these timbers can be modelled as a set of defects, randomly distributed within a matrix of clear wood. Two procedures will be discussed; one is related to the evaluation of in-service bending strength and the other is for use in quality control operations.

Conclusion
Exact evaluation of in-situ structural properties can be assessed only by simulated in-service measurements. However, such evaluations are expensive, and it is proposed that for practical purposes many of these evaluations can be replaced by good estimates based on making use of secondary parameters such as the visual appearance of a defect and the magnitudes of the spacings between the defects.

Some rough analyses were used to estimate the efficiency of applying the proposed procedures to the bending strength of F5 radiata pine. In the case of in-service strength evaluations with a specified accuracy, the use of defect biased strength measurements led to a reduction in the required sample size by a factor of about 3. In the case of quality control operations, the number of test specimens required could be reduced by a factor of about 4 through the use of defect-edge biased specimens and a factor of about 15 through the use of defect-edge-stick biased specimens.

It should be noted that the use of biased specimens is often recommended in existing testing standards, but usually the rules of such standards do not correctly take into account or take useful advantage of such bias.

24-17-2 R H Leicester, F G Young
Equivalence of characteristic values

Introduction
Methods used for defining characteristic values often differ from country to country. For purposes of trading in structural timber products, it is obviously important to be able to establish the equivalence between these various definitions of characteristic values.

However, there is another equally important reason for being able to establish equivalences. This is because it is essential to do this to be able to transfer technology between countries. Many of the parameters affecting the structural performance of timber components are stated in terms of their effect on characteristic values; typical examples are the influence of load-duration, moisture and size effects. Thus, a change in the definition of the characteristic value will usually require a change in the predicted effects of these parameters.

There are at least two major types of differences in the way in which characteristic values are defined. These are as follows;

(a) the structural properties are measured in different ways, and

(b)the test data is processed in different ways.

The processing of the data differs with respect to choice of percentile, statistical confidence limits, and factors related to the design code format. Examples of factors related to the design code format include the partial factors for material resistance and the modification factors for size which are specified in timber engineering design codes that use the characteristic values; in data processing due account must be taken of these factors, even though they are not intrinsic properties of the timber material.

To some extent, the differences in characteristic values arising from differences between methods of data processing are an irritant rather than a barrier, because methods for evaluating these differences can be easily developed or the data can be reprocessed. However, differences in characteristic values that arise because of differences in test methods are usually either difficult or impossible to evaluate. For example, in recent times the USA and Canada have spent well over US $ 10 million for in-grade measurements on their structural timbers; yet because of the test procedures used, to date no satisfactory method has been found to process this data to evaluate characteristic values of structural properties that are acceptable to the EC countries.

The following gives a discussion on the influence of test methods used for evaluating the structural properties of sawn timber in bending, followed by some brief comments on the evaluation of characteristic properties for joint system.

Conclusions

For solid timber, the methods used for establishing the characteristic values of structural properties differ from country to country. This creates a barrier both to the trade of structural timber and a barrier to the transfer of technology from one country to another. There is some potential for developing an acceptable procedure for establishing equivalences between the various methods used for establishing characteristic values in particular grades and species.

For the case of connectors, the complexity of jointing systems used would appear to inhibit the development of methods for evaluating the equivalences between characteristic values. This could be a major barrier to technology transfer, particularly with respect to new connector systems. An effective short term action would be to endeavour to obtain limited international harmonisation on standardised joint configurations and on theories of connector actions.

24-17-3 Y H Chui, R Turner, I Smith
Effect of sampling size on accuracy of characteristic values of machine grades

Introduction

Machine stress grading of lumber was first suggested over three decades ago. To facilitate commercial production of machine stress graded lumber, two quality control systems have evolved: output and machine control systems. Output control was first developed in North America and it is currently the only approved quality control system in the USA and Canada. Commercial production of machine stress graded lumber in Europe is confined to a few countries: the UK and the Nordic countries. In those countries machine control system is used exclusively, although both systems are listed in British Standard BS4978 (BSI, 1988) and in the draft European standard on machine stress grading of timber (CEN, 1991).

Under the North American output control system, machine grades with a broad range of primary design properties (bending stress, bending modulus and tensile stress) can be produced as long as quality control data show the material to be "on grade". Individual sawmills are required to have online equipment and qualified personnel to conduct tests on selected samples of the graded material. The so-called CUSUM charts are provided by the grading authorities to assist with determining whether properties of the graded material agree with the target design specifications ie "in control" (NLGA, 1987). If the process is "out of control", machine settings are readjusted and further quality control samples are taken and tested. This process is repeated until the process is "in control". Adjustment of settings to maximize yields is also permissible.

The machine control system places more emphasis on the characteristics of the grading machines and a pre-determined relationship between strength and flatwise E_{min} measured by a machine. In contrast to the output control system only those grades listed in design standards are produced. Machine settings (E_{min} boundaries) for sorting material into these stress grades are given by the grading authorities for each combination of species, size of lumber and type of machine. These settings are derived from strength/E_{min} relationships which have been established by testing representative samples of lumber prior to the actual production of the machine graded material. In general no in-house quality control tests are required under this quality control system as the process assumes that the sampled material adequately represents those subsequently graded and all machines of the same type have essentially identical performance at the time of manufacture. Periodic verification by the grading authorities ensures that machines perform consistently.

The above outlines the main differences between the two quality control systems for production of machine graded lumber. As mentioned earlier, the establishment of the strength/E_{min} relationship, and thus sampling of material for the calibration exercises, is central to the reliability of the machine control system. This paper examines how the characteristic values of samples drawn randomly from a typical "parent" population deviate from the target values when sample size changes.

Conclusions

1. Sample sizes to meet certain tolerance and confidence interval requirements for characteristic values of machine grades vary depending on the methods of determining the characteristic values. Required sample sizes estimated using non-parametric approaches are considerably higher than a parametric method based on a distribution functions transformable into bivariate normal distributions.
2. Based on the use of a typical data set of 972 pieces of softwood lumber as the "parent population" and a parametric approach, it was shown that suitable sample sizes for determining machine settings are in the order of a few hundred. The likely range is between 200 and 350 pieces depending on lumber species and the combination of grades to be sorted simultaneously.

24-17-4 R H Leicester
Harmonisation of LSD codes

Introduction

During the recent NATO 'Advanced Research Workshop' on 'Reliability Based Design of Engineered Wood Structures' (Florence, Italy, 3-5 June, 1991), it was generally agreed that, while the global trend towards use of the LSD format was encouraging, the lack of harmonisation between the various countries was a disturbing feature. In the short term, lack of harmonisation would be detrimental to technology transfer between countries (thereby necessitating the involvement of unnecessary costs and delays) and it would also act as a barrier to trade in structural products. In the longer term, the lack of harmonisation could reduce timber to the status of a regional or local commodity rather than a global one, thereby leading to a reduction or perhaps even a total loss in the use of timber as a primary structural material.

In the following, examples of the current lack of harmonisation will be discussed. Some of these are critical while others can be circumvented, although usually at the cost of adding unnecessary complexity and delays to implementing efficient timber utilisation. However, it should be emphasised that in this context harmonisation is not meant to imply uniformity; it does however imply compatibility in the sense that the equivalence between various codes may be easily established. Codes of differing complexity can be tolerated provided there is harmony of intent and compatibility of format between them. This is particularly relevant when codes of developing and developed countries are being compared.

Concluding comment

Global harmonisation in the sense of establishing the equivalences between various design codes is a critical matter in terms of trade, technology transfer and the acceptance of timber as a structural material. Currently there is a lack of harmonisation between the design codes of various countries. For some important matters, such as characteristic properties, it would appear that harmonisation may not be achievable unless concerted international effort is made to do so.

25-17-3 D W Green, J W Evans
Moisture content adjustment procedures for engineering standards

Introduction

Over the last 10 years, a considerable amount of work has been conducted at, or in cooperation with, the U.S. Forest Products Laboratory (FPL) on the effect of moisture content (MC) on the mechanical properties of standard 38-mm (nominal 2-inch) thick structural lumber (Green and Evans, 1989). These studies have shown that the change in properties with the change in MC is a function of lumber quality and that strength does not necessarily increase with decreasing MC. These studies have produced new analytical models for relating lumber properties to change in MC (Evans, et.al. 1986). Additional simplified models for incorporating this information into engineering design standards have been recently approved in the U.S. (ASTM, D1990) and replace analytical models given in ASTM D245. The FPL research, as well as research by Madsen (1975, 1982) and Hoffmeyer (1978), was used to establish proposed MC-property adjustment recommendations for Eurocode 5. The research on which the U.S. models for adjusting bending properties are based utilize test results from only two species: Douglas-fir and Southern Pine (McLain. et. al., 1984, and Aplin, et al., 1986). To evaluate the applicability of the D1990 simplified adjustment procedure, and the proposed Eurocode 5 procedure, to other species a limited study was conducted on MC-bending property relationships for 38- by 89-mm (nominal 2-by 4-in.) lumber of five additional species (Green and Evans, 1991).

The objective of this paper is to evaluate analytical models for describing the effect of change in MC on the bending properties nominal 2x4 dimension lumber for five additional species. This work is limited to models applicable to engineering design codes, and to MC's of 10-percent or more.

Conclusions

From the results of this study we conclude the following.

1. For adjusting modulus of rupture for change in MC, any of the better models provides a better property estimate than taking no adjustment. The adoption of some MC-MOR adjustment model would be especially important for the higher stress class levels.
2. The simple linear model of ASTM D1990 is recommended. Its use should be restricted to MC's above about 10. The results of this study suggest that the upper limit of the adjustment be the values determined in this paper, or 2% for untested species.
3. For modulus of elasticity either the constant percentage model of ASTM D1990, or a constant percentage model of the type used to compute compound interest be adopted . The value should be based on those chosen for MOR.
4. There is a need to go back and re-evaluate the full Douglas-fir and Southern Pine MC-MOR data sets using the knowledge gained in this study. This work is in progress.

27-17-1 R H Leicester, H O Breitinger
Statistical control of timber strength

Introduction

In comparison with most other structural materials, the strength of stress-graded timber is highly variable. Because of this, timber strength is defined in statistical terms. Typically, it is defined as the five-percentile value of a stress-graded population.

In order to ensure reasonable confidence in obtaining the specified strength of this highly variable material, frequent strength measurements are made as part of quality control systems. In drafting Australian Standards, the following types of quality control measurements are under consideration:

– in-grade evaluation: a complete evaluation of all significant structural properties for all grades and sizes;
– annual check: an in-grade evaluation of a limited set of properties to ensure that there has not been a drift in the quality of the output of a mill;
– daily check: a special structural test on a small sample to ensure that the stress-grading process is in control;
– random check: a limited set of in-grade tests by a customer who may suspect that the quality of timber is below the quality claimed by the producer.

The relative frequencies of these four types of measurements are illustrated in Figure 1. In the above, the terminology 'annual check' and 'daily check' are used as descriptors to provide a rough guide as to the frequency requirements of this type of check. The actual frequency of checks of any type should be based on sampling at a rate per m3 of material processed as specified by a quality control scheme.

The criteria for quality control tests should be simple. The use of complex rules only serves to obscure the impact of the rules; furthermore, the implied accuracy is not justified because of the lack of adequate knowledge of the characteristics of the tails of strength distributions.

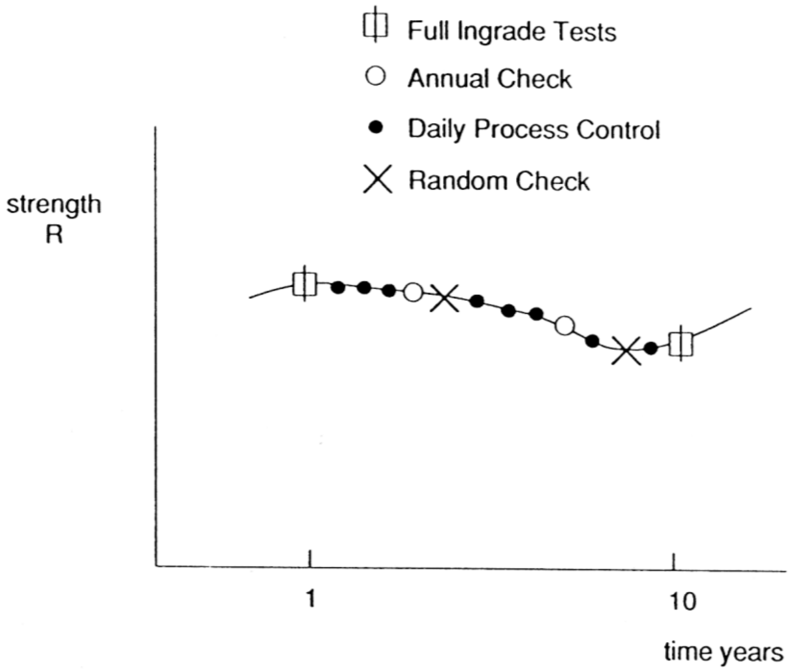


Figure 1. Schematic illustration of frequency of quality control checks.

A more effective method of assessment of a quality control criterion is to test it with Monte Carlo simulation; the statistical distributions used in these simulations should cover the range of distributions found in structural timber. For practical purposes it is reasonable to use a range of distributions bracketed by a Weibull and log normal distribution, and a coefficient of variation between 0.1 and 0.5 as shown in Figure 2.

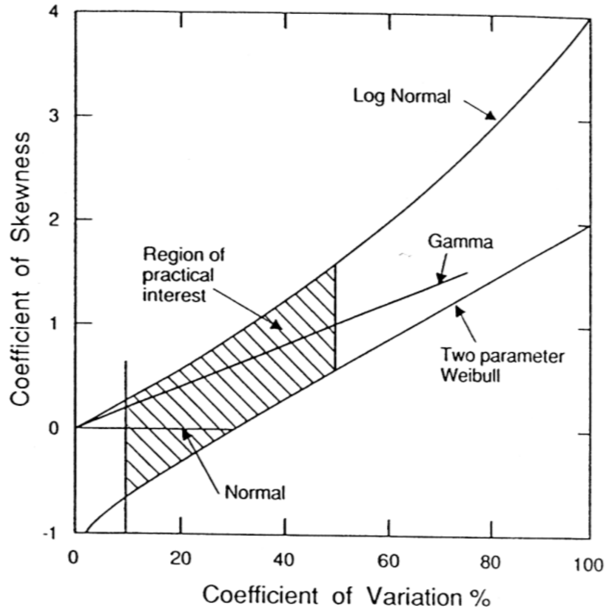


Figure 2.

30-17-1 F Rouger
A new statistical method for the establishment of machine settings

Abstract.
When verifying settings, a major concern is to quantify the effects of down- or up-grading, and give limits on these errors. Because we never know the correct strength class for a piece, which is only defined by fractiles, the first step is to assign a strength class regardless of the grading method. Once this target strength class is given for each piece, the result of the grading process is compared to the target value. For each error which occurs in this process, an elementary cost is assigned. This cost is based on safety concerns for upgrading and on economic losses for down-grading. It depends on the set of strength classes to be produced.

Another part of this paper is dedicated to verification of settings. Rather than making a separate sampling for verification, a crossed-validation procedure is proposed. This procedure aims at reducing the sampling, so that the sample can be used simultaneously for the establishment and verifica-

tion of settings. Since this procedure is repeated several times on different parts of the sample, one can ensure that a piece is never used for both.

By using the cost assignment method and the crossed-validation procedure, a global cost matrix is formulated. It can subsequently be used for comparison to limiting values, the result being an agreement for a specific grading machine producing specific strength classes. Some examples illustrate the method.

Conclusions

The method described in this paper presents two innovative aspects. Firstly, an estimate for the errors of classification has been formulated, based on safety and economic considerations. Secondly, a procedure for checking machine settings and repeatability has been proposed. This method presents the main advantage of replacing experimental work by computing work, which is more affordable. The limiting values corresponding to the requirements are still under investigation, but should be incorporated into a new version of EN 519 in a close future.

35-17-1 J Köhler, S Svenson
Probabilistic modelling of duration of load effects in timber structures

Missing in proceedings

38-17-1 R Steiger, J Köhler
Analysis of censored data - Examples in timber engineering research

Introduction

In timber engineering research one is often confronted with situations where test results enclose not only quantitative data e. g. failure loads but also accompanying information. Examples of such situations are:

– modelling mechanical properties with special emphasis to the lower tail of the underlying probability distribution.
– proof loaded specimens with strength higher than the (fixed) threshold,
– "run-outs" in the course of fatigue or duration of toad tests, i.e. endurances reached without failure of the specimen, either because the test was terminated at a predetermined limit or because some other part of the specimen failed.
– tensile tests on timber structural joints (glued-in rods as an example) carried out on symmetrical specimens with identical joints at both ends.
– different failure modes within the same test series (e. g. shear and bending failures)

Resulting data in the above mentioned cases is said to be "censored". There are four possible ways to handle censored data:

(i) treating all run-outs as though they were (mode-conform) failure points,
(ii) neglecting the run-outs,
(iii) a graphical analysis (averaging) of all test results within the scatter band,
(iv) the Maximum Likelihood Estimation Technique.

The Maximum Likelihood Method (MLM) was successfully applied to the analysis of fatigue test results by Edwards and Pacheco (1984) as well as by Spindel and Haibach (1979).

Making reference to Lawless (1982) Yeh and Williamson (2001) conducted a censored data analysis to combine shear strength test values of glulam beams with frequently occurring bending failure.

Köhler and Faber applied MLM for the probabilistic modelling of graded timber material properties. The authors introduced Censored Maximum Likelihood Estimation as a means for estimating the parameters probability density functions (PDF) of timber properties with special emphasis on the lower tail domain and the statistical uncertainty associated with this: only observations in the lower tail domain i.e. below a given predefined threshold value are used explicitly. The other observations are only made use of implicitly to the extent that it is recognized that they exceed the threshold.

Van de Kuilen and Blass (2005) used a similar method reported by Douwen et al (1982) to correct the mean value and the standard deviation when calculating the characteristic values of shear strength derived from tests on beams with two spans, thus accounting for the fact that actually the lower of two possible test values was found.

The paper discusses the influence of regarding/disregarding the additional information provided by the run-outs on the quantification of the parameters of corresponding probability distribution functions by making use of a Censored Maximum Likelihood Estimation Technique. Beside numerical studies of generated samples, as an example calculations are performed on a data set derived from pull-out strength tests of axially loaded steel rods bonded in glulam parallel to the grain and connected to the tension test machine by identical interfaces at both ends of the specimen (Fig. 1, left). In comparison to unsymmetrical configurations (Fig. 2, right) this so-called pull-pull set up has the advantage of being simple when producing the specimens and regarding the transfer of forces from the testing machine into the specimen (Fig. 2).

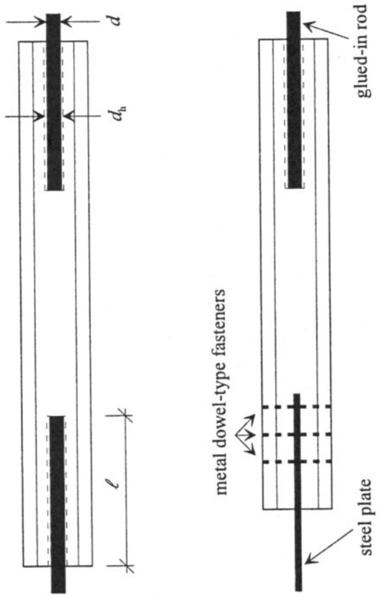


Fig. 1: Symmetrical and unsymmetrical specimen for pull-out strength tests on glued-in rods. (The dowel joint has to be stronger then the rod joint!)

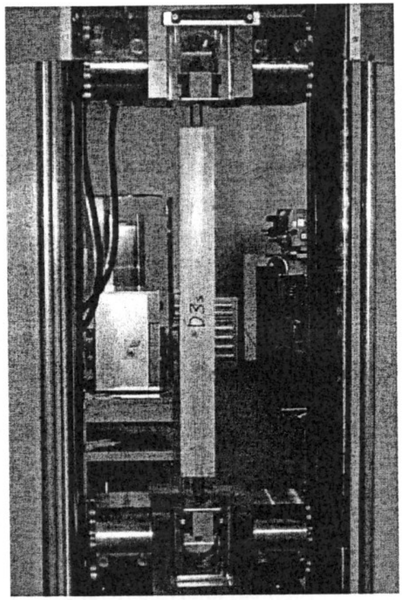


Fig. 2: Pull-pull configured tensile test on steel rods glued in glulam parallel to the grain.

The failure loads derived from tests in pull-pull configuration are censored for the following typical reasons:

(i) For symmetrical test set ups (Fig. 1, left) a sample consists of the strength values of the "weaker" side of each of the n specimens but implicitly carries the information that n sides survived the test. Ie. it is also measured that the strength of these survivors is higher than the strength of the opposite joint, which failed.

(ii) For both symmetrical and unsymmetrical test set ups (Fig 1) different failure modes can occur. When optimizing the capacity of a glued-in rod joint, timber shear failure next to the rod (Fig. 3) is aimed. But sometimes steel failures (Fig. 4) or timber tensile failures (Fig. 5) occur. When estimating the probability distribution function of the shear failure data, the measured steel or timber tensile strength has to be considered as exceeding the shear strength.

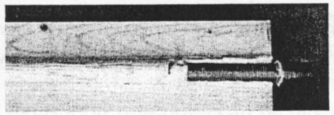


Fig. 3: Glued-in rod pulled out (shear failure in timber.)

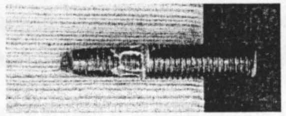


Fig. 4: Steel failure of a glued-in rod.

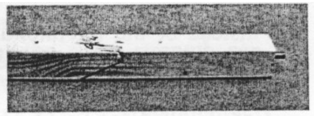


Fig. 5: Timber tensile failure.

39-17-1 I Smith, A Asiz, M Snow, Y H Chuff
Possible Canadian/ISO approach to deriving design values from test data

Abstract

This paper proposes principles for a standard practice for deriving factored design resistances of structural components or subsystems directly from test data. This applies to, for example, members, connections, trusses and shear-walls. The approach is intended as a fully consistent alternative to practices embodied in existing design codes. It is assumed that parent design standards (national loading and timber design codes) are based on Load and Resistance Factor Design (LRFD) concepts. The likely level of consistency between design solutions based on testing evidence and those based on existing LRFD codes is evaluated based on examples applicable to Canadian wood products.

Conclusions

The approach proposed here for determining design properties for structural components and subsystems directly from test data is very simple, yet it appears to provide good consistency with provisions of existing code rules. Although created within the context of design practices in Canada it is believed that what is suggested is also the suitable basis for creating an ISO standard for data evaluation. A key concept embedded within the proposed approach is that the reliability index used within any analysis of test data be related to classification of the failure mode and the end use situation. Classification of failure modes would be according to a brittle-ductile scale. Definition of the intended end use would distinguish between whether a parent structural system incorporating components or subsystems of the type investigated is of a type capable to developing an alternative load path were any component or subsystem to fail. This dovetails with suggestions in a companion CIB-W 18 meeting paper on Systems Level Design.

2.9 STRENGTH GRADING

21-5-1 T Nakai
Non-destructive test by frequency of full size timber for grading

Introduction
The main purpose of this experimental work is to tap the possibility of using the frequency measurement on full size timber for grading. There are many ways for estimating strength of timber through non-destructive parameters. One of the most usuful parameters is modulus of elasticity, which could be also obtained by various methods too.

In searching of the more feasible method of measuring modulus of elasticity at saw mill or lumber yard, the method to measure the fundamental vibration frequency of timber was investigated. It was concerned about the effect of moisture content on density, which is one of the essential items to apply this method.

At the same time, another three methods to obtain the static modulus of elasticity and dynamic one were selected for comparison. These methods were deflection, ultra sonic propagation time and stress wave propagation-time measuring.

The frequency measurement has merit of easy handling, because non-contact measuring is possible using relatively reasonable price equipment.

The reason why this work has conducted reflects the current Japanese timber construction tendency, which is directing to large scale building. Supply of well stress graded timber with reliable strength is the key to keep and promote this trend. Mainly due to the larger cross section of Japanese timber than that of dimension lumber which is used world wide, it is impossible to use today's commercial grading machine, that has normally the limitation of the maximum height of timber to 75 mm.

Conclusion
It was concluded that the fundamental vibration frequency measurement had high possibility to obtain modulus of elasticity, which could be used for stress grading, at saw mill or lumber yard.

Frequency modulus of elasticity could be obtained in green or air-dried moisture condition with the apparent density at test and the fundamental vibration frequency.
Frequency modulus of elasticity could be measured on log and large scale glue-laminated timber, for which it is usually hard to measure.

Among the four moduli of elasticity measured by deflection, ultrasonic propagation time, stress wave propagation time and fundamental vibration frequency, little difference was observed for the purpose of grading timber.

21-6-2 A R Fewell, P Glos
The determination of characteristic strength values for stress grades of structural timber.

See 2.8 Statistics

22-5-1 T Nakai, T Tanaka, H Nagao
Fundamental vibration frequency as a parameter for grading sawn timber

Introduction
Due to large cross section of structural timber in Japan, non-destructive tests for measuring modulus of elasticity to estimate the strength of timber have been conducted at Forestry and Forest Products Research Institute. Various non-destructive tests such as measuring deflection caused by dead load, measuring ultra sonic propagation time and density, stress wave propagation time and density and fundamental vibration frequency and density, have been tried to find out the most feasible method at saw mill or lumber yard.

Measuring fundamental vibration frequency and density was considered as one of the favourable non-destructive testing methods, as it is a non-contact measuring method and can be used for a large size timber or log.

This paper presents some extended results obtained under a three year project at FFPRI following the previous **Paper 21-5-1**. Main targets are to get the relationship between modulus of rupture and Efr, the modulus of elasticity obtained by measuring fundamental vibration frequency and density, tensile strength and E_{fr} and compressive strength and E_{fr} for grading sawn timber. And also when measuring fundamental vibration frequency, the effect of length divided by the height of specimen was experimentally investigated mainly to know the minimum ratio of length by height for measuring the frequency of short column specimen.

Conclusion
It was concluded that the fundamental vibration frequency measurement could be used as a useful parameter for grading sawn timber to estimate not only modulus of rupture, but also tensile and compressive strength. According to the experimental results, for measuring the fundamental vibration frequency, the ratio of length by height of specimen should be larger, than six.

26-5-1 F Rouger, C De Lafond, A El Quadrani
Structural properties of French grown timber according to various grading methods

Abstract
Since 1983, CTBA has carried out a wide research program on strength properties of French grown species. This program, not only led to the establishment of a new visual grading system, but also gave a large amount of data to be analysed. One of the original ideas of this program is a forest based sampling. Trees are sampled in the producing regions according to the weight of each region. Boards are sampled in order to represent the variations of properties along the tree and through the cross-section. Knots measurements and physical properties are correlated to strength properties in order to evaluate the influence of sylvicultural methods, but also to establish a new visual grading system. This program has been performed on five main species (Whitewood, Douglas Fir, Black Pine, Maritime Pine and Poplar). More than ten thousand boards were tested. This paper is aimed to summarize the basic properties of the resource, and to focus on grading results. The world wide tendency of machine grading is also taken into account to analyze the data. The last part of the paper concerns depth effects, which have been already investigated by many authors.
Introduction
Since 1940, a huge amount of forest has been planted in France in order to increase the resource. New species as well as imported species have been implemented. Their properties were known only from small clear specimens. For this reason, CTBA decided in 1983 with the producers and with the end-users to study this material, with two objectives:

(1) Investigate the properties of the resource for structural and non structual uses and give advice on new sylvicultural programs.
(2) Establish a grading system, based on full size tests, which could be used to revise the French Grading standard of 1946.

These studies have produced a large amount of data which is still being analysed. For each board, different measurements have been performed :
- location of the board within the tree,
- location of the tree within the forest,
- knots measurements (face knots, edge knots, KAR, volumes),
- rate of growth,
- density of the board,
- MOE, MOR in bending,
- MOR in tension,
- small clear specimens tests (tension perp., shear, tension parallel,...).

These measurements are entered in a database to carry out statistical analysis. The results of these analyses have been published in research reports available at CTBA, but not yet in technical meetings. This paper will only describe the grading results, in accordance with the CEN work on strength properties of structural timber.

Conclusion
This paper focused on the relationship between structural properties and grading methods. Three major ideas were discussed. Firstly, an optimal grading method was developed to get an estimate of the potential of the resource. Secondly, it was demonstrated that the MOE and density profiles were strongly dependent on the grading technique, and could not always be attributed to the material behaviour. Lastly, an investigation of the size effect exhibited a "grading effect" which was much larger than the material depth effect. This last point confirms some of the ideas developed previously and can be discussed. Of course, much research needs to be done to improve the grading methods and optimize the resource with respect to its structural properties. A program is initiated to develop stochastic finite element analysis of full size specimens and to try to understand the behaviour of such a composite material. This research should lead to accurate predictors of the strength which could be incorporated in grading methods. A worldwide cooperation is wished by the authors to work on this aspect.

28-5-1 S Ohlsson
Grading methods for structural timber - principles for approval

Introduction

Selection of timber has a long tradition. Craftsmen have practiced different methods of timber selection for ages. Trees with suitable properties for certain applications in shipbuilding have for instance been identified in the forest, and subsequently logged and used for the intended purpose. Such a procedure may in modern terms be considered as a selection process based on quality criteria and executed by visual inspection. The prerequisite for such a procedure is that the primary parameters representing the required qualities either are directly assessable by eyesight or are strongly correlated to the appearance of the growing tree.

At the end of the present century of industrialisation, visual inspection is still the dominating method for grading of sawn timber including timber for structural applications. For several reasons, alternative methods for grading of structural timber are being developed. The aim of this paper is to present some aspects on the means for assessing the merits of such methods.

Concluding Words

Grading of structural timber has been discussed in conjunction with the present European process of standardisation. Attempts to stimulate the development of new grading methods with improved performance require that methods for assessing the merits of such methods are rationally based and free from unnecessary restrictions that may be inherited from present technology for grading machines.

Criteria for grading methods should aim on the ability to assure some of the required properties for each piece of timber with a given degree of statistical certainty and independently of the procedure chosen.

The three different contexts in which timber properties are discussed here are:

- Assessment and possibly certification of grading methods
- Industrial grading of structural timber
- Structural use of strength graded timber

It is essential that the level of detail and accuracy in specifications devoted to these different processes are harmonised. At present, this is not the case. Severe economical and legal consequences may result if the standards that are supposed to specify the different procedures are not brought in better harmony. One type of problem that exists in the present versions of the different draft standards is that both the goal and the means to achieve the goal are prescribed. Such 'double' regulation may often be impossible to obey, thus constituting a legal problem.

An example of this type of mistake is that prEN 338 clearly describes the target for a population of timber in a certain strength class, while prEN 519 also attempts to describe suitable means to reach this target. Two kinds of problem are identified here. The first is that prEN 519 is not restricted to describe methods for assessment of the ability of a grading process to meet the targets of prEN 338 but also specifies some aspects of the prospective methods in detail. The second kind of problem stems from a need to minimise the effort needed for assessment. The harmonisation between the different standards should here include a choice of a common set of parameters to be used for verification independent of the grading method.

28-5-2 N Burger, P Glos
Relationship of moduli of elasticity in tension and in bending of solid timber

Abstract

This paper compares moduli of elasticity determined in edge wise and flat wise bending and in tension. It shows that the modulus of elasticity of full-size structural timber in contrast to small clear specimens depends on the type of load (bending, tension) and specimen orientation (edge wise, flat wise bending) as well as on timber quality (grade effect). On average variations amount to approx. 8 to 9 %.

So far neither EN 384 nor EN 338 indicate how to deal with these differences when assigning structural timber to a specific strength class on the basis of tests other than bending tests.

Conclusions

Summarising the results of the investigations the following conclusions can be drawn:

- The modulus of elasticity of structural timber, in contrast to small clear specimens, depends on the type of load (bending, tension) and specimen orientation (edge wise, flat wise bending).

– The difference between E in edge wise bending acc. to EN 408 and DIN 52186 186 is about 9%. This can be entirely explained by the shear deformation, if a ratio of G/E = 1/30 is assumed.
– The difference between E in edgewise bending and in tension acc. to EN 408 is about 9%. The gauge length for the determination of E in tension was 4b and 4.6b respectively as compared to 5b acc. to EN 408.
– The difference between E in edge wise bending acc. to EN 408 and flat wise bending acc. to DIN 52186 is about 17 %. When shear effects are taken into account assuming a ratio of G/E = 1/30 the difference is about 8 %.
– The differences between E in edgewise bending acc. to EN 408 and DIN 52186 and between E in bending and in tension depend on timber quality.

The results are in contrast to published results obtained in tests with small clear specimens. This may be due to the different behaviour of structural timber and small clear specimens.

Whether these differences should be taken into account in the design of timber structures needs further discussions in a wider context. At any rate these variations must not be disregarded when allocating structural timber into strength classes according to EN 338 on the basis of tension or flat wise bending tests.

29-5-1 T Courchene, F Lam, J D Barrett
The effect of edge knots on the strength of SPF MSR lumber

Abstract
An experimental program has been performed to evaluate the impact of edge knot visual quality level on the tension and bending strengths of 1650f-1.5E 38 x 89 mm Spruce-Pine-Fir machine stress rated lumber. A closed form solution has been developed to establish the strength property probability distribution of a new combined grade containing visual quality level knots. Simulations have been conducted verifying the closed form solution. The results indicate that the amount of visual quality level material to be included in the new grade is important. In this study, the new grade exceeded the bending strength and edgewise modulus of elasticity requirements but failed to meet the tension strength requirement due to the quality and quantity of visual quality level material included in the grade mix.

Conclusions
The determination of knot area ratio showed that many of the knots that were manually measured did not meet the criteria for MSR edge knots. This fact must be addressed by the MSR producing mills to increase lumber grader accuracy in the determination of MSR edge knots.

A closed form solution was developed to assess the strength properties of a new grade of MSR lumber with inclusion of a proportion of large VQL material. The results indicate that the new combined grade exceeds the requirements in bending strength (23.9 MPa) for KAR groups 1 to 5 but fails to meet the tension strength criteria (14.8 MPa) for KAR groups 1 to 5. When considering KAR groups 4 and 5, the new combined grade just fails to meet the tension strength requirements.

The tension strength of the new combined grade is very sensitive to the amount of KAR material included. However, it should be noted that the frequency of large VQL is a function of the raw material and the manufacturing process. The testing program conducted here is in effect a snapshot in time where large VQLs were evaluated during a moment of production. This picture corresponds to a certain timber resource from a particular location with specific growing conditions influencing the resulting lumber strengths and large edge knot frequency. Therefore, if a mill wishes to qualify a larger VQL than currently allowed, representative testing of regional conditions must be performed to assess lumber strength characteristics. Daily quality control procedures, including tension strength proof testing may be needed to ensure product performance.

29-5-2 T D G Canisius, T Isaksson
Determination of moment configuration factors using grading machine readings

Abstract
A method to determine moment configuration factors using currently available data from the Lund University test programme on along-member strength variation in timber beams is presented. The grading machine readings for the beams and the statistical data on weak-section strengths are

used in the study. Although sufficient data are not available, and more research is needed, the method presented here may be considered to provide the required information in a conservative manner. The moment configuration factors determined for some typical loading cases are also presented.

Conclusions

The following conclusions can be made with respect to moment configuration factor (MCF) determination procedure presented in this report.

– The presented procedure is able to provide, for the cases considered, MCFs which may be considered as reasonable and conservative.
– The accuracy of the determined results was assessed through a comparison with the results determined through laboratory tests. As the latter itself possesses some uncertainty due to the limited number of weak sections that could be tested, a comparison of the relative sizes were made, and found to be satisfactory.
– The present method has some shortcomings, mainly due to the unavailability of data. More research, as described in the discussion, is needed to eliminate these unknowns. Until then, when using the presented method for other moment configurations such as those present in continuous beams, it may be prudent to make the MCF results more conservative through some form of artificial reduction.
– Test data are available only with respect to a single species, grade and cross-section of timber. It is necessary to carry out tests with respect to other species, grades and cross-sectional sizes of timber so that the currently derived factors could be confidently used with them.
– The current derivation of MCFs did not consider the possible importance of size effects. This needs to be addressed in the future.
– In order to carry out proper reliability based studies, it is necessary to carry out tests with samples larger than the currently used 131 beams.
– The currently derived MCFs, viz. 1.05 for uniformly distributed load on simply supported beam and 1.20 for a central concentrated load on a similar beam, can be recommended for use in designs until more reliable results become available through further research. The above MCF of 1.05 for uniformly distributed loads on simply supported beams may also be used with continuous beams until the results for them are derived in the future.

30-10-1 P Kuklik
Nondestructive evaluation of wood-based members and structures with the help of modal analysis

Introduction

Non-destructive testing of members and structures is a process during which we investigate the properties of the respective members and structures without serious damage. There is a whole range of non-destructive methods and their accuracy is different.

The paper describes in detail the method of modal analysis which was most frequently used and evolved during its application to thin-flanged beams. For the verification of the modal analysis method thin-flanged beams were tested in parallel by means of statical tests without and up to the attainment of load carrying capacity (Fig. 1.1, 1.2).

Test samples of thin-flanged beams were made of spruce wood and water resistant spruce plywood. The ribs and the reinforcement of thin-flanged beams were connected by nails. The test samples were placed on the supports of the breaking path and they were loaded through distributing steel component of an overall mass of 2.94 kN (Fig. 1.1) by means of a hydraulic press of capacity of 100 kN. The loading force was measured with 1.5 percentage precision.

The thin-flanged beams were first loaded by the so-called test loading Q_t during a period of 72 hours. After unloading they were further loaded up to ultimate load (fracture

The test load was determined according to the relationship:

$$Q = \frac{1+\gamma_{Q,i}}{2} \sum Q_{k,i}$$

where

$Q_{k,i}$ are the characteristic value of variable actions
$\gamma_{Q,j}$ is the partial safety factor of variable actions

The test without reaching the ultimate load capacity concerning thin-flanged beams was evaluated on the basis of the principle of a comparison of the values of permanent deflections (ascertained ideally 72 hours after the removal of the test loading) and the values of total deflections (after 72 hours of application of the test load).

On the basis of long-term experience it is possible during tests without reaching the ultimate load capacity to consider the component as possessing sufficient bearing capacity under the assumption that the deformation ratio (permanent deflection/total deflection) does not exceed the value 0.3. As regards this criterion experience has proved that it can be applied even in other cases. For example, in this way we have tested also frames with screwed comers of hall structures.

Conclusion
It is obvious from the results of experimental testing of thin-flanged beams and their subsequent modelling by means of the Finite Element Method (See section 5) that modal analysis is a convenient method for the verification of computation models.

Modal Analysis has recently proved useful in the determination of characteristic values of the mechanical properties of structural timber and its grading. Up to the present day we have carried out three series of experiments with structural timber taken from three different regions of the Czech Republic. The tests were carried out with samples with the dimensions of 100 mm x 120 mm x 2 800 min and of 100 mm x 150 mm x 2 950 mm. Structural timber first underwent visual grading according to the standard DIN 4074. Further tests were carried out by means of the modal analysis method and the ultrasonic method. Eventually destructive tests were carried out according to the standards EN 408 and EN 384. The purpose of the destructive test was the verification of non-destructive test results. The test results of one series of 43 samples of structural timber, which corresponded to the strength class C 22 according to the standard EN 338, are shown in Figure 6.1.

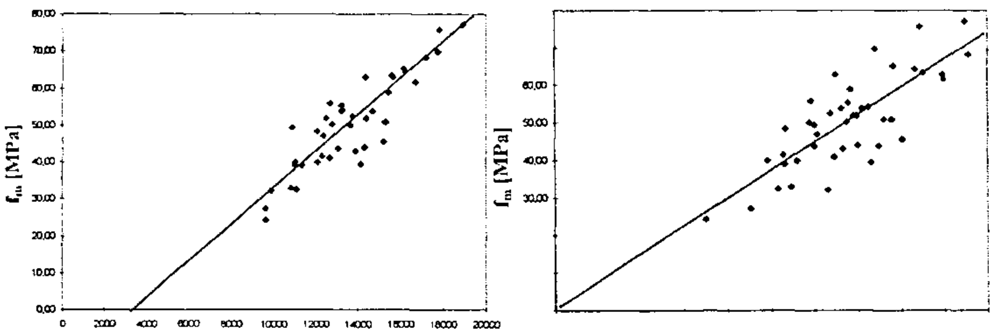


Fig.6.1 Results of experimental testing of structural timber

Another non-destructive method which we have recently applied in the analysis of a frame comes detail in the frame construction of a Prague sports hall, with a span of 50 m, was the photoelastic measuring method.

31-5-1 S Ormarsson, H Peterson, O Dahlblom, K Person
Influence of varying growth characteristics on stiffness grading of structural timber

Introduction
The common practice in investigating stiffness and strength properties of sawn timber is to load the specimens and measure the deflection. The longitudinal modulus of elasticity (MOE) is obtained as some kind of average value, determined on the basis of elementary beam theory. This value is correlated to the strength of the board. The results obtained from measurements are strongly influenced by grain deviations with respect to the longitudinal direction and variation of material properties with the position in the log (often explained by juvenile wood and influence of compression wood). The value obtained from a measurement may therefore be regarded as an "effective modulus of elasticity". In addition to influence from fibre misalignment and property variation, the grading procedure is often disturbed by twist deformations of the board caused by spiral grain.

To improve the stiffness and strength grading process for sawn timber, it is important to clarify how the material properties and the internal structure affect the stiffness properties. In the prediction of timber stiffness, the fibre orientation, growth ring width distribution, juvenile wood and compression wood are of considerable importance. Because of the complex growth characteristics of wood and of various imperfections, the stiffness prediction may require computer simulations based on experimental data.

In the present study, finite element simulations have been performed to investigate how a number of basic parameters primarily affect the stiffness properties and indirectly the strength properties. Some preliminary results are also presented from an experimental investigation of basic material properties of spruce. The properties studied are stiffness and shrinkage parameters and grain deviations. The measurements have been carried out for stems from different stands. The specimens have been sawn at different distances from the pith and at different heights in the stem. The aim is to gain information about the variation of properties with the distance from the pith and with the height in the stem, and also about the influence from growth conditions on the properties examined.

Some concluding remarks

The choice of proper stiffness parameters to be used in structural grading of timber is by no means a simple matter. Usually the coupling effects between tension, bending in two directions and twist are neglected. This may lead to a much too crude approach for grading of high quality timber and in the structural analysis of beam structures.

Due to strongly varying material properties, especially from pith to bark, it is often of great importance for the grading where in the log the sawn board comes from. This is today normally not considered in practice. The sawing pattern influences the positions of the boards to be sawn and might thus have a considerable influence on the strength and stiffness properties of the timber. Another phenomenon that must receive much more attention in grading of timber is the drastic reduction of strength and stiffness caused by compression wood.

Fibre misalignment, like spiral grain deviations, may have a considerable influence on the stiffness properties. It might substantially reduce the stiffness moduli for tension and bending and it must be considered properly in stiffness grading for high quality timber.

31-5-2 R H Leicester, H Breitinger, H Fordham
A comparison of in-grade test procedures

Abstract

The design properties derived for structural timber depend on the in-grade test procedures used to measure these properties. In this paper, the data obtained from a limited set of bending and tension tests undertaken according to European, North American and Australasian procedures is used. It was found that the European procedure can underestimate the in-service bending strength and stiffness and slightly overestimate the in-service tension strength.

Conclusions

The data measured provides useful information on the differences that can be obtained between different in-grade test procedures. It also provides information for calibrating equivalencing models proposed in an earlier paper. The size and grade of timber used in this study probably provides the greatest differences likely to occur in practice.

Additionally, it is of interest to compare the data of the CEN and AS/NZS procedures. The AS/NZS procedure may be considered as an attempt to measure the properties of timber that would occur in-service. By comparison, the human bias applied in selecting the test specimens according to the CEN procedure leads to an underestimate of design bending strength and stiffness but (because it is imperfect) a small overestimate of tension strength.

32-5-1 K Frühwald, A Bernasconi
Actual possibilities of the machine grading of timber

Introduction

In the Austrian and German sawmilling and wood construction industry structural timber is visually graded according to DIN 4074. But visual grading only allows a prediction of the very wide variation of mechanical properties within a piece of lumber or between several pieces. The dimensioning values including relative high safety factors have to be established to reach the lowest strength and stiffness values, which can not be realized by visual grading. Therefore of that only a limited utilisation of the potential timber properties is possible. Additionally visual grading only fulfils partly existing requirements of quality control. Generally the competition between wood and other material in the structural building sector puts structural lumber under high technical and economical pressure. This requires more efficient utilisation of the technical possibilities of timber which is only possible with a reliable prediction of the mechanical properties of timber. This is one reason why research and the industry have been engaged in the further development and practical utilisation of machine strength grading.

The aim of the study "Actual possibilities of machine grading of timber" is the comparison of the various technologies for strength grading for timber in Europe. The efficiency of grading is described as

a) yield in each strength class and
b) their mechanical properties which have to meet the requirements of the international EN- or national DIN-standards.

Conclusions

The possibilities of visual and machine grading can not be estimated by the yield in the strength classes. That requires a correct grading in the different strength classes. Therefore according to EN 338 all "characteristic values of bending strength, density and mean modulus of elasticity (have to be) greater or equal to the values" of the given strength class.

None of the tested grading systems fulfils all in EN 338 resp. the German NAD required characteristic values.

The quality of the tested grading systems (i.e. the fulfillment of the required characteristic values and a least possible classification of the boards in a higher strength class than the potential strength class) depends less on the actual quality of the grading (the quality of separation of the different strength classes and the dispersion of the single strength values in a strength class), but it depends on the yield in the higher strength classes MS13 + MS17.

The higher the machine or visual determined yield in MS13 + MS17, the lower are the characteristic values (above all the strength), and the boards are graded in a higher strength class than the correct strength class according to the tensile resp. bending tests.

The rules for grading and classification of timber should be discussed based on the results of the timber properties in the different strength classes and compared with the requirements of the standards (EN 338 + NAD, DIN 4074 and DIN 1052).

What would have been the results, if all grading machines produced the same yield in the different strength classes?

32-5-2 A Bernasconi, L Boström, B Schacht
Detection of severe timber defects by machine grading

Introduction

The grading of timber in strength classes by machine-grading systems is influenced by the detection of different characteristics or defects of a piece of timber. Typical characteristics are knots, their position and dimension, fibre deviation, bending stiffness, density variation. The prediction of the strength of the timber - as the most important characteristic in grading - is mostly derived from measurements of other parameters, such as the determination of deflection, the determination of the bending force necessary to achieve a deflection, (ultrasonic) wave propagation time, x-ray technique or the determination of vibration frequency.

Different systems of grading machines supply reliable results for normal timber characteristics and are currently used. The setup of the grading system is the result of many experimental tests and is usually given by a correlation between directly measured parameters and the expected strength of each piece of timber. The basis of the determination of this correlation is the high variation of the wood characteristic, because the grading system should be able to grade each piece of timber by given conditions: for example spruce with defined dimension of the cross section and moisture content. But it is still unknown, whether the grading machines are able to detect severe defects and determine correctly the mechanical properties of the tested timber, in order to assure a proper classification of the timber.

The objective of this project was to test the ability of different grading machines to detect significant timber defects. For this, three groups of spruce timber specimens with particular characteristics was first graded by different grading machines and later tested by loading to determine the effective strength and stiffness. The specimens were picked out of the regular production of a sawmill by visual means regarding that the defects were "well fitting". In fact the specimens were rejected in the sawmills by more or less accurate visual grading. Thus, it has to be stated that the total number of chosen samples are not representative and could not be compared with standard timber graded in a sawmill.

Conclusions and questions

The investigations carried out show clearly that machine-grading of timber with severe defects leads to some difficulties. Severe grain deviation is a frequent quality mark in common timber production. Severe grain deviation can be detected more or less easily by visual quality control. But machine grading should guarantee for the correct grading and should be used as substitute for the not always reliable judgment of the grading personnel. In case of the grading system investigated, only the measurement of the vibration frequency allows the detection of timber defects and gives good results for practical timber usage (required bending strength not fulfilled in up to 5 % of the samples).

Grading machines mostly work on the bending principle, so that the detection of severe grain deviation is not possible. Therefore, "mistakes" by grading seems possible.

Other investigated defects - pre-broken timber and slip planes - were very poorly detected and the result was a high percentage of "mistakes" by grading. The requirement for bending strength was not fulfilled in up to about 50 % of the investigated samples. Pre-broken timber normally should not be graded, but in case of grading pre-broken timber it should be expected that these samples will be automatically downgraded and rejected and not graded, as for example C30. Also slip planes are not so infrequently found in timber production. It was relatively simple to find sawmills, forestry agencies or timber companies offering defect timber. The reliable detection of slip planes does not seem to be possible for the investigated grading systems. Unfortunately, slip planes have an important influence on resistance values.

The chosen grading systems are practically used for the machine-grading of timber. But the use of timber with higher mechanical properties (C30 and higher) for structural purposes is possible since new codes appeared (EN 519, EN 338, EC 5, or the new DIN codes for grading and strength classes for timber). The difficulties to detect some of the timber defects can lead to a reduction of the safety of some buildings.

This work should not only end with the presentation of the results. Also questions should be asked on the requirement of grading systems, if the actual grading fulfilled the requirements on safety and if the non-detection of severe defects in timber can be accepted as normal characteristic of the grading machine. If so, it is necessary to mention the reasons related.

34-5-1 F Lam, S Abayakoon, S Svensson, C Gyamfi
Influence of proof loading on the reliability of members

Abstract

Proof loading concept is a recognized quality control technique to improve the characteristics of the lower tail of strength distributions of structural timber products. Very few comprehensive studies are available to quantify the effectiveness on the use of proof loading relating to the choice of proof load level, the potential damage on the members resulting from the application of proof load, and the improvement of performance in the context of reliability based design methods. One of the difficulties is the need of a rather large sample size for an experimental-based study to develop statistically meaningful solutions. This paper illustrates the use of damage accumulation model and reliability based design analyses to quantify the effectiveness of proof loading.

The performance of No. 2 and better Western Hemlock 38 x 140 mm dimension lumber in bending is considered. Damage accumulation laws are established to consider the residual strength of members that survived a proof load. Reliability analyses are conducted to compare the performance of proof-loaded and non-proof loaded members subject to snow load conditions in two locations in Canada. For a given reliability index (β), the improvement of performance can be quantified as characteristic strength adjustment factors for proof loading in terms of the ratio of the performance factors (φ) between the original and proof load material. Conversely the gain in reliability, β, for a given φ value is also apparent. The adjustment factors depend on the proof load level, the level, and the distribution selected for fitting the strength data after proof loading.

Conclusions

In this study a damage accumulation model is used to assess the damage caused by proof loading in Monte Carlo simulations. The bending strength distribution and residual strength distributions were evaluated in reliability analysis against dead loads and snow load conditions for two Canadian cities.

The relationships of the proof loaded members show a shift to the right in comparison with the original set. For a given reliability index, the improvement of performance can be quantified as characteristic strength adjustment factors for proof loading in terms of the ratio of β values between the original and proof load material. Conversely the gain in reliability for a given β value is also apparent. The percentage increase depends on the proof load level, the level, and the distribution selected for fitting the strength data after proof loading.

36-5-1 C Bengtson, M Fonselius
Settings for strength grading machines – Evaluation of the procedure according to prEN 14081, part 2

Abstract

This paper presents experience gained during the application of the new standard prEN 14081-part 2. The standard contains a procedure for derivation of setting values for strength grading machines. The standard was applied for representative raw material from the Nordic countries (Sweden,

Finland and Norway). Consequences when the raw material properties are better than those of the required grades and the importance of the choice of representative raw material data for derivation of settings are discussed. A "method for calculation" which would facilitate the use of the standard is suggested.

Although the three criteria presented above are also valid for visual grading it has in Europe been agreed to use some well defined national grading rules for assigning timber to the European strength classes. This means that the grading requirements are given for each individual timber piece instead of for a population of pieces. There are no possibilities to include low graded timber to a better class whatever high graded timber is included.

Conclusions and recommendations

During the work in CEN/TC124/WG2/TG2 by formulating and applying the suggested procedure for calculating settings different interpretations and understandings have come up. A clear procedure with examples, as presented under section 3.1 in this paper, would improve the standard considerably and facilitate the use of the standard.

The procedure for derivation of settings when the raw material is too good or the wanted grades are too modest need to be revised. A suggestion is given in this paper.

In the paper it is illustrated that the raw material used when deriving the settings must be representative for the material to be graded in production. It is shown that by deleting one of the sub-samples used in the present analysis the setting for C30 can vary by around 25%.

Further, consequences by deriving combinations of settings from already existing ones are pointed out. It is also pointed out that the methodology can result in considerably different settings for the same strength class.

36-5-2 J Köhler, M H Faber
A probabilistic approach to cost optimal timber grading

Introduction

Timber is by nature a very inhomogeneous building material. On a large scale the material properties are a product of e.g. the specific wood species and the geographical location where the wood has been grown. The material properties of timber may be ensured to fulfil given requirements only by quality control procedures – hereafter referred to as grading. For mechanical grading various schemes have been developed using different principles, however, the basic idea behind them all is that the relevant material properties such as, e.g. the bending strength, are assessed indirectly by means of other indicative properties (indicators) observed during a grading procedure such as e.g. the density or the modulus of elasticity. The allocation into different grades is taken as basis in acceptance criteria, which is formulated in terms of the indicators. An acceptance criterion can be implemented by means of the settings of a grading machine, whereby a given grade is allocated to timber for which the indicators have values belonging to defined intervals. The timber graded in this way has to match given requirements implicitly defining the indicator value intervals. As an example, the European standard EN 338 defines grades in terms of the lower 5% fractile value of the graded timber bending strength, the mean value of the bending modulus of elasticity and the mean value of the density. These requirements can be implemented by adjusting the settings of the applied grading machine. The acceptance criterion, i.e. the grading machine settings, leading to normative grades are normally obtained and calibrated by performing many test of the graded timber.

Due to the special way timber material properties are ensured by means of grading in the production line, special considerations must be made when modelling their probabilistic characteristics. Previous work on this subject is reported in e.g. Glos (1981) and Rouger (1996). In Pohlmann and Rackwitz (1981) a bi-variate Normal distribution model is suggested in order to describe the probabilistic characteristics of the graded timber. In Faber et al. (2003) a Bayesian approach is proposed allowing for a generalization of the bivariate Normal distribution model such that the prior probability distribution function of the un-graded timber material properties may be chosen freely in accordance with statistical evidence.

The selection of a grading procedure, i.e. the type of grading machine and the acceptance criteria, could be made based on cost benefit considerations. Different procedures have different costs and different efficiency characteristics. In the present paper it is demonstrated how an optimal (in terms of monetary benefit) set of timber grades can be identified. Therefore, the approach suggested by Faber et al. (2003) is utilized to identify timber grades and quantify the probabilistic characteristics of their relevant material properties. This requires that the probabilistic characteristics of the relevant material properties of the ungraded timber are known together with their correlation to the indicator. An optimization problem can be defined for identifying the grading procedure, leading to the optimal set of timber grades.

Discussion and Conclusions

It has been demonstrated how an optimal (in terms of monetary benefit) set of timber grades can be identified through the solution of an optimization problem. The objective function of the optimization problem is defined based on the findings reported previously by the authors where the statistical assessment of timber material properties has been considered with special emphasis on the modeling of the grading of timber. The identified timber grades can be described by means of the probabilistic characteristics of the relevant material properties as e.g. the bending strength, the bending modulus of elasticity and the density of the timber. The simplex algorithm for the optimization of non-differentiable object functions in conjunction with a simulation procedure has been applied for the identification of the optimal grading procedure. The constraints to the optimization problem, in terms of the requirements for timber grades according to EN 338 have been incorporated directly into the objective function.

An example has been presented illustrating the suggested approach to optimal timber grading. The assignment of monetary benefit to the different grades of timber has, however, been based on judgment rather than true values. In practice the benefit associated with timber of a particular grade would depend on a number of factors such as the size of the individual timber specimen, the total amount of available timber for a given grading, the production capacity of a given sawmill, the available grading machines and not least the market price for the different timber grades. The implementation of the proposed approach in practice would have to incorporate these and other factors more accurately into the formulation of the benefit function. Further studies in close collaboration with the timber industry should be undertaken and discussed to clarify these aspects and to set up a rational basis for their assessment. However, according to the preferences of a sawmill owner the proposed approach facilitates the identification and the calibration of a grading procedure and thus an increase in the overall production benefit.

36-7-11 A Ranta-Maunus, A Kevarinmäki
Reliability of timber structures, theory and dowel-type connection failures

Introduction

This paper has two separate parts both dealing with the reliability of timber structures. The first part is a theoretical one related to strength distribution of timber materials and to the theoretical effect of lower tail strength on safety factors in code calibration. Some results on the strength distributions of machine strength graded timber are discussed.

In the second part a recent failure case is reported. This is considered urgent because the failure was caused partly by inadequacy of the prestandard version of Eurocode 5, which can now be used in many European countries. Because of the accident, supplementary national guidelines will be given in Finland concerning the design of dowel-type joints, including block shear failure. The draft guidelines are given in appendix.

Summary

This paper reports on the theoretical investigation of strength of sawn timber, which suggests that higher grades are relatively much better than low grades, with the consequence that structures made of C40 have clearly higher structural safety than structures made of C18, when present European strength grading and structural design standards are used.

38-5-1 M Arnold, R Steiger
Are wind-induced compression failures grading relevant

Introduction

Compression failures (CF) are defects in the wood structure in the form of buckled cell walls of the wood fibres. They may be wind-induced in the standing trees, if the stems are bent so much by frequent or strong winds that the proportionality limit of the wood in axial compression is locally exceeded on the inward (leeward) side of the bow. The distorted fibres of CF remain as weak points in the wood and can lead to brittle fractures in processed timber already at a relatively low stress in bending or tension (Fig. 1). Therefore CF are regarded as unwanted structural defects.

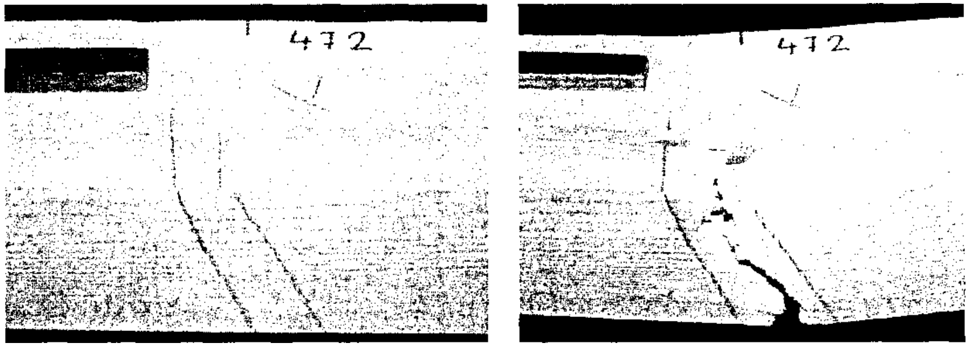


Fig. 1. A wind-induced compression failure in a squared timber beam before (left) and after (right) a destructive bending test. Initially the CF is barely visible on the planed surface, but the fracture was initiated directly in the previously marked CF and is extremely brittle and short-fibred

CF are a well-known 'natural' phenomenon and are observed quite frequently in some of our lower density native softwoods such as spruce (Picea abies). Particularly after heavy storm damages in the forests, questions regarding their influence on the utilisation of timber from the salvaged trees arise anew.

CF are complex three-dimensional geometric structures with more or less fuzzy boundaries and appear in a broad range of intensities. Their size can range from minute deformations in the cell wall to wide bands of several millimetres in width, which can affect more than half of the stem's cross-section. Because of their diverse appearance various terms are in use. Regarding processing and grading it is important to note that CF is usually difficult to detect, particularly in rough sawn timber.

The consequences of CF on the utilisation of affected timber and particularly on its mechanical properties are still debated. While a reduction of the mechanical properties (mainly in bending and tension) at the fibre level and in small clear wood specimens is generally acknowledged, the effect is less clear with structural timber, where the effect of CF may be confounded by the presence of other defects such as knots or grain deviations. Particularly regarding the effect of CF in structural timber only few comprehensive studies have been published so far.

However, because of their potential safety risk, many grading standards explicitly or implicitly ('mechanical damages') exclude CF from timber elements in load bearing structures. CF therefore can impose serious restrictions on the utilization of wind-damaged timber and require additional efforts regarding grading and quality control. Historical example: CF used to be particularly feared of in the production of wooden ladder rails or wooden parts for airplanes (e.g. wings).

The objective of this paper is to present new extensive data concerning the effect of wind-induced CF on the mechanical properties of structural timber stressed in bending and to draw conclusions regarding the consideration of CF in grading rules and procedures.

Summary and conclusions

Based on this case study the following conclusions are drawn regarding the influence of wind-induced CF on the mechanical properties of spruce structural timber:

1. There is a statistically significant reduction of MOR and MOE in bending of squared timber beams containing CF compared to beams without visible CF. The effect is more pronounced regarding MOR than regarding MOE.
2. Despite the general reduction of strength and elasticity, the limits for the characteristic values of the strength classes C20 and C24 of visually graded structural timber (according to the Swiss standard SIA 265/1) are still exceeded. Considering the decreasing influence of other structural defects, this may however not be the case in the higher strength grades.
3. Because the modulus of elasticity is only slightly affected by the presence of CF, machine stress grading methods relying on the dynamic or (low stress level) static assessment of the modulus of elasticity are not able to reliably detect CF. Machine stress grading of timber containing CF without an additional visual inspection may therefore lead to an overestimation of expected MOR and wrong strength grades.
4. The macroscopically visible appearance ('size') of the CF is only a weak indicator for the potential reduction of MOR and MOE. This makes it impossible to distinguish between 'benign' and 'malignant' CF and to define allowable 'size' limits for CF for visual grading procedures. Thus only a strict exclusion of CF seems practical.
5. The failure behaviour of timber containing CF is frequently abnormally brittle and exhibits low-strain, short-fibred fractures.

6. Because of the potential safety risk and the difficult prediction of their strength reduction, detected CF should be excluded from load bearing structural elements stressed in tension or bending and explicitly addressed in the relevant grading standards (as in SIA 265/1). Timber containing CF should only be used in compression loaded or not load-critical applications.

39-5-1 J Köhler, R Steiger
A discussion on the control of grading machine settings – current approach, potential and outlook

Background

Typical problems in structural engineering such as design, assessment, inspection and maintenance planning are decision problems subject to a combination of inherent, modelling and statistical uncertainties. Recent developments in the field of structural reliability together with the formulation of probabilistic models for the structural response and for loads enable the structural engineer to quantify these uncertainties and establish his decisions on a consistent basis. The results of these advances are summarized in the Probabilistic Model Code (PMC) published by the Joint Committee on Structural Safety (JCSS).

The PMC contains general guidelines for uncertainty modelling, reliability assessment and probabilistic models for loads and material resistance for building materials as concrete and steel. Lately, also a probabilistic model code for structural timber has been developed by an international group of researchers organized under the umbrella of the COST action E24 'Reliability of Timber Structures. During the COST action E24 it has become apparent that several fundamental issues require more research and development. In particular it has been found that the present practice in regard to strategies to quality control or grading of timber raw material introduces significant uncertainties on the performance of timber structural components. In the present paper control schemes for timber grading machines are discussed from the perspective of the probabilistic modelling of material properties of graded timber.

Introduction

Reliability analysis of structures for the purpose of code calibration in general or for the reliability verification of specific structures requires that the relevant failure modes be represented in terms of limit state functions. The limit state functions define the realizations of resistance parameters, i.e. the material properties and the load variables resulting in structural failure. In reliability analysis of timber structures the probabilistic modelling of the material properties is an issue of special interest due to the particular way this material is "produced".

Timber is by nature a very inhomogeneous building material. On a large scale the material properties are a product of e.g. the specific wood species and the geographical location where the wood has been grown. Given species and geographical location the material properties depend on factors such as the age, the diameter of the timber logs and the number of knots together with the moisture contents and the duration of loading. In comparison to other building materials such as steel and concrete, the properties of timber materials are not designed or produced by means of some recipe but may be ensured to fulfil given requirements only by quality control procedures – hereafter referred to as grading. Quality control and selection schemes are implemented in the production line, typically already at the sawmills where the construction timber is produced from the timber logs. Various schemes for grading have been developed using different principles, however, the basic idea behind them all is that the material properties of interest such as, e.g. the ultimate compression stress, are assessed indirectly by means of other properties such as e.g. the density or the modulus of elasticity.

Conclusion and Summary

Machine grading is introduced as a prerequisite for the safe and efficient utilization of timber in load bearing construction. It is focused on machine grading schemes and the currently applied strategies for the control of grading machine settings are discussed. The CUSUM method is described as a typical methodology for output controlled systems; the methodology first proposed by Rouger and now implemented in the European Standard EN 14081 is taken as an example for a machine controlled system. It is noted that both control methods do not allow for a probabilistic assessment of the graded timber material properties based on the corresponding formalism of the control methods. Furthermore, several shortcomings within the machine controlled method according to the European standard EN 14081 are outlined and discussed.

Based on Faber et al. an alternative method is described; the statistical assessment of timber material properties has been considered with special emphasis on the modelling of the effect of different schemes for quality control and grading of timber. The suggested approach not only forms a very strong tool for the statistical quantification of the material characteristics of timber but furthermore provides a consistent basis for quantifying the efficiency of different quality control and grading procedures. The probabilistic models for the graded timber material properties have been formulated such that they readily may be applied in structural reliability analysis.

It is of utmost importance that the statistical characteristics of timber material properties are assessed and treated in consistency with the implemented quality control and grading procedures. Only then a consistent basis may be established for the quantification of the reliability of timber structures - the basis for codification of design and assessment. The suggested probabilistic modelling seems to provide the required framework for establishing such a basis by means of quantifying the efficiency of the different quality control and grading procedures. It is envisaged that different quality control grading procedures may be described by means of their regression characteristics and acceptance probability curves corresponding to different grading criteria. A format for the standardisation of the probabilistic modelling of timber materials subject to different quality control and grading procedures is suggested by Faber et al. It is important that the appropriateness of such a format is discussed and that a consensus is achieved in this respect in the near future.

Based on Köhler and Faber in section 4.2 it has been demonstrated how an optimal (in terms of monetary benefit) set of timber grades can be identified through the solution of an optimisation problem. The objective function of the optimisation problem is defined based on the methodology presented in section 4.1. The implementation of the proposed approach in practice would have to incorporate factors such as the marked price of timber, potential demand of the building sector, etc into the formulation of the benefit function. Further studies in close collaboration with the timber industry should be undertaken and discussed to clarify these aspects and to set up a rational basis for their assessment. However, according to the preferences of a sawmill owner the proposed approach facilitates the identification and the calibration of a grading procedure and thus an increase in the overall production benefit.

It is also discussed how new information obtained during the operation phase of the grading machine can be used for updating the model parameters involved. Bayesian statistics constitutes the basis for these updating schemes and a publication is plan which aims the illustration of typical updating situations in timber engineering along some illustrative examples.

39-5-2 R Katzengruber, G Jeitler, G Schickhofer
Tensile proof loading to assure quality of finger-jointed structural timber

Introduction / Problem / Motivation

Timber as a natural growing raw material displays large variations in its mechanical characteristics like strength and stiffness in comparison to other materials such as e.g. steel. These variations can be considerable precise with the beam-shaped product structural timber, characterised by lack of homogenisation over the cross-section through gluing of individual components. A statistical 'system effect' which can be considered for glulam or bi- or trilam is not present for single sections. Although grading criterions are defined in DIN 4074-1 with the currently common grading processes strength reducing defects such as the global and local grain deviation, compression failures, reaction wood, pre-broken timber or damage of treetop are only with difficulty and often not economically ascertainable. Rogues in the lowest quantile area of strength cannot be excluded for sure. The grading process within the production of structural timber is therefore still a challenge.

Even so, performance and minimum production requirements for finger joints of structural timber are regulated in EN 385, a similar difficulty comes up with the joining. This is because for internal and external quality control only the bending strength and mode of failure of few randomly taken finger joint samples are determined in destructive tests. This also results in the fact that structural timber with features responsible for poor finger joint strength can reach the customers.

Discussion and Conclusions

The completed cyclic stress tests, as described in point 4, confirm that a low tensile stress not leading to failure, only minimally affects the strength of structural timber. The evidence that the material is not significantly damaged is herewith clearly adduced. The number of tested specimens (4,886 #) or rather 39.000 # with the referred test length of 1.6 m of series A in relation to the number of faults with slightly reduced strength characteristics after the first stressing seems to be sufficient to confirm that statement. The triple stress test of series B confirm further, that a tensile load that could be sustained once (not leading to failure) can be sustained in 99,47 % of the cases again and in 99,29 % of the cases a third time, indicating not being damaged. The results of experimental research work as presented on that high number of specimens show clearly that there is NO appreciable damage to surviving timber due to tensile proof loading at low load levels. Or in other words: The timber is not significantly damaged within tensile proof loading as described in this paper.

The conclusion therefore clearly is that it is better to have tensile proof loaded timber in structural applications than the risk of 'rogues' with poor strength characteristics. Further there should not be any doubt of stressing timber up to the level of design strength which is specified for grade C24 with $f_{t,0,d} = k_{mod} f_{t,0,k} / \gamma_m = 1{,}1 \cdot 14 / 1{,}3 = 11{,}8$ N/mm^2, assuming an instantaneous load duration.

Because the length effect in wood is characterised by reduced scatter and decreased strength characteristics with increasing length its consideration is important for design purposes when it comes to long structural elements. However the testing length has no influence on the result of tensile proof loading in respect to failure recognition. When at a certain point of the specimen the fracture occurs due to the loading the residual volume is released. Further defects of the specimen with also poor strength characteristics, but higher than the one before can only be detected within a repeated proof loading process. In connection with the failure modes and the associated 'learning effect', the proof loading method represents a significant possibility of systematically improving grading within the production process, whether visual or mechanical.

A further area of application of the test method presented here and implanted on an industrial level exists for other sawn timber products in the branch. Glulam production is particularly considered here. It is conceivable to also implement the presented proof loading method in an adapted form for the online quality assurance of finger jointed single lamellas. Furthermore application of the method for testing finger jointed flange sections of I-profiles and nail plate binders is considered sound.

Generally, a timber product with a more reliable minimum strength should be made available to the construction industry by the presented tensile proof loading method as every piece in the lower area of the strength distribution is rejected. The increased reliability for proof loaded finger jointed structural timber could also be reflected in a more favourable partial-coefficient. The corresponding quantification in dependency of the proof level and coefficient of variation of the base material is part of further investigations.

40-5-1 K Crews
Development of grading rules for re-cycled timber used in structural applications

Introduction

Until recently, the usual method of disposal of timber used in structures has been demolition and disposal. For example, at the time of writing, Australians are placing approximately 1 million tonnes of wood waste into landfill sites. However, reduced availability of native hardwoods has created a situation where use of recycled timber has significant environmental and economic potential, particularly where recycled products can be incorporated into new construction or in some cases retrofitting of existing buildings and structures.

Currently, there are no standards or recommendations for assigning design properties for structural reuse of wood and the use of recycled timber in decorative products tends to rely on subjective application of visual grading rules developed for new timber. In order to address this problem and utilise the recycled timber resource effectively and reliably, the author has undertaken a research project in conjunction with Timber Queensland (funded by the Forest and Wood Products Research and Development Corporation).

The aim of this project is to develop appropriate (visual) grading systems that take into account the properties of recycled timber; in particular, how the history and previous use of the timber has effected its properties in terms of being fit for purpose in a re-use application. The paper will present the findings of this project to date involving research to quantify the mechanical properties and develop appropriate (visual) grading systems that take into account the properties of recycled timber; for use in both structural and aesthetic applications.

Conclusions and recommendations

Only 90 specimens have been tested to date and additional testing is currently being undertaken to gain more data and hopefully improve confidence in the results. However, from analysis of these preliminary results some trends are apparent.

Based on the initial test results, it can be concluded that application of existing visual grading rules (AS 2082) can be used to assign properties for timber cut from larger members (and not containing defects such as degrade and bolt holes) on the following provision:
1) MoE values are valid and can be assumed to be similar to those of new material.
2) MoR values must be reduced when compared to new material to take into account duration of load effects. These reductions are estimated to be between 35 % for material with a load history of short term / low magnitude loading (such as roof structures with light weight cladding), and 50% for material with a load history of longer term / high magnitude loading (such as warehouse / wharf storage floors).

Two interim grading documents are currently being prepared. Both drafts have been developed in consultation with relevant industry groups.

The first deals with Appearance Grade products and is based on AS 2796, but incorporates specific clauses for classification of recycling characteristics and their "impact" on appearance – such as holes from fasteners and surface "imperfections" resulting from previous usage.

The second draft details structural grading rules for recycled hardwood timber and has been prepared as a blend of both AS 2082 and AS 3818 to cover both smaller end-section material and larger end-section material included in AS 3818. The draft will be a stand alone document specific to recycled timber and considers separately smaller end-section timber where there is closer alliance to AS 2082 and larger end-section timber that is more closely aligned to AS 3818.

Due to the inherent differences between recycled and "new" timber, traditional "strength groups" have not been used in the draft, however a similar concept referred to as "Species Group" has been included, so that species of recycled timber displaying similar properties and characteristics affecting the stress grade can be grouped together. This has been done to facilitate an efficient means of applying the stress grades, whilst at the same time keeping the number of rules to a workable minimum.

It should also be noted that the draft documents are being developed as interim "Industry Standards" to provide an orderly introduction into the marketplace, within a shorter time frame than would be possible implementing using the "Standards" development process. Whilst the documents are specific to recycled timber, they have also been developed to be very much in line with current applicable Australian Standards, with the intention that after "evolution", the documents will achieve full "Standards" status.

It is anticipated that the outcomes of this project will enhance the use of recycled timber products in the market place, whilst at the same time providing appropriate standards for defining "fit for use applications" and ensuring safe characteristic properties are used by designers when using recycled timber members in structural applications.

40-5-2 M Sandomeer, J Köhler, P Linsenmann
The efficient control of grading machine settings

Abstract
Machine grading systems operate according to similar principles; one or more indicative properties of the timber to be graded are measured by the machine and based on these measurements a sample of ungraded timber is subdivided into sub samples of graded timber. The grading acceptance criteria are formulated in form of boundary values for indicative properties that have to be matched to qualify a piece of timber to a certain grade. These boundaries are termed grading machine settings. The grading performance, i.e. the statistical characteristics of the output of grading machines strongly depends on these settings, and in general very much attention is given on how to control them.

Currently applied procedures to control the grading machine settings are either machine controlled or output controlled, however, both procedures do not facilitate the efficient use of information which is gathered prior and during the runtime of the grading machine. The utilization of the entire available information is required for the accurate representation of uncertainty in modelling the material properties of graded timber.

In this paper the efficiency of existing control procedures for timber grading machines is discussed with regard to the capability of the procedures to assess systematic changes in the quality of the timber supply. Experience gained during the application of these methods is presented. An alternative approach for the control of grading machine settings is introduced which can be seen as a combination of the machine and output control procedure. It facilitates the consistent consideration of new information gained prior and during the grading process. The approach is summarized in a coherent and implementable format and possible benefits of its application in practice are discussed.

Conclusion

The goal of the present paper is to simulate situations where a noticeable quality shift between initial tested timber material and subsequent applied input material is observable. A method for the machine control (cost matrix method), the output control (CUSUM) and the combination of both (probability based approach) is used to assess the macro and meso variability of the material supply and the results are compared. The capability of the methods to incorporate statistical uncertainties as well as model related uncertainties into the grading process is of special interest.

The results show that the macro variability between different sub samples of the timber material supply is detected by the cost matrix method only faintly. In the case that quality shifts are identified a substantial reassessment of adjusted grading machine settings is required. The incorporation of uncertainties is done implicitly; i.e. by sampling different sub samples of different origins and by comparing assigned strength grades with the optimum grades. However, as long as the characteristic values of the derived strength classes are assessed by means of sample statistics considering solely the underlying set of data it is not possible to represent all uncertainties in a consistent matter.

The application of the output control by means of CUSUM control charts is observed to be capable to detect the aberrations in the quality of the material supply. However, compared with the results of the variables chart the average run length of the attributes chart indicates a remarkable delay in identifying the quality shift of the material input resource. Though very much attention is given on how to sample the test specimens randomly in order to manage the underlying statistical uncertainties, the method is still based on sample statistics and therefore only capable to qualify shifts of quality of the timber supply but not to quantify them.

The probabilistic approach shows to be a consistent combination of both methods described above. There is a crucial difference in the basic statistical concept of the models. The presented approach is based on a regression model which represents the interrelation of properties of interest and indicating properties. Based on this information, predictive distributions for the material properties of interest can be quantified taking into account a certain set of grading machine settings. This probabilistic approach offers also the chance to assess shifts in the material quality by controlling the values of the indicating properties for the material properties of interest continuously. Note, that this information is always available without additional costs. Assessed aberrations of quality may be counteracted by means of updating the regression parameters of the probabilistic grading model. In consequence of updating the regression model it is a straightforward task to assign adjusted grading machine settings without additional substantial test procedures.

The statistical assessment of timber material properties has been considered with special emphasis on the modeling of the effect of macro and meso variability within the timber material supply. The suggested probabilistic approach not only forms a very strong tool for the statistical quantification of the material characteristics of timber but furthermore provides a consistent basis for quantifying the efficiency of different quality control and grading procedures.

42-5-1 R Ziethén, C Bengtsson
Machine strength grading – a new method for derivation of settings

Abstract

This paper presents and analyses a new method for derivation of setting values for strength rading machines. The method uses a model for the relationship between indicating property measured by a grading machine, IP) and the grade determining property (GDP). This model is of course not perfect and by summarising the errors in the model a confidence interval for the model can be calculated. The confidence interval is used for creating a prediction interval. The lower limit of the prediction interval, the prediction limit, is used to predict the GDP.

The prediction limit method is analysed and evaluated by well defined input data. It is shown that fewer experimental data than required by the method in EN 14081-2 today is needed to determine reliable settings but the producer is awarded with less conservative settings with an increased number of experimental data. A weak correlation between IP and GDP or a high coefficient of variation also results in conservative settings. The settings are not dependent on average strength of the raw material used for deriving the settings.

Conclusions and recommendations

The evaluation of the proposed prediction limit method for calculation of settings has shown that:

– The calculated settings are not dependent on average strength of the raw material

- A higher COV of the raw material results in more conservative settings
- Weak correlation between indicating property and grade determining property results in more conservative settings
- Few observations for calculation of settings results in more conservative settings

The method is simple and based on common statistical theory. Such a simple method is on the market today operates with multiple settings and the prediction limit method can be adopted also to this case. The method gives one setting for each grade independently of grade combination. Depending on the percentile level chosen the calculated settings result in graded material with different safety levels.

The prediction limit studied in this paper is the lower 5% limit and the achieved characteristic alues of the graded material are compared to the characteristic values according to EN338. The prediction limit method opens for a possibility to calculate another limit such as the lower 1% or lower 0,1 % limit and compare the result to other required values. It is the combination of confidence level of the prediction limit and the required value that defines the safety for the end user of the graded timber.

In the comparison between the achieved characteristic values after grading based on the prediction limit and the requirements in EN338 it is mainly the density that fails to be fulfilled. One reason for that is obviously the poor correlation between strength and density since the settings were determined based on the relation between IP and MOR. Other reasons can be an unrealistically chosen density in the data sets or an unrealistic required density in EN338.

The authors of this paper recommend a deeper analysis of the prediction limit method for calculation of settings for grading machines. In connection to this also new requirements for the graded material are needed.

43-5-1 F Rouger
Quality control methods - Application to acceptance criteria for a batch of timber

Abstract

In the ratified version EN 384, a method was stated in section 9, regarding quality verification of a batch of timber. This paper aims at showing that the current approach is not correct, and proposes a new approach that is in line with existing theories. This new method is based on quality control concepts. For stiffness control, average quality criterion has to be used, whereas for strength control, fraction defective criterion shall be applied. Both criteria use a risk concept:

- The risk for the producer, denoted a, is the risk that a batch is rejected by the consumer whereas it is correct
- The risk for the consumer, denoted b, is the risk that the consumer accepts the batch whereas it is not correct

This paper firstly describes the theory, and illustrates the efficiency of the method on simulated and real experimental data sets. It also demonstrates the inefficiency of standardized approach. As a conclusion, the proposal based on this investigation, and which has been incorporated in the new version of EN 384 is briefly described.

Conclusions

Theoretical concepts of quality control have been successfully applied on the acceptance criteria for a batch of timber. Numerical investigations on “real data” clearly show that the existing approach of the standard is not correct, and that the new approach gives very good results. This justifies the revision which has been incorporated in the new version of the standard which is going to be ratified in a close future.

43-5-2 P Stapel, J W v. d. Kuilen, A Rais
Influence of origin and grading principles on the engineering properties of European timber

Introduction

Due to increasing effort of grading machine manufacturers the amount of timber which can be used for approvals according to European standard EN 14081 increases. The available data can be used to obtain uniform settings that cover large parts of Europe. The European standard for machine approval EN 14081-Part 2/3 was developed when much less data was available. Alternative methods for machine grading in Europe have recently been proposed. This paper deals with the current system. The improved experience in deriving machine settings of today, allows a critical analysis of the rules and methods decided upon at the time. This can be tested today, as a dataset of almost 5000 boards is available. The approval procedure requires the use of modification factors from a number of standards, of which EN 408 and EN 384 are the most important.

Timber quality (in terms of bending strength, modulus of elasticity and density) influences the settings for the machine. Low quality timber usually leads to higher settings and vice versa. However, the settings are not defined on the basis of the weakest sample only, but on a mean value of all sub-samples used in the approval procedure. The origin of the trees has an important influence on the mechanical properties of the sawn timber. If the user of a grading machine uses timber from the same sources and in a similar proportion as used for the derivation of settings, the output material can be safe. On the contrary, if a sawmill only uses timber from a limited radius around the production site, the results can be far from safe when the original approval testing covered a much larger area. If the incoming material is of low quality only, the graded material will not meet the required characteristic values on a regular basis.

Factors used in the derivation of settings include a depth factor k_h and a k_v - factor, which will be discussed in more detail. According to Eurocode 5, the depth effect may be applied on beams with a depth less than 150 mm. For depths above 150 mm, no correction for strength is allowed. However, during the derivation of characteristic values according to EN 384, the characteristic 5th-percentile value of sub-samples with a depth of more than 150 mm, shall be increased with the depth factor k_h given as:

$$k_h = \left(\frac{150}{h}\right)^{-0,2} \tag{1}$$

It means that characteristic values following the dotted line in Figure 1, will be used by designers as following the continuous line. A discrepancy of around 10% can be observed for beam depths typically used for floors (> 200 mm).

In addition to the depth effect, EN 384 allows a k_v-factor to be applied on the characteristic value of the samples graded by a machine for strength classes up to C30 according to EN 338. The value of k_v is set fixed at 1.12, independent of the quality of the machine or strength class combination. The idea behind this factor is that the variation of the characteristic values of sub-samples is smaller for machine graded timber as compared to visually graded timber.

Other factors that influence the quality of visual and machine grading are the length of the pieces that were used in the analysis as compared to the lengths used in practice. Other than the influence of the length of the pieces during the bending tests according to EN 408 / EN 384, length effects have not been taken into account here.

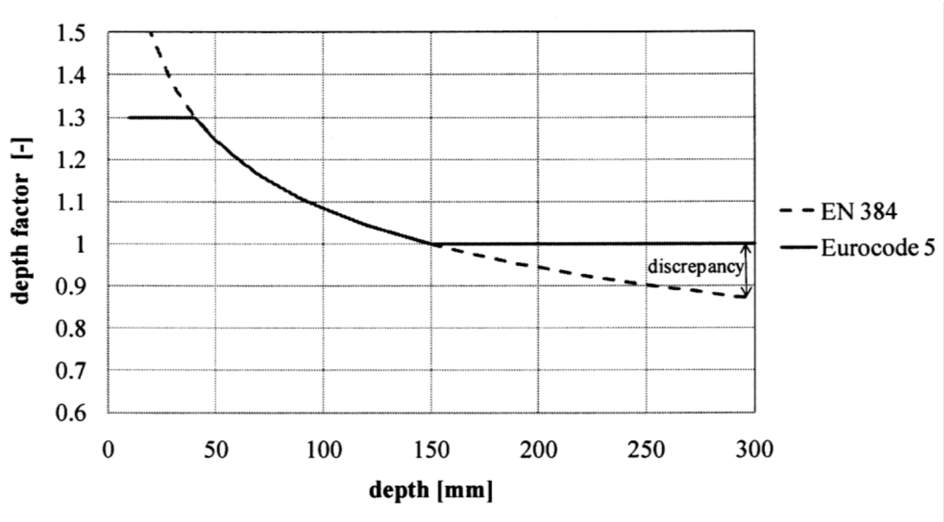


Figure 1: Discrepancy in the depth effect between EC 5 and EN 384.

As no pan-European rules are used for visual grading of timber, EN 1912 regulates the application area of national grading rules. These grading areas are usually interpreted very wide. If visual grading is not sensible to the timber origin, this can be justified. The objectives and the analysis given in this paper can be summarized as:

I) What is the influence of the origin and the grading principle on the characteristic value of the bending strength using the European standards EN 14081 and EN 384, after testing in accordance with EN 408?

II) Does visual grading allow large growth areas like indicated in EN 1912?

III) EN 384 and EN 14081 allow the use of some adjustment factors as mentioned previously:
   i. To what extend do these factors influence the grading output?
   ii. Is the k_v-factor (k_v =1.12) justified by a lower variation of machine graded timber compared to visual graded timber?
   iii. Is the variation in 5th-percentile values of sub-samples such, that different safety factors can be justified depending on the type of grading?

Conclusions

Visual grading of wood into strength class C30 seems not to be justifiable, certainly not for the application range (growth areas) given in EN 1912. The coefficient of variation of the 5th-percentile strength values is such that higher safety factors need to be applied than currently specified in Eurocode 5. In addition, in some sub-samples the yield is so low that no characteristic values can be determined.

Visual grading into strength class C24 is possible on its own, leads however to a large scatter in 5th-percentile strength values for the different sub-samples. This in return should require high γ_M values, much higher than currently specified.

When only C24 is graded, also machine grading leads to a large variation in characteristic strength values within the grade (sub-sample level). When grading C24 by machine, a higher strength grade should be produced at the same time, increasing the threshold value for C24 and decreasing the variation within the grade. Grading of only one strength class by a machine should not be permitted, unless higher safety factors are prescribed for such material.

Adjustment factors given in EN 384 and EN 14081 intensify the problems caused by deriving settings for large areas according the current standard. Therefore it is proposed to delete the k_v-factor from EN 384/14081-2. The k_v-factor allows more test results from the low strength range to become part of the sub-sample, having a higher scatter and consequently requires higher safety factors. The k_h-factor from EN 384/EC 5 should also be deleted. Apart from the fact that no evidence of its existence is found in the test results, there is a clear inconsistency in the standards. When deriving settings according to EN 14081 the weakest sub-sample should be taken into account and limits on the allowable deviation from the target value should be specified.

The advantage of machine grading (i.e. more reliable material with smaller scatter in strength properties) should lead to the specification of different safety factors for visual and machine graded timber. For grades C30, C35 and C40 produced by a machine, there is clearly a much lower scatter at characteristic strength level, justifying a difference in safety factor between visual grading and machine grading. This could be done by specifying different γ_M values, but another option is the specification of a modification factor, taking into account the difference in variability between visual and machine grading.

2.10 STRENGTH VALUES

7-6-1 K Möhler
Strength and long-term behaviour of lumber and glued laminated timber under torsion loads

Aim of the Research Work
In modern wood-constructions especially with glued laminated timber the wooden members are often strained by torsional loading. Therefore they must be designed to withstand these stresses. Publications concerning the torsional strength and the torsion behaviour of larger wood-structures can hardly be found. The German design standard as well as the standards of other countries gives no special values for the allowable torsional stresses and the torsion modulus of large sections of lumber and glued laminated timber. Finally no methods for the calculation of the torsional stresses and the distorsions are known for the anisotropic material wood, having natural faults. The methods, published until now were only applicable to the faultless wood rod which has a fixed position to the rings and the grain direction. The necessary values for designing torsional members could only be gained by tests with samples of real sizes. Therefore it was the aim of this research work to determine values for the torsional strength and the torsion modulus of timber and glued laminated timber cross-sections using structural wood. Also the influence of duration of loads had to be observed. Besides this, for glued laminated timber, the bearing- and torsion behaviour, by simultaneously acting of torsion-, bending- and shear-stresses should be investigated.

Conclusions
For timber and glued laminated timber with a density of $\rho_{12} \geq 380\,\text{kg/m}^3$ and with a humidity of 12%, using a rectangular cross-section and a depth to the width ratio $h/b \geq 2$ allowable torsional stress of

$$\tau_{TVH} = 1{,}2 \ \text{N/mm}^2$$

for timber and of

$$\tau_{TBSH} = 1{,}6 \ \text{N/mm}^2$$

for glued laminated timber could be assumed.

Thereby the safety referred to the 50%-fractile values is at least of 2.5. In a simultaneous acting of shear from shear loads these values must be multiplied by the factor

$$k_{\tau V} = 1 - \left(\tau_V/3\right)^2$$

τ_V is the actual shear stress caused by the shear loads.

In practice a shear modulus of G_T = 330 N/mm^2 for timber and G_T = 500 N/mm^2 for glued laminated timber could be used.

24-6-1 T A C M van der Put
Discussion of the failure criterion for combined bending and compression

Summary
One of the conclusions of the stability group of CIB-W18A was that the Code must allow for analytical solutions for stability design based on quasi linear behaviour and must revert to these methods (e.g. to the Larsen-Theilgaard method for columns). Although a new parabolic failure criterion for bending with compression is proposed for the Eurocode, based on glulam simulation, the interaction equation for in plane buckling of the code, being applicable for short columns as well, suggests a much less curved failure criterion. It therefore could be seen as a task of the stability group to reconsider the failure criterion.

For that purpose a derivation is given of a consistent simple improved failure criterion for bending with compression that may account for the influence of quality and moisture content, leads to simple interaction equations for beam-columns and may meet the requirements of the Eurocode.

Together with the proposal of **Paper 23-15-2** a possible consistent design method for braced and free beam-columns is proposed for the Eurocode and is used in the new Dutch Code.

In the appendix an explanation is given of the bearing- and shear strength.

Conclusion
It is shown that a simple design failure criterion for bending and compression is possible that can be explained by the elastic-full-plastic approach, with linear elasticity for tension up to failure, determined by the volume effect, and by unlimited flow in compression. This criterion:

$$Y = 1 - X + \frac{4X(1-X)}{3s-1}$$

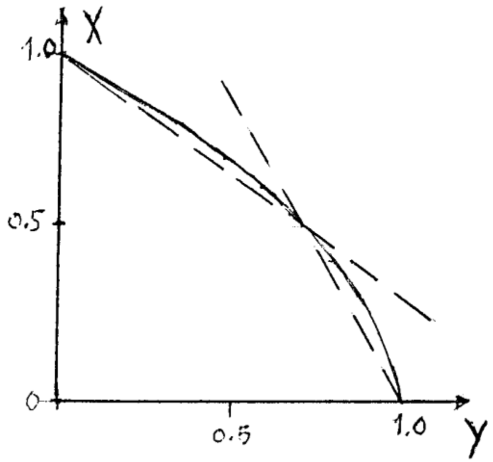
accounts for quality and moisture effects by the value of s.

For s = 1.67 this is:

$$Y = 1 - X^2$$

The new criterion of the Eurocode, is applicable at constant m.c. except for the highest four grades above 15% m.c. Here s = 2.3 is safe giving: $Y = 1 - X/3 - 2X^2/3$.

Because there will always be a linear cut off of the failure criterion, a simple linear approach of the parabolic failure criterion is appropriate, and lines can be drawn through the Y values of point $X = 0$ and point $X = 0.5$ and the Y values of the points $X = 1$ and $X = 0.5$ (avoiding too high estimates).



bi-linear failure criterium for bending with compression and shear

Thus the failure criterion then becomes in general:

$$Y + c_1 X = 1 \text{ when } X \leq 0{,}5 \text{ with } c_1 = (s-1)/(2-0{,}33)$$

$$c_2 Y + X = 1 \text{ when } X > 0{,}5 \text{ with } c_2 = (s-0{,}33)/(s+0{,}33)$$

or when smaller: $c_2 = f_m \tau_d / f_{v,d} \sigma_{m,d}$ where for rectangular cross sections:

$$Y = \frac{6M}{f_m b h^2} \; ; \; X = \frac{N}{b h f_{c,0}}$$

26-6-1 T A C M van der Put
Discussion and proposal of a general failure criterion for wood

Introduction

Failure criteria, like the Norris, Hoffmann, Tsai-Wu- criteria etc., can be seen as forms of a polynomial expansion of the real failure surface. This expansion of the failure surface in stress space into a polynomial, consisting of a linear combination of orthogonal polynomials, provides easily found constants (by the orthogonal property) when the expanded function is known, and the row can be extended, when necessary, without changing the already determined constants of the row. When choosing in advance a limited number of terms of the polynomial, up to some degree, the expansion procedure need not to be performed, because the result is in principle identical to a least square fit of the data to a polynomial of that chosen degree. This choice of the number of terms may depend on the wanted precision of the expansion and the practical use.

Based on this principle of a polynomial expansion of the failure surface, the failure criterion is general, satisfying equilibrium in all directions.

A general approach for anisotropic, not orthotropic, behaviour of joints, (as punched out metal plates), and the simplification of the transformations by 2 angles as variables, is given in a paper by Van der Put. A confirmation of the results of by means of coherent measurements (in the radial-longitudinal plane) and the generalization to an equivalent, quasi homogeneous, failure criterion for wood with small defects, showing, as will be discussed here, a determining influence of crack propagation on the equivalent main strengths, There thus is no reason to maintain the used invalid approximations and to not apply this consistent criterion, also for the Codes, for all cases of combined stresses. Thus far only this criterion gives the possibility of a definition of the off-axis strength of anisotropic materials.

Conclusion

- All following conclusions apply for the normally used softwoods.
- The tensor polynomial failure criterion can be regarded as a polynomial expansion of the real failure criterion. As a consequence, when a least square fitting procedure is used instead of the expansion procedure, a, in principle, complete polynomial up to the chosen degree is necessary (whereby terms of this compound polynomial may vanish by general symmetry conditions). Further this fit by a limited number of terms need not to show the precise right values of the expanded components and need not to pass all mean values of the measurements precisely and also p.e. the normality rule and convexity requirement need not to apply exactly.
- In transverse direction a second order polynomial is sufficient to describe the strength. When for compression (perpendicular to the grain) the strength is defined as the value after flow and some strain hardening, a lower bound of the overall strength can be chosen that will be directional independent and the behaviour can be regarded to be quasi isotropic in the transverse direction.
- There is a strong indication that for initial yield, or when the tangential plane is determining (as follows from direct measurements and the oblique-grain test), also for the longitudinal strengths a second order polynomial (with F12 = 0) is sufficient as yield criterion. When the test becomes unstable early, at initial crack extension, as for instance in the oblique-grain tension test or for compression in the "Shereisen" test (probably also in the radial plane), there are no higher order terms. Higher order terms thus are due to hardening effects (real hardening or equivalent hardening by crack arrest) depending on the type of test that may provide stable or unstable crack propagation after initial yield.
- It is shown that, when the Hankinson parameter n = 2 for tension and compression, all higher order terms are zero. It is probable that this is a general property for timber, because when shear failure is free to occur the plane of minimum resistance, as usually in large timber beams and in glulam, it occurs in the tangential plane, showing no higher order terms.
- For clear wood (and wood with small defects), in a stable test, the longitudinal shear strength in the radial plane increases parabolically with compression perpendicular to this plane depending on the coupling term F266 giving the Mohr equation or the Wu- equation, that can be explained by collinear micro-crack propagation in grain direction. This increase is an equivalent hardening effect, due to crack arrest by the strong layers, causing failure only to be possible by longitudinal crack propagation. It is shown that the increase of the shear strength is not due to Coulomb friction, being small for wood.
- Because of the oriented crack-propagation, explaining the Wu-equation, F166 ≈ 0 for clear wood because of is in the same direction as the flat crack and thus not influenced by that crack. Except for small clear specimens at compression, (providing a high shear strength by the volume effect), there also is no indication of an influence of the normal coupling terms F12' F122 and F112 (due to hardening by confined dilatation).
- For wood with (small) defects and local stress and grain deviations, an equivalent polynomial failure criterion is possible, showing therefore an influence of higher order terms. At least a fourth order polynomial is necessary for a reasonable description. A precise description by a third order polynomial is possible when 2 different criteria are regarded, one for longitudinal tension and for longitudinal compression (similar to the 2 Hankinson equations).
- The general form of the criterion for the uni-axial off-axis strength for wood with defects, is at least determined by a fourth degree equation in σt) and can always be factorized as a product of two quadratic equations. This leads to extended Hankinson equations, for higher order terms, when n is different from n = 2.
- Because of grain- and stress deviations, F166 will not be zero for timber, as for clear wood, because crack extension along the grain has components in longitudinal and transverse directions when there is a grain deviation and F166 and F266 are connected as components depending on the (local) structure. As crack extension component F166 will show a similar cut-off parabola as F266' indicating the common cause.
- For the same reason, the uni-axial tensile strength in the main direction is determined by the shear strength of the oblique material planes and and F112 represents F266 the real material planes, showing the same Wu-parabola. Similar to F166, F122 may act as component in transverse direction of F112.
- For wood with defects, when the principal strengths in the main planes ($\sigma_c = 0$) are determining, F112, F12 and F122 are zero, for longitudinal tension (due to early instability of the test). For combined shear failure (equivalent hardening), there are, for this case, small positve values of

F112 and F122. It is however shown that for a practical criterion these terms can be neglected and only F166 and F266 remain for longitudinal tension.

- For longitudinal compression at $\sigma_c = 0$, equivalent hardening by crack arrest, (high F112) as well as hardening by confined dilatation (showing a negative F122 and F12) may occur. This last type of hardening probably only occurs in the torsion tube test, because the negative F122 and F12 predict a compression peak that does not occur in the oblique grain test. For structural elements, this effect thus has to be neglected and the lower bound criterion with only F166 and F266 (and zero F12, F122 and F112) applies also for compression in the radial plane as follows from the good fit.
- Because in the tangential plane, the higher order terms can be zero, the quadratic polynomial eq.

$$\frac{\sigma_6^2}{S^2}+\frac{\sigma_1}{X}-\frac{\sigma_1}{X'}+\frac{\sigma_1^2}{XX'}+\frac{\sigma_2^2}{YY'}+\frac{\sigma_2}{Y}-\frac{\sigma_2}{Y'}=1$$

- should be used as lower bound for the Codes in all cases, for timber and clear wood, and because the equation represents initial yield as well it will apply for the lower 5th percentile of the strength.
- For large sized timber and glulam, where shear failure (or longitudinal tensile failure) may pass radial as well as in tangential directions in the same failure plane, the following will apply:

$$\frac{\sigma_6^2}{S^2}\left(1+c_{266}\frac{\sigma_2}{Y'}+c_{166}\frac{\sigma_1}{X'}\right)=\left(1+\frac{\sigma_1}{X}\right)\left(1+\frac{\sigma_1}{X'}\right)+\left(1+\frac{\sigma_2}{Y}\right)\left(1+\frac{\sigma_2}{Y'}\right)-1$$

- where c166 and c266 follows from oblique-grain tests based on the measured Cd and Ct values: F166 = Ct/X' + Cd/X; F266 = Ct/Y' + Cd/Y.
- The Norris equations are not generally valid and only apply for uni-axial loading, identical to the Hankinson equation with n = 2, when the right (mostly) fictive shear-strength is used. These equations thus should not be used any more.
- Therefore, for tapered beams and for all other cases with determining off-axis uni-axial strength, the general Hankinson equations for tension and compression (with n different from n = 2, depending on the measurements) should be used or the exact equations.

28-6-1 F Lam, H Yee, J D Barrett
Shear strength of Canadian softwood structural lumber

Abstract
An experimental study has been conducted to evaluate the longitudinal shear strength of Canadian softwood structural lumber using a two span five point bending test procedure. Three species groups, Douglas fir, Hem-fir and Spruce-Pine-Fir, have been considered. Approximately 40% of the failures can be attributed to shear failures. Two test configurations have been considered: test span to specimen depth ratios of 6 and 5. American Society for Testing and Materials (ASTM) shear block tests have been conducted to evaluate the shear strength of small clear specimens. Based on the ASTM shear block test results, finite element analyses coupled with Weibull weakest link theory have been used to predict the median shear failure loads. Good agreement between predicted and measured median failure loads has been observed.

Conclusions
An experimental study was undertaken to evaluate the longitudinal shear strength of three species groups of Canadian softwood structural lumber using a two span five point bending test procedure. Two test configurations were studied: span to depth ratios of 6:1 and 5:1. Longitudinal shear type failure was achieved in approximately 40% of the test cases. Based on the ASTM shear block test results, finite element analyses coupled with Weibull weakest link theory was used to predict the median shear failure loads. Predicted and measured median failure loads agreed well with an maximum under prediction error of 13%.

28-6-2 B Madsen
Shear strength of Douglas fir timbers

Preamble

This paper serves two purposes:

A)

It demonstrates that the presently used characteristic strengths for shear are unduly conservative.

It suggests that action be taken to change the design requirements for shear in the timber code to requirements which more correctly reflects the structural behavior of the material, such as the "Weakest Link Principle" and "Fracture Mechanics Considerations" proposed by Drs. J.D. Barrett and R.O. Foschi.

It also shows that the shear capacity in many cases can be doubled by providing an overhang equal to the beam depth.

B)

The case described shows that the code committees must be more careful not to demand unnecessarily stringent requirements which act as detriments to the timber industry, often in the form of lost markets.

In evaluating or accepting proposed new design methods, the committee must realistically investigate the relevance of the present code formulations and compare their soundness to that of the proposal.

In the case presented, it would needlessly have cost one Canadian railroad more than $ 200.000.000 to follow the CSA-086 design code because of the code committee's reluctance to accept changes.

Summary of paper

- For the load configuration with a cantilever at each end the bridge ties predominantly failed due to bending. Only 6 shear failures out of a potential 904 were observed. The 5th. percentile could not be established, due to lack of failures, but the 0.7 percentile level would appear to be greater than 4.7 MPa.
- For the flush condition, 47 shear failures were found amongst 557 tests. The 5th. percentile was 4.5 MPa.
- It was demonstrated that the cantilever portion pays a major role in increasing the shear capacity.
- When the ties were dapped they lost 24 % of their original bending capacity.
- The old dapped ties had -after 30 years service- a bending capacity 18 % less than the new dapped ties.
- A computer program based upon the fracture mechanics Mode II failure criterion was able to predict the test results for the condition to within 4 %.
- A design method was proposed based upon the computer analysis conducted by Dr. R.O. Foschi.
- The characteristic shear stress for Douglas Fir was found to be 4.5 MPa for the flush condition (in the Canadian code this value is multiplied by 0.8 i.e. 3.6 MPa. to allow for Duration of Load. The comparable shear value in the code for Glulam is 2.0 MPa and for timber 1.2 MPa.)

30-6-1 N Burger, P Glos
Strength relationships in structural timber subjected to bending and tension

Introduction

As a rule small, clear specimens have greater tension strength than bending strength. In early standards for the design of timber structures bending and tension were usually assigned the same design values for timber members. However, tests conducted from the mid-60ies onwards using specimens in structural dimensions have shown, that in comparison to bending strength tension strength is more strongly affected by material defects such as knots and related local grain deviations than originally assumed. Tension strength is therefore lower than bending strength in specimens in structural sizes.

Based on these findings the design values for timber members loaded in tension were gradually reduced for almost all structural timber design standards. Tensile strength (f_t) to-day is about 60 % in relation to bending strength (f_m) for all commonly used strength class systems for structural timber. The characteristic strength values for European strength classes according to EN 338, too, are based on a factor of $f_t/f_m = 0.6$. The relative values for permissible tension stresses according to DIN 1052 amount to between 0.57 and 0.71, in relation to the grade.

Literature references indicate that the relationship of strength values depends on timber quality and dimensions. The ratio of tension to bending strength tends to increase with rising timber quality and decreasing dimensions. Regarding the safety (for low timber quality) and the economy (for high timber quality) of timber structures it would be desirable to take these effects into consideration when determining characteristic strength values.

Discussion and conclusions

The present investigations involving structural timber from Central Europe (mainly spruce) confirm that the ratio between tensile and bending strength depends on timber quality. This relationship can be derived using brittle fracture theory and expressed in terms of dependence on bending strength.

According to the brittle fracture theory there is a constant relationship f_t/f_m which is independent of timber quality and dimensions. When taking into account the different material behaviour of timber loaded in tension and in bending by calculating different parameters for the strength distributions, the ratio of f_t/f_m increases with strength and can be said to be more realistic.

For 2,642 tension and 1,739 bending tests in 28 and 27 samples respectively using test specimens in structural dimensions (constant dimensions per sample) ungraded timber resulted in the relationship

$$f_t = 0,36 f_m^{1,20}$$

The bending tests were conducted with the weakest section located in the tension zone, so that a relationship in conformity with EN 384 based on bending tests involving random location of weak sections shows lower ratios.

With consideration of test conditions involving non-random positioning of weak sections the 5 percentiles of tension strength were related to the 7.5 percentiles of the bending strength. For strength graded timber there is a noticeable effect of both grading method and dimensions on the f_t/f_m ratio. There is no clear effect of timber quality on the ratio f_t/f_m in timber visually graded according to DIN 4074, while increase in ratio is remarkable for machine graded timber.

No size effect was found for ungraded timber. A more pronounced size effect in strength graded timber loaded in tension lead to a size effect, on average, of

$$\left(\frac{l_{mr} b_{mr}}{l_m b_m}\right)$$

for visually graded specimens tested according to EN 408. Due to a constant ratio of b/l the effects of test length or span and specimen width or height could not be determined separately. There is considerable scattering regarding the ratio f_t/f_m. For safety reasons the lower ratios were more strongly weighted when determining the relationship for characteristic strength values $f_{t,k}/f_{m,k}$. In conjunction with the test data on visually and machine graded timber from literature the following equation was derived:

$$f_{t,k} = 0,26 f_{m,k}^{1,28} \tag{14}$$

Table 4 compares characteristic tensile strength values for strength classes according to EN 338 derived from this equation with currently valid strength values. This reveals that tensile strength is being overrated as regards the lower strength classes while in the higher strength classes tension strength is being given too conservative. For economy and safety reasons proper consideration of the influence of timber quality would therefore be desirable, which would adequately highlight the particular advantages of machine graded timber.

Tab. 4.- Comparison of characteristic strength values for strength classes according to EN 338 for load in bending and in tension with tension strength values calculated acc. to eq. (14).

[N/mm²]	C 14	C 16	C 18	C 22	C 24	C 27	C 30	C35	C 40
$f_{m,k}$	14	16	18	22	24	27	30	35	40
$f_{t,k}$	8	10	11	13	14	16	18	21	24
$f_{t\,(14)}$	7.0	8.3	9.7	12.5	14.0	16.3	18.7	22.7	27.0

36-6-1 P Glos, J Denzler
Characteristic shear strength values based on tests according to EN 1193

Introduction

Until today the effect of wood properties on shear strength of softwood is not completely understood. The European standard EN 384 assumes a direct relationship between shear and bending strength: $f_{v,k} = 0,2 \cdot (f_{m,k})^{0,8}$.

The strength properties proposed for the strength classes according to EN 338 are based on the same relationship. This means that the characteristic shear strength values are increasing with higher strength classes i.e. with increasing bending strength and density.

In contrast to this, the German standard E DIN 1052 defines a constant characteristic shear strength value for all strength classes. Up to strength class C 24 this value is higher than the corresponding values in EN 338 and for strength classes C 30 and above the value is lower than the values in EN 338. This is justified by the fact that so far a positive influence of density on shear strength has not yet been established and that the greater knot-ratio in the lower strength classes has a positive effect on shear strength.

Moreover, the different characteristic shear strength values may be explained by the fact that so far no satisfactory test method is available covering all strength influencing factors such as the cross sectional dimensions, the sawing pattern and fissures. Meanwhile a European standard for the determination of shear strength exists. Originally, this method was given in EN 1193. By now this method is transferred to EN 408 without being changed. However, in this paper we refer to EN 1193.

This test method was agreed upon after EN 338 and EN 384 had been drafted. Therefore it seems necessary to carry out tests according to EN 1193 in order to find out whether the test results are in accordance with the characteristic values as given in EN 338 and EN 384.

Conclusions

1. The influence of the density on the characteristic shear strength as given in EN 338 is not confirmed. A constant characteristic shear strength value independent of density as given in E DIN 1052 seems to be more appropriate.
2. The test results do not match with the shear strength values given in both standards. The test results are generally higher. Basically this difference seems justified because EN 1193 does not contain all negative influence factors on shear strength such as fissures. However, the values specified in a product standard must correspond with test results based on the test standard referenced in the product standard. To overcome this discrepancy, the product standards should consider these additional effects by applying an appropriate reduction factor.
3. According to the test results, the shear strength values of glulam beams should not be much higher than the shear strength values of structural timber because of the mainly tangential growth ring orientation of the laminations. At present the characteristic shear strength value for glulam beams in E DIN 1052 is 30% higher than the one for structural timber.
4. With regard to size effects on shear strength the test results according to EN 1193 were compared with results in the literature of test pieces with bigger cross sections. Spengler tested the shear strength of test pieces with cross sections from 22 x 80 mm^2 to 32 x 140 mm^2. His results ranged from 3 to 7 N/mm^2. Therefore based on the data presently available no size effect on shear strength values can be noticed.
5. The test method according to EN 1193 does not include all influences on shear strength. The small size of the test pieces does not allow covering all kinds of fissures and knots. In order to avoid glueline failures in the test pieces/steel plates interface special attention has to be paid on the gluing process and to the appropriate selection of the adhesive. With the two-part epoxy used in this study hardly any glueline failures were observed.

37-6-4 F Rouger
A review of existing standards related to calculation of characteristic values of timber

Abstract

Different standards currently exist to calculate characteristic values for wood based materials. In the case of solid timber, the calculation involves sampling issues as well as statistical approaches which differ upon the standards. In all cases, the underlying methods have been mainly calibrated by using Monte-Carlo simulations. The purpose of this paper is to test these methods on a real database, to compare the results and to propose a unique approach that could give equivalence factors.

The database which has been used in this investigation comprises more than 10 000 test pieces, from 10 different species and 4 grades.

Four different standards have been investigated:

– EN 384: applicable for solid timber
– EN 14358 applicable for wood based products
– ISO 13910: applicable for solid timber
– ISO 12491 applicable to building products

EN 384 relies on stratified sampling and on ranking. ISO 13910 proposes to calculate the characteristic value as the lower bound estimate, given by Weibull distribution, of the sample fifth percentile, given by ranking. ISO 12491 proposes to calculate the 5' percentile as the lower bound estimate

by assuming a normal distribution. EN 14358 follows basically the same approach as ISO 12491, but assuming a log-normal distribution and an unknown coefficient of variation.

The investigations show that EN 14358 and EN 384 give the same results. If only one European standard is maintained, one suggests that EN 14358 is kept, since it is applicable to all wood-based products and that it is much easier to i use. We should only slightly modify it in order to have a minimum sampling (e.g. 200 specimens, made of 5 distinct samples).

Despite ISO 12491 is applicable to all building materials, we suggest not to use it, since it gives a big scatter.

In ISO 13910, there is an equivalence factor with EN 384, which varies from 1,0 to 1,3, depending on the strength property, the size and the quality of timber, taking into account different test methods. Our simulations show that the way of calculating the fifth percentiles gives another correction factor, which encounters the previous one. By combining both factors, we could use a straightforward correspondence between both standards.

Conclusions

The investigations show that EN 14358 and EN 384 give the same results. If only one European standard is maintained, one suggests that EN 14358 is kept, since it is applicable to all wood-based products and that it is much easier to use. We should only slightly modify it in order to have a minimum sampling (e.g. 200 specimens, made of 5 distinct samples).

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2.11 SHEAR

13-6-3 J D Barrett, R O Foschi
Consideration of shear strength on end-cracked beams

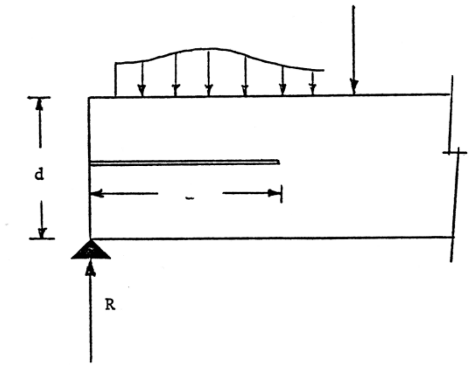


FIGURE 1

Figure 1 shows a case of a beam with an end-crack (split) of length a. The reaction at the support where the crack is present is *R*, and the cross-sectional dimensions of the beam are, respectively, *b* and *d*.

The stress intensity factor K_{II} at the tip of the crack for such a configuration has been studied by Barrett and Foschi and Murphy and can be expressed as follows:

$$K_{II} = 2,8\frac{Ra}{bd^{1,5}} + 0,75\frac{R}{bd^{0,5}} \tag{1}$$

According to the hypothesis of Linear Elastic Fracture Mechanics, the end-crack will propagate in a shear mode when the stress intensity factor K_{II} equals a critical value K_{IIc} . If the shear stress τ at the support is defined as

$$\tau = 1,5\frac{R}{bd}$$

Eq.(1) can be re-written in terms of τ :

$$\tau = \frac{K_{II}}{d^{0,5}\left(1,867a/d + 0,500\right)}$$

Therefore, the design condition must be

$$\tau \le k_{crack} K_{IIc}$$

where the factor k_{crack} is given by

$$k_{crack} = \frac{1}{d^{0,5}\left(1,867a/d + 0,500\right)}$$

and K_{IIc} is the characteristic value for the critical stress intensity factor. This characteristic value must be shown, along with other characteristic values. Unless determined by tests for different species, a conservative estimate is

$$K_{IIc} = 2,0 f_{ASTM}$$

where f_{ASTM} is the shear strength determined with the ASTM standard block.

A beam with an end-crack must also be verified for shear strength using the formulation for un-cracked beams, as the crack may be located over the support corresponding to the beam end least stressed in shear.

18-10-2 R O Foschi
Longitudinal shear design of glued laminated beams

This short commentary refers to the design recommendations for longitudinal shear in glued-laminated beams, as they are currently implemented in the Canadian Code CSA-086.

The design criteria are based on research conducted during the 1970's at the (then) Western Forest Products Laboratory in Vancouver (now Forintek Canada Corp.). This research was based on the premise that longitudinal shear failures are of a brittle nature and that, therefore, the statistical theory of strength of brittle materials could be used to interpret test data and to formulate design procedures.

The fundamentals of brittle fracture theory are discussed by V.V. Bolotin in his book "Statistical Methods in Structural Mechanics". The application of the theory to shear in timber has been shown in two publications in the Canadian Journal of Civil Engineering. The first of these discussed the theory and the second focusses on the derivation of a design procedure, which is essentially identical to the one adopted for CSA-086.

19-12-3 F Colling
Influence of volume and stress distribution on the shear strength and tensile strength perpendicular to grain

See 2.13 Size and configuration factors

24-10-1 R H Leicester, F G Young
Shear strength of continuous beams

Introduction
Shear strength tends to be more important in the design of continuous rather than single span beams. For example, in the case of centre-point-loaded beams having a span to depth ratio of 6:1, the ratio of nominal (applied) shear stress f_s to nominal (applied) bending stress f_b is given as follows

single span: $f_s/f_b = 0.083$ double span: $f_s/f_b = 0.152$

Because of this, there has been a proposal in Australia, that double span test specimens should be used for in-grade shear strength measurements. However, in some recent studies using LVL (laminated veneer lumber) some anomalies were noted in short-span shear tests, including the fact that shear failures appeared to be inhibited in double-span tests. Some aspects of these tests are reported herein.

Discussion
Two possible causes for the observed anomalies are as follows:

(i) Because of the short span/depth ratio used in the double span test specimen, the true values of peak shear and bending stresses could be significantly different from the nominal values.
(ii) Failure in shear at the mid-support of the double span beam cannot (because of symmetry considerations) develop as a longitudinal split as is normally observed for single span specimens,

27-10-1 R Görlacher, J Kurth
Determination of shear modulus

Introduction

In prEN 338 "Structural timber - Strength classes" and prEN 1194 "Timber structures - Glued laminated timber - Strength classes and determination of characteristic values" the ratio of the mean modulus of elasticity $E_{0,mean}$ to the mean shear modulus G_{mean} is assumed to be constant for all strength classes with

$E_{0,\mathrm{mean}} / G_{\mathrm{mean}} = 16$

This assumption presumes that material properties such as density or knots-area ratio (KAR) have no or equal influence on both the E- and the G- value; otherwise, the E/G ratio can not be constant for all strength classes.

Test methods for determining the modulus of elasticity and the shear modulus are specified in EN 408. The test specimen shall normally have a minimum span of 18 times the depth of the cross section. The values of E and G then represent average values over the whole test specimen length.

In a former research project on glulam beams the modulus of elasticity of board sections of 15 cm length was determined in order to obtain data about the variation of E along the board length. A special test procedure based on vibration time measurements was used.

Conclusions
Tests with boards lead to the finding that the ratio E_0/G depends on the modulus of elasticity. With increasing modulus of elasticity the ratio E_0/G is also increasing. Since in EN 338 the modulus of elasticity is assumed to be different in each strength class, the E_0/G value should also be different in each strength class. A proposal is given in Table 1.

Table 1 Comparison of test results with assumptions in prEN 338

According to EN 338				Test results	
Strength class	$E_{0,\mathrm{mean}}$ N/mm^2	E_0/G	G_{mean} N/mm^2	E_0/G	G N/mm^2
C18	9000	16	560	16	580
C22	10000	16	630	17	600
C24	11000	16	680	18	620
C30	12000	16	750	19	640
C35	13000	16	810	20	650
C40	14000	16	880	21	660

It should be mentioned that the tests were carried out only on test specimens with dimensions of cross sections of boards with thicknesses up to 35 mm. Further investigations are desirable with other cross-sectional dimensions. In any case, the proposal could be realistic for the strength classes of glued laminated timber, as these beams are made of boards.

31-21-1 G Schickhofer, B Obermayr
Development of an optimised test configuration to determine shear strength of glued laminated timber

Abstract
Based on international known research projects on the determination of shear strength of timber we try to define an optimised test configuration for GLT. Research on this field restricts to solid timber cross-sections of small dimension. There is a lack of vast research on cross-sections of bigger dimension and hence a secure determination of characteristic shear strength values for GLT can not be provided. Additional to the ordinary influences on the specimen the respective test configuration exercises an very important influence on the determination of shear strength in a test. Literature mentions various configurations in this field. Moreover we have to face the problem of the combination of shear, bending and compression (also tension) perpendicular to grain stresses. All these influences lead to a reduction of the percentage of specimens failed in shear and require therefore a higher test effort (enlarged number of specimens).

To avoid that fact we examine one test configuration - the I-cross-section form. Additional to two test series (2 · 6 =12 specimens) with different depths *(h* = 608mm and *h* = 320mm) we do finite element analysis for the respective test configuration to find out the distribution of shear stress and of compression stress perpendicular to grain within the range of support. This proceeding is necessary since some of the specimens did not show a clear failure in shear as reason for fracture. As a result of our work, referring to compression perpendicular to grain within the range of support, we present an optimised I-cross-section and a rectangular cross-section with reinforced flanges. The depth of reference amounted to h=608mm and *h* =592mm. The relation of span *L* to depth *H* of this three point bending test configuration was 5 : 1. Three m ore series (3 + 3 + 6 = 12 beams) with the respective dimensions of reference were tested, to verify the optimised results of the series 1 and 2.

Conclusions
By doing thise tests we aimed at the development of a test configuration which had structural size and reached a high number of failures in shear (21 of 24 specimens: 87.5%). When we chose the I-cross section we were able to fulfil both requests.

The depth of reference of 608 mm, the test beams had, corresponded to prEN 1194 of 600mm for test of accordance as far as grading is concerned based on the characteristic values of bending strength.

Failures in bending could be almost excluded by the use of an I-cross-section. The problem of this configuration is represented by the concentrations of compression perpendicular to grain between bottom chord and web. Those concentrations are not the only reason for a failure, but they cause a premature failure in shear. The test pointed out that first of all the supreme bottom chord is concerned. The problem, however, can be reduced by arranging the annual rings in the right way (right side down). Another but more laborious way is to round the edges between bottom chord and web, so that the effect of notch can be reduced. We adopt this method mainly for solid timber, since in that case I-cross-section is milled out.

Moreover we favour the system of single span three point bending with a relation between span and depth of 5:1.The advantage of that system compared to the two span five point bending test configuration is that the problem of the compression perpendicular to grain can be obviously reduced by means of supporting rolls. Moreover the test is not by a failure at the edge of the beam. Tests made by R.H. Leicester and F. G. Young prove that the system of two span five point bending test configuration show elevated values because of a restrained failure in shear. Their tests on failures in shear of laminated veneer lumber had examined both systems. According to their results the two span five point bending configuration reached a shear strength of 40% superior to that one of the single span three point bending test configuration.

The configuration can be used for solid timber and GLT of any size, as well, so that an influence of volume gets realisable.

The following relations are indicatives for the determination of test beams:
– H/L: 5:1
– H/H_F: 5:1

– W_F/Ww: 2:1

The single span three point bending test configuration with rectangular cross-section and reinforced chord represents possibly an alternative to the I-cross-section. Mainly there doesn't rise any problem of compression perpendicular to grain between bottom chord and web. The problem of compression perpendicular to the grain in the range of the loading can be solved by lengthening the loading plate. More information has to be gathered, however, on how the mechanical properties of the material us to reinforce the flanges influence the strength values which had been quite various at the tests

35-21-1 B Yeh, T G Williamson
Full-scale edgewise shear tests for laminated veneer lumber

Abstract
The shear strength of laminated veneer lumber (LVL) has traditionally been determined based on the results of small block shear tests conducted in accordance with ASTM D 143. In recent years, there has been a significant interest in determining the shear strength of engineered wood products using full-scale bending test methods in lieu of small block shear tests. However, due primarily to different shear-to-bending strength ratios among a variety of engineered wood products, the use of a prismatic cross section and test setup similar to those adopted for full-scale shear tests of glulam does not normally produce an acceptable shear failure rate in the edgewise or joist orientation (loads are applied parallel to gluelines), as required for LVL. Therefore, special considerations should be given to the test setup and specimen configuration for LVL edgewise shear tests.

Conclusions
The following conclusions can be substantiated by the test results presented above:
– The full-scale test method used in this study is adequate for qualification of the edgewise shear strength of LVL.
– The difference in the characteristic shear stresses between the specimens tested at the standard environmental conditions (9.0% moisture content in this study) and as-received conditions (6.8% moisture content in this study) are negligible.
– The mean shear strength derived from the ASTM D 143 small block shear tests is approximately 50% higher than the mean shear strength determined from full-scale shear tests, suggesting the necessity of considering a size effect.
– The small-scale QA specimen configuration and test setup used in this study can be used for edgewise QA shear tests.

38-6-3 H Klapp, H Brüninghoff
Shear strength of glued laminated timber

Introduction
The current knowledge in timber research gives no applicable computation method for the shear bearing capacity of glued laminated timber beams. The dependence of the shear strength of the lamination tensile strength, set in EN 1194, could not be confirmed by the tests of Schickhofer in Graz, so that in the new German standard DIN 1052 instead a constant value of 3,5 N/mm^2 was set. This value was determined from the test results computationally as five percent quantile, whereby it is to be stated that about 10 percent of all test results fell below the strength of 3,5 N/mm^2. In the context of statistics, this doesn't seem to be alarming. But a negative influence of the volume on the shear bearing capacity is to be assumed as in the case of the likewise brittle transverse tension failure. The tests mentioned were conducted on comparatively small beams, so that lower shear strengths for dimensions of practical use can be supposed.

In view of this uncertainty concerning the shear strengths as well as the current cases of damage, there is an urgent need for further research. However larger test series with practical dimensions are costly in terms of resources and labour. For this reason the alternative solution of a simulation model, which considers the varying material properties, was selected.

Simulation model
The simulation model is based on the assumption that the shear strength of glued laminated timber depends on the properties of the laminations. In contrast to EN 1194 a dependence not on the tensile strength is supposed, but on shear strength. Furthermore brittle fracture behaviour is applied by assuming failure as soon as the shear strength of the lamination is reached. According to this the shear strength of a glulam beam can be easily determined by comparing the shear stresses of the laminations with the corre-

sponding shear capacities with consideration of the varying properties.

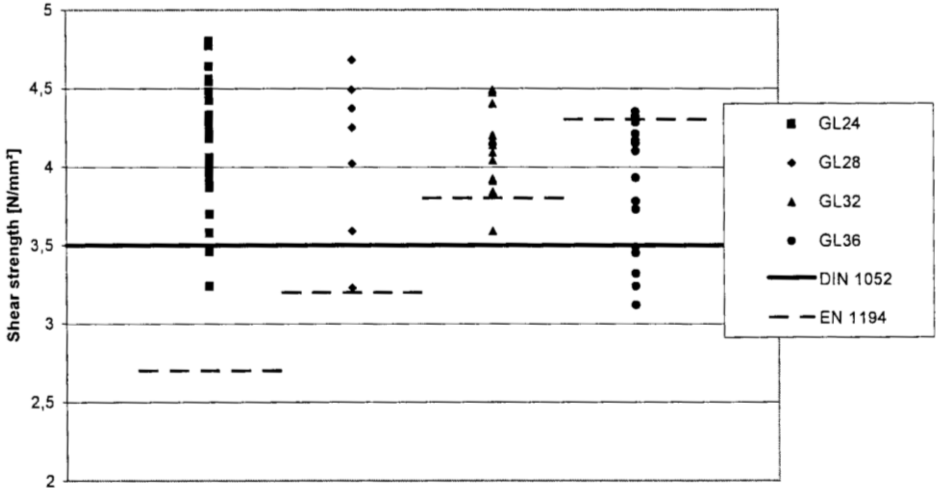


Fig. 1.1: Comparison of the shear strengths from tests of Schickhofer and current standards.

In the face of this simple failure criterion it is obvious that the quality of the results depends on an appropriate choice of the laminations shear strength. Shear tests (EN 408) on 272 timber specimens were carried out by Glos and Denzler and the following findings can be pointed out:

– The shear strength depends slightly on the density
– An influence of knots on the shear strength could not be determined
– Higher values for shear strength in higher strength classes could not be confirmed

According to this it is not necessary to distinguish between strength classes. The tests of Glos and Denzler as well as tests of Spengler on timber laminations lead to a mean value of 5,3 N/mm^2 and a 5%-quantile of 3,8 N/mm^2. With application of these values and the normal distribution a good accordance with the test results of Glos and Denzler can be achieved (Fig. 2.1).

Conclusions

A mathematical model was used to simulate the shear strength capacity of glued laminated timber beams. The varying properties of the material and the influence of different dimensions were introduced. Good correspondence between the results of simulation and tests made in Graz were

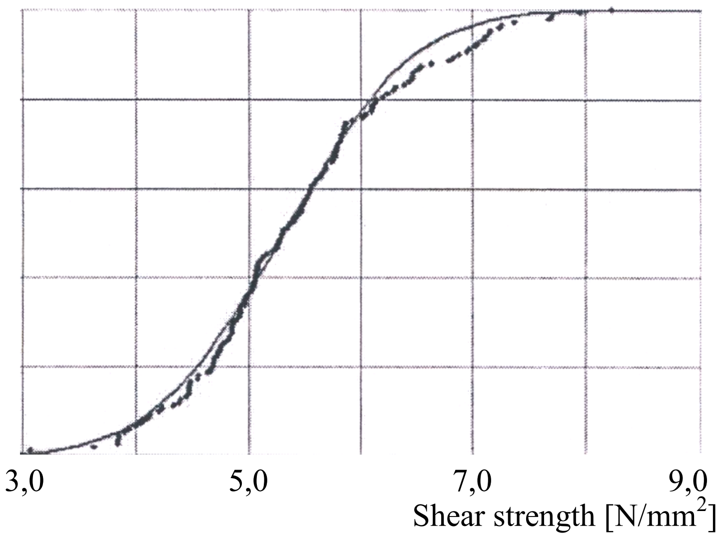


Fig. 2.1:Comparison of normal distribution and tests

found. These tests by Schickhofer have been made with relatively small specimens. Assuming that the simulation procedure developed here is correct a high influence of volume effect is given on the bearing capacity of big glulam beams. This effect has not been taken into account in the present European standards. Big glulam beams are overestimated in their real bearing capacity.

Based on a large amount of simulations in this study a design method is proposed, which improves the current rules and is easily to handle in practice. Caused by the assumptions of the model it is possible that the proposed method is conservative. A brittle failure of the beams was assumed and a rearrangement of shear forces in load bearing areas was not taken into account.

Some further simulation calculations have been made with different coefficients of variation (COV). The characteristic strength was adjusted; the other results did not change in principle. It is recommended to perform more tests with glulam beams to get reliable shear strength for a reference beam. As alternative the shear strength of laminations could be defined, serving as a basis for further simulations.

40-6-2 M Poussa, P Tukiainen, A Ranta-Maunus
Experimental study of compression and shear strength of spruce timber

See 2.1 Compression perpendicular to grain

43-12-2 R Crocetti, P J Gustafsson, H Danielsson, A Emilsson, S Ormarsson
Experimental and numercial investigation on the shear strength of glulam

Introduction
According to EC5, the shear resistance of a structural timber element should be determined on the basis of the characteristic shear strength of the material, along with classical beam theory. For glulam, the characteristic strength values are given by the European standard EN 1194 which assumes a direct relationship between tensile strength and shear strength of the lamination

$$f_{v,k}=0.32\left(f_{t,0,k}\right)^{0.8}$$

As an example, the characteristic shear strength of glulam class GL28c, consisting of inner laminations with characteristic tensile strength $f_{t,0,k}$ = 14.5 MPa, would be

$$f_{v,k}=0.32\left(14.5\right)^{0.8}=2{,}9\ \text{MPa}$$

However, recent investigations both on glulam members [4] and on timber members have shown that the shear strength of spruce is higher than the shear strength obtained by means of the model proposed by EN1194. Moreover, the studies show that the shear strength is nearly constant, regardless the strength class of the timber material.

Discussion and conclusions
The proposed test method, i.e. the modified EN 408 with two 155 mm cuts at the bottom part and at the upper part has the advantage, that it allows for a state of stress in the region between the cut tips characterized by predominant longitudinal shear (i.e. shear parallel to the grain). Moreover, the presence of the cuts significantly reduces the tension perpendicular to grain at the corners of the specimen. The disadvantage of such a test method could be the high stress concentration that occurs at the cut tips, which may negatively influence the shear strength of the timber On the other hand, the cuts oblige" the failure surface to occur at a given plane, which may positively influence the shear strength of timber. Specimens with width between 40 mm and 215 mm were shear tested according to modified EN 408. Similar shear strengths were achieved for all these specimens, indicating that the width of the specimen does not have a remarkable influence on shear strength.

In general it can be stated that testing according to modified EN 408 gives shear strength lower than shear strength obtained by beam testing. However, if the beam specimens with applied compression at the top of the support and the beam specimens on three supports were excluded, the characteristic shear strengths obtained by the two methods would become comparable.

For the modified EN 408 tests, no remarkable shear strength differences between specimens with rectangular cross section and specimens with I-cross section could be observed. On the other hand higher shear strength could be observed for beams with 1-cross section than for beams with rectangular cross section. The reason for such a discrepancy could be the possible beneficial influence of higher perpendicular to grain compression in the web of the 1-beams. Moreover, it should be observed that 50% of the beam specimens with rectangular cross section failed in bending. Perhaps, if bending failure had been prevented, shear.
Four specimens were tested according to modified EN 408 method, loading in the radial direction instead of loading in the tangential direction – as occurred in all other cases. It was observed that the shear strength in the radial direction was slightly lower than the shear strength for specimens loaded in the tangential direction. This is in line with the results by Dahl et al, but not with the results obtained by all the other authors cited in this paper. However, the number of specimens loaded in the radial direction is too low for drawing any relevant conclusions.

Overhangs do not seem to have any evident effect on the shear strength for the tested timber beams. Neither, has the action of a external tension applied on the upper part of the beam at the support, in the direction perpendicular to grain. On the other hand, external perpendicular to grain compression stress applied on the upper part of the beam at the support seem to affect the shear strength positively. However, this conclusions cannot be extended to cases were the applied external tension or compression are higher than those used for these experiments.

In the case of beam with three supports higher shear strength than the shear strength for simply supported beams was observed. This may be due to the fact that at the intermediate support the transmission of internal forces occurs in a great extent by "strut action" and the zones with pure shear action are rather small.

Table 9. Comparison of test results with EN338 values (MPa)

	Visual grade	Test result EN408	EN338	Alternative test result
$f_{c,0,k}$	C18 C24 C30 all	22,5 26,2 29,8 27,9	18 21 23	
$f_{c,90,k}$	all	2,2	2,6 (C27)	2,9 beam end, both edges loaded 3,7 middle support, both edges loaded 5,2 ASTM D 143
$f_{v,k}$	all	3,9	2,8 (C27)	7,2 I-beam

2.12 SIZE AND CONFIGURATION FACTORS

13-6-2 R O Foschi, J D Barrett
Consideration of size effects in longitudinal shear strength for uncracked beams

Two recent publications in the Canadian Journal of Civil Engineering report on research conducted at the Western Forest Products Laboratory on size effects in the longitudinal shear strength of uncracked beams. Results from this research form the basis for new design recommendations in the Canadian Code CSA-086 (1980).

In the context of the CIB Code, Clause 5.1.1.4 specifies that the shear stresses, τ , calculated according to the theory of elasticity, shall satisfy the condition

$$\tau < f_v \tag{1}$$

where f_v is the characteristic value given in Table 5.1.0.a (with the appropriate modification factors) and τ is the maximum shear stress

$$\tau = 1.5\, Q_M / A \tag{2}$$

where Q_M is the maximum shear force and A the beam cross-section.

The characteristic value f_v changes with the volume of the beam and the loading condition can be written as follows:

$$f_v = \beta f_{v,0} \left(\frac{V_0}{AL} \right)^{1/k} \frac{Q_M}{Q_0} \tag{3}$$

where

$f_{v,0}$ = characteristic shear strength value for the volume V_0,uniformly stressed and arbitrarily chosen (0.02 m^3 if chosen the same as for tension perpendicular-to-the-grain);
Q_0 = total load applied to the beam;
k = size effect coefficient;
β = factor dependent on the loading condition and the coefficient k;
AL = total beam volume, A being the cross-section and L the beam length.

The coefficient k must be obtained by tests. For Douglas-fir, k = 5.53; for Hem-fir [3], k = 5.94.

It may be assumed, for convenience, that k = 6.0 represents a value sufficiently accurate and approximately species-independent.

The loading condition factor β is obtained by integration of the shear stress distribution over the volume of the beam, and the background for this integration is given in. The general formulae for β is:

$$\beta = \left\{ F(k) \sum_{i=1}^{N} \left[\frac{L_i}{L} \int_{-1}^{1} \left(S_c \frac{Q_c(\xi)}{Q_o} + S_d \frac{Q_d(\xi)}{Q_o} \right)^k d\xi \right] \right\}^{-1/k} \tag{4}$$

where F(k) is the integral

$$F(k) = \frac{1}{4} \int_{-1}^{1} (1 - \xi^2)^k \, d\xi \tag{5}$$

16-6-1 A R Fewell
Size factors for timber bending and tension stresses

Introduction

The writer was asked to present a paper 1983 on size factors. Fewell and Curry have written a paper which analyses existing PRL and Canadian data to examine the effect of depth in the determination of characteristic bending stresses. That paper is to be published in The Structural Engineer but due to delays in publication some of the information in that paper has been outdated by committee decisions in producing the final draft of the revised Code of Practice for timber structures, BS 5268:Part 22. Rather than re-produce the data analysis of that paper, this short note updates it by pointing out the subsequent committee changes and adds information on the adjustment of bending and tension design stresses given in BS 5268.

Conclusions
From the results of this analysis it is concluded that:

- The effect of section depth on the characteristic bending stress of visually stress-graded softwood timber, in sections from about 100 to 300 mm deep, can be defined by the general equation
  $K = (200/h)^{0.4}$
  where K is the factor by which (a) the stress value obtained from tests on sections with a depth of h mm should be divided to obtain the stress corresponding to a depth of 200 mm or (b) the stress value for a section depth of 200 mm should be multiplied to obtain the corresponding stress for a section depth of h mm.
- In applying the general equation no distinction need be made between species or visual grade.

18-6-2 R H Leicester
Configuration factors for the bending strength of timber

Synopsis
Timber is a heterogeneous material containing dispersed defects. As a result, the measured bending strength of timber depends on many test factors such as the length of the beam, the method of loading, and any bias used in selecting the test beam. The influence of these various factors is described in terms of two 'configuration' factors. The paper uses simple statistical concepts together with an examination of characteristics measured by direct test to provide an insight into these factors. It is concluded that the broad features of configuration factors arc adequately described by simple statistical models, but that more sophisticated models are desirable for practical purposes.

Concluding discussion
Some insight into the configuration factors associated with the measurement of bending strength has been obtained through examination of the test data and the application of simple statistical concepts which were developed to predict the effect of the loading configuration. These studies indicate that configuration factors encountered in typical practical situations are sufficiently large that they should not be ignored; the various factors studied can be cumulative.

Although the effectiveness of the test data cited is limited by the small size of the samples used, they are sufficient to demonstrate that while the simple statistical models used are adequate to describe the broad features of configuration factors, they are not accurate enough to predict the magnitudes of the low percentiles with an accuracy sufficient for practical applications.

It is apparent then, that to make further progress it will be necessary to test larger samples of timber and also to develop statistical models of strength that can describe the characteristics of timber that contains dispersed isolated defects,

18-6-4 B Madsen, A H Buchanan
Size effects in timber explained by a modified weakest link theory

Abstract
Size effects in timber have been included in the design process for some of its minor strength properties (shear, tension perp.). This paper demonstrates that size effects should also be considered when designing with its major properties (tension, bending and compression).

Using a modification to the weakest link theory, reflecting the anisotropic nature of timber, good agreement has been obtained between tests and the theory. The theory enables us to compare tests with different spans, and/or load configuration with an understanding not previously possible.

Results from several large testing programs have been used to quantify some of the necessary size effect parameters. A simple design method for bending members, taking length effects and load configuration effects into account, has been proposed.

Information for tension members is less comprehensive; nevertheless a tentative suggestion for the design of those members has been included.

Summary and conclusions
Various size effects observed in lumber have been explained by a modified weakest link theory. The modifications to the traditional weakest link theory consist of assigning different size effect parameters to the different directions of the anisotropic material.

Some of the salient parameters have been quantified based upon analyses from several large testing programs.

While the estimates of the size effect parameters varied considerably a sensitivity analysis showed that the value of the length effect parameter k = 3.5 gave sufficient precision for engineering purposes.

The concept allows comparisons of test with different spans and/or load ration to be made resulting in greater conformity between them than previously possible.

A standard beam configuration (3 m span, 1/3 point loading) is suggested to be used as the base for referencing bending strength of lumber.

A design method is proposed for inclusion in timber design codes which takes into account both length effect and load configuration effect in a simple manner.

Tentative suggestions are also made for design of tension members but further work in this area is urgently needed.

18-6-5 H Riberholt
Placement and selection of growth defects in test specimens

Preface

During the investigation of the significance of the placement of defects in test specimens it was revealed that the strength depends on the definition of the structural element for which it is applicable.

In this paper the strength of two different elements has been defined:

– The strength of a cross section in a beam element of wood
– The strength of a structural element, typically a beam or a bar.

Further, some aspects were found in the definition of the strength of elements which can be characterized as unclear or may be arbitrary. A discussion should be initiated on how the tests can be performed, and how the test results can be transformed, so that they are applicable for the modelling of real structures. This paper treats some of the aspects which should be taken up in the discussion.

Recommendation

Due to the fact that the strength of the cross sections varies along the length of boards, planks or lumber the placement of growth defects will influence the result of strength tests.

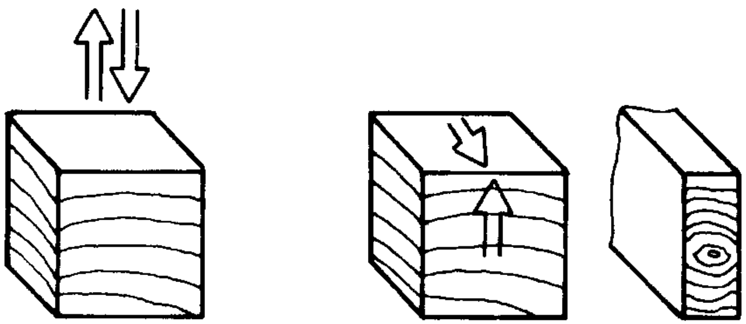
It is recommended that the placement of growth defects is done according to the table below. The recommendation is intended used for coniferous wood.

The selection of the worst growth defect, i.e. the weakest cross section, in a test specimen is done by an estimation. This can be based either on visual judgment or on non-destructive measurements.

It is recommended that the growth defects are selected so that the size of the defects belongs to the whole interval between two grade limits. This should reflect practice, and it would be conservative to tend to have defects close to the maximum allowable limit.

For shear and compression or tension perpendicular to the grain, it is recommended to use specimens either with small growth defects or made of clear wood. The load should be applied so that the loose spring wood will become decisive for the strength. Figure 1.1 shows the recommended load directions.

Test type	Placement of the worst growth defect
Bending	In the moment span in the tension side
Compression parallel to grain	At the middle of the length of the test specimen
Tension parallel to grain	At the middle of the length of the test specimen



Tension or compression Shear Plank cut for shear test

Figure 1.1 Direction of applied stress or force in test specimens.

19-12-3 F Colling
Influence of volume and stress distribution on the shear strength and tensile strength perpendicular to grain

Abstract
In 1939 Weibull developed a theory, which allows to estimate the influence of the size of the stressed volume and the stress-distribution over this volume on the strength of homogeneous, isotropic materials with brittle fracture behaviour. Although wood as material is neither homogeneous nor isotropic, it has been shown, that the application of Weibull's theory is possible. Weibull's theory has even entered several design codes (CIB-structural timber design code, Euro-Code 5, Canadian code ...). It is used especially in the case of shear and tension perpendicular to the grain, not least because of the existing brittle fracture behaviour.

In this paper, the application of Weibull's theory in the case of shear and tension perpendicular to the grain is discussed.

Summary and conclusions
Weibull's theory of brittle fracture is used to describe the influence of the stress distribution and the size of the stressed volume on the strength of a beam.

The determination of the so-called fullness-parameters A (which stand for the fullness of the stress-distribution) is shown. Also a mathematical relationship between the expected ratio of the strength of two beams and their fullness-parameters and their stressed volume has been deducted. The application of this theory and a possible design method has been shown in the case of shear stress and tensile stress perpendicular to the grain.

Because of the differences between the theoretical and the experimental results in the case of tensile stress perpendicular to the grain, further investigations are required. The application of a modified weakest link theory (with weighted influences of the beam-length and depth) as well as the further dependency of the strength on the wood-properties (density, growth rings, knots ...) will probably be investigated in a proposed research program in Karlsruhe.

22-6-1 J D Barrett, H Griffin
Size effects and property relationships for Canadian 2-inch dimension lumber

Introduction
Strength of structural lumber evaluated on a full-size basis, varies with the member dimensions. In full-size members, size-effects are significantly greater than previously recognized in the development of design values based on small clear specimen tests. Size effects observed from timber tests are of such significance that size factors have been introduced explicitly in design codes such as the Canadian Code for Engineering Design in Wood (CANS 086-M84) and the British Standard BS 5268 Part 2 (BSI, 1984). New size adjustment factors are being proposed in design property evaluation standards including the ASTM standard under development for evaluation of design properties of structural lumber based on full-size tests and the CEN standard for determination of characteristic values of mechanical properties of timber.

The Canadian lumber industry sponsored, through the Canadian Wood Council's (CWC) Lumber Properties Project, a comprehensive evaluation of strength properties of on-grade dimension lumber samples for a range of grades and widths commonly produced in Canada. This paper presents an analysis of the influence of member size on the bending, compression and tension strength derived using results from tests of 38 mm thick dimension lumber.

Conclusions
The CWC Lumber properties data base provides large, property matched data sets suitable for analysis of size effects factors for Canadian structural dimension lumber.

The results of the size effects analysis suggest that:

- Bending, tension and compression strength properties can be significantly affected by changes in member width and length. Width effects in bending and tension members appear similar while length effects are somewhat greater in bending than tension. Width and length effects are smallest for compression members.
- Comparison of tension and bending size parameters derived herein show similar results for Canadian and US commercial species groups which strengthens the case for applying a common set of size parame-

ters for all grades and species. Size parameters are also independent of property level. The only major inconsistency observed in the results, as presented in this paper, is the difference in size factors for compression which may be due to the significant size difference in test specimens employed.

- Size factors for adjusting test data or modifying code design properties size must be expressed in a manner consistent with the associated test standards. If test standards employ members of constant length to width ratios then the size factors will be different than would be employed with constant length test members.
- The CWC test data suggests that a size parameter of the order of $S = 0.2$ could be applied for bending and tension width and length effects. The size parameter $S = 0.1$ could be applied for compression width and length effects. These size factors would provide a basis for developing a set of simplified expressions for adjusting tension, bending and compression strength data for both data interpretation and code docurnents.

22-12-1 M Badstube, W Rug, W Schöne
The dependence of the bending strength on the glued laminated timber girder depth

Introduction

With a view to determining the influence of the girder depth on the bending strength of glued laminated' timber, experimental investigations and tests are being carried out. It is the objective of these tests and investigations to find out data and information concerning the 'application of the "depth dependence" modification factor k_h (depth factor) for glued laminated timber

Summary

The evaluation of the bibliography (publications) and our own specific tests and investigations performed by means of glued 1pminated timber girders are demonstrating that for the failure type A ("finger joint") failure within the stressed zone) there doesn't exist any interrelation between the girder depth and bending strength.

As for the failure type B (knot or timber failure within the -heavily stressed zone), a statistically covered and verified interrelation between the stressed volume or girder depth and the bending strength can be determined:

The tests are verifying the depth factor as indicated in the Eurocode 5 for the failure type B.

23-10-3 J D Barret, A R Fewell
Size factors for the bending and tension strength of structural timber

Introduction

For many years it has been recognized that the bending strength, and more recently tension strength, of timber are affected by the size of the specimen. While this effect may in reality be associated with the stressed volume, grade, and the size and age of the tree from which it was cut, it is generally described as a depth effect (for bending) or a width effect (for tension).

This paper examines the available test data to determine the effect of length and depth or width on bending and tension strength and provides depth and width factors applicable to Eurocode 5 and the supporting CEN standards.

Conclusions and recommendations

From an analysis of test data comprising many different grades and species, the following conclusions were reached:

- The factor (k) for adjusting characteristic values in codes and standards for both the depth effect on bending strength and the width effect on tension strength, when each in property is based on a constant span to depth ratio, is given by
  $k = (200/h)^{0.4}$
  where h is the depth or width of the member for which the strength value is required.
- In using the factor given above a minimum size of around 35mm x 47mm needs to be specified for tension members.
- The factor (k_W) for adjusting tension or bending stresses to other depths or widths, when the length remains constant, is given by
  $k_W = (A/B)^{0.23}$
  where A is the width or depth relevant to the stress value to be adjusted and B is the width or depth relevant to the required stress value.

- The factor (k_L) for adjusting bending and tension stresses to other member lengths when the width remains constant, is given by
  $k_L = (A / B)^{0.17}$
  where A is the length relevant to the stress value to be adjusted and B is the length relevant to the required stress value.

24-6-3 I Czmoch, S Thelandersson, H J Larsen
Effect of within member variability on bending strength of structural timber

Introduction
In this paper a simple model of the length wise variation of strength of a piece of timber is used. The strength is modelled by means of composite random point series: Random series of strength are assigned to randomly distributed weak zones.

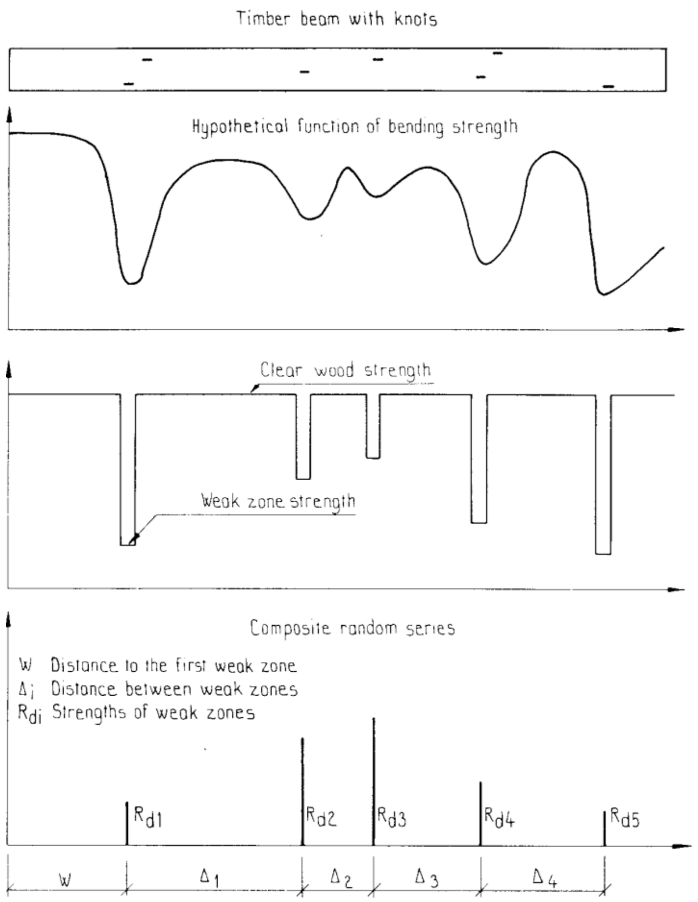
It is assumed that
- timber is composed of short weak zones connected by sections of clear wood
- the weak zones correspond to knots or groups of knots and are randomly distributed
- failure occurs only in the middle of the weak zones
- the strengths of the weak zones are random.

The basis for this model is the fact that failure almost always occurs in the vicinity of knots because of grain distortions around knots resulting in stresses perpendicular to the grain, stress concentrations caused by knot holes and encased knots, and because of differences between the properties of the knot and the surrounding wood.

The model can be used to evaluate
- the influence of length on the strength of timber members
- the influence of load configurations deviating from the standard test configuration used for assigning characteristic strength values
- the influence of different test procedures e.g. the difference between North American practice (where the length to be tested is chosen randomly) and European practice (where Eurocode 5 prescribes that the tested length shall contain a grade determining defect)
- the influence of the test procedure on the reliability parameters (e.g. safety index).

Standard test procedures result in extreme value distributions. If they are used instead of the parent distribution (i.e. the distribution of the strength of the weak zones), the reliability of timber structures will be higher than for material like steel and reinforced concrete for the same safety index or partial safety factors.

Comments
The suggested procedure will give a result which is very close to the minimum strength over a length equal to the original length of the boards. The result will depend on this length, and an adjustment with respect to this length might be considered in the evaluation of the characteristic value.



Modelling of length-wise distribution of strength of timber beam.

25-10-2 T D G Canisius
The influence of the elastic modulus on the simulated bending strength of hyperstatic timber beams

See 3.14 System effects

25-6-1 T D G Canisius
Moment configuration factors for simple beams

Abstract

In the design of a timber beam, its strength is assumed to be constant along the length and to be equal to the (factored) characteristic strength. This can give rise to an unnecessary increase in the safety level of the member and a consequent reduction in economy. This effect may be somewhat reduced by the application of a Moment Configuration Factor to modify the design formula. In order to find these factors, which depend on the bending moment diagram, it is necessary to determine the strengths of beams under different load and support conditions. This paper presents a theoretical investigation of moment configuration factors for some simple beams.

Conclusion

A study of moment configuration factors (MCFs) for simple beams were presented in this paper. Several statically determinate beams were computer simulated while considering the spatial correlation of the strength properties. The method of Taylor and Bender, based on different property element lengths, was used in generating the beam properties. Also some results obtained with the Riberholt-Madsen model was presented.

The results obtained by the use of the present model showed that for a given beam length, the MCF values are almost constant under small correlations. The MCFs on the fifth percentiles were smaller than those on the expected values when the correlation coefficient was smaller. The presence of correlation, expressed in terms of the property element length, provided lower MCFs than those obtained by methods based on the assumption of statistical independence.

The length effect on the MCFs for a simply supported beam with a central concentrated load was seen to be negligible under small correlations. Increased correlations in terms of the property element length only increased the MCF values when the beam length was doubled. Hence if no length effect is to be considered, and correlations are to be present, then it is advisable to base the factors on a small beam length where the latter can be important. Otherwise, lower levels of safety may result.

As the length effects on the strengths of clamped beams are said to be different from that of a simply supported beam, also the MCFs for them should vary with the beam length. Therefore, it is advisable to base these also on a reference beam length. Otherwise, there will be a loss of economy in the case of longer beams and a higher risk of failure in the case of beams shorter than the reference length.

It is expected to carry out further theoretical studies with respect to these, but at least some of them should be verified experimentally for further confidence in results.

25-6-5 J D Barrett, F Lam, W Lau
Size effects in visually graded softwood structural lumber

Abstract
A brittle fracture model has been evaluated for predicting the variation in bending, compression and tension parallel to grain strength of visually graded dimension lumber. Size effect factors for visually graded lumber bending, tension and compression strength are established. Size effect parameters for visually graded lumber are anisotropic. Size effects for changes in member width are typically greater than the size effects for changes in length. Relationships between width (S_w), length (S_L) and constant ratio (S_R) size factors have been introduced. Size factors derived from length effect tests are used to estimate ratios of tension strength to bending strength. Results agree closely with tension/bending property ratios published in the literature. Size factors for bending and tension are recommended for international harmonization of procedures for derivation of characteristic properties of visually graded lumber at standard sizes.

Conclusions
Brittle fracture theory provides a rational framework for evaluating the influence of member size and loading conditions on tension and bending strength properties of structural lumber. Weakest link brittle fracture theory was used to establish size factors for bending, tension and compression strength for visually graded softwood structural lumber. The results of the analysis support the following conclusions:

- Size effects in visually graded structural lumber are remarkably consistent. Hypothesis tests that specific size factors are equal across grades, species and property percentile level were not rejected at the 75 percent significance level.
- Length effect factors for tension (S_{Lt} = 0.17) and, bendirg (S_{Lb} = 0.17) are similar.
- Width effect factors calculated for bending S_{wb} = 0.28) are slightly higher than the tension factor (S_{wt} = 0,2) for Canadian commercial species groups.
- The length and width size factors for compression arc S_{Lc} = 0.10 and S_{wc} = 0. 11,
- The relationship between tension and bending strength for member of the same width can be determined using a weakest link failure model and a size factor S_{Lt} = S_{Lb}= 0.17.
- The effect of load condition on bending strength at a fixed width (depth) can be calculated using weakest link analysis and a length effect size factor S_L = 0.17.
- The size factors S_{Rb}, S_{At} and S_{Ac} derived in this study agree closely with those required to achieve constant reliability and therefore the same size factors may be used for both allowable stress design and load and resistance factor design code applications.
- For international harmonization of procedures for adjusting tension, bending and compression strength properties, the width and length factors can be taken to be equal. For bending and tension S_W = S_L = 0.2 and for compression S_W= S_L= 0.1 for softwood visually graded lumber.

27-6-2 F Rouger, T Fewell
Size effects in timber: novelty never ends

Introduction
Size effects in timber have been widely investigated by different authors. These authors apparently find results which are conflicting, and the end result is a compromise, among them which has been adopted for code purposes (Eurocode 5). Recently, a wide number of species has been investigated in France, using a forest based sampling. The results were analysed and provided additional information to the size effect problem. The purpose of this paper is to discuss the apparent discrepancy of the size effect.

Conclusions
In this paper, we demonstrated that the basic assumptions of the Weibull theory (brittle failure, statistical homogeneity) were not verified. We also demonstrated that the so-called "size effect" could be significantly influenced by the sawing pattern and by the visual grading method. For this reason, further research needs to be achieved to isolate and quantify the

different things that affect the size effect. A theory that undertakes the ductile behavior and the statistical heterogeneity of timber material has to be formulated. Further testing programs also need to keep a precise track of the sawing pattern, the log selection and the grading practice. These elements will help to provide a compromise in design codes.

28-6-4 T Isaksson, S Thelandersson
Effect of test standard, length and load configuration on bending strength of structural timber

Abstract

The within member variability of bending strength in structural timber has been investigated in an experimental study of 133 boards from Norway spruce (Picea Abies). For each board the bending strength was determined in 4-7 of the weakest sections along the length. The choice of test sections was based on knot measurements and readings from two different types of machine grading.

The results from the experimental investigation are used to compare three different standards (European, North American and Australian) for determining characteristic bending strength as well as to study length and load configuration effects on bending strength. These studies were made by direct simulation from the test data on one and the same sample, which makes this investigation unique compared to other studies of these problems.

It was found that the difference between the three test standards is very small for the tested material. In a statistical sense no significant difference could be found although the methods of selection of test section are quite different between the standards.

Conclusions

The following main conclusions can be drawn from the investigation reported in this paper:

– A reliable method of testing the same timber board in several sections along the length has been developed.
– The test method was used to determine the bending strength in 4-7 sections in a sample of 133 boards of structural timber from Norway spruce.
– These data were used to simulate length effects and load configuration effects as well as comparisons of different test standards (EN, ASTM and AS/NZ) on one and the same sample. In this way, statistical errors associated with lack of matching between samples can be eliminated.
– The difference between the test standards was found to be very small for the tested material. In a statistical sense no significant difference could be found, although the methods of selection of test section are quite different between the standards.
– The results from the simulated tests according to all three standards were significantly higher than the strength of the weakest section within the board both for 50th and the 5th percentile of the sample.
– The simulations showed very small length effects for the sample investigated. No significant difference could be shown between lengths of 2.2 in and 4.2 m, although both the mean and the 5th percentile for the longer beams were slightly smaller than for the shorter ones.
– The effect of load configuration was found to be statistically significant, however, when comparing by simulation beams of equal length loaded with constant moment, uniformly distributed load and point load at mid-span.
– The results indicate that the widely used Weibull concept used to describe length and load configuration effects has limited validity. The reason for this is that the Weibull theory assumes uniformly distributed defects with random properties over the volume of the body.

28-6-5 T Canisius, T Isaksson, S Thelandersson
Grading machine readings and their use in the calculation of moment configuration factors

Abstract

The Lund University is currently involved in a project for the determination of along-member variation of timber beam strengths. This paper present some preliminary results obtained through simulation based on the results from this project and grading machine readings for the beams to be simulated. This study is only a test of the method used for the simulation of beam strengths. The considered method seems to provide good prospects in predicting strengths of beams when grading machine readings are available.

Conclusions

The following conclusions can be made from the limited analyses carried out so far with respect to beam strengths.

- The present project on the determination of along-member beam strengths seems to provide good prospects for the calculation of moment configuration factors.
- It is desirable to gather information on the way and the extent to which the strength reductions due to defects may spread along a beam.
- It is advisable to have information on the possible strength coefficients of variation (COVs) with respect to the regression analysis predictions.
- It is necessary to study the effect of the possible subjectivity of manually selected defect positions on the generated strength results. This subjectivity occurs for beams with many smaller close depressions in the graph of force vs. the beam length.
- The determination of valleys in the grading machine readings, which may correspond to defects, need to be automated. For this purpose objective criteria need to be developed.
- Firmer conclusions can be made only when the strength predictions of all the 133 beams are simulated and a statistical analysis of results is carried out.

28-12-2 E Aasheim, K H Solli
Size factor of Norwegian glued laminated beams

Background

A research project where the aim was to determine the size factor k_h, of Norwegian glued laminated beams has been carried out by The Norwegian Institute of Wood Technology in co-operation with two Norwegian producers of glued laminated timber. The purpose of this project was to compare the laboratory results with the corresponding values of k_h given in Eurocode 5:

$$k_h = \left(600/h\right)^{0,2}$$

with the following limits: $1 < k_h \leq 1{,}15$

The limits correspond to h = 600 mm ($k_h = 1$) and h = 300 mm ($k_h = 1{,}15$).

Conclusion

Based on Eurocode 5 it should be expected that

$k_h = f_{m,k,300} / f_{m,k,600} = 1{,}15$

The result from this project shows a lower value of k_h . By using the values from the Weibull-3 parameter distribution the following value of k_h was given for a depth of 300 mm:

$$k_h = \frac{f_{m,k,300}}{f_{m,k,300}} = \frac{34{,}1}{32{,}0} = 1{,}07$$

The corresponding size factor calculated from the mean values was $k_h = 1{,}12$.

Based on observations of the lamellae and their distribution of defects it is assumed that much of the size effect can be explained by the test method and the grading accuracy.

If this assumption is correct, then the size factor k_h is not only a function of the depth, but also of the grading accuracy and the strength class.

29-6-1 N Burger, P Glos
Effect of size on tensile strength of timber

Introduction

When designing timber members according to Eurocode 5 the size of the member has to be taken into account by means of size factors. These factors were derived from investigations mainly carried out in North America and Great Britain using wood species and grading rules common in these countries. Since it is most likely that size effects may also depend on wood quality, grading rules and, perhaps, wood species, the purpose of this study was to investigate whether these size factors also apply to wood species and grading rules currently in use in Germany.

Towards this end the influence of specimen size on the tensile strength was investigated using 750 specimens from timber of native spruce and 200 specimens from timber of native Douglas fir, with varying cross-sectional dimensions and lengths.

Discussion of Results

Results from the tension tests conducted in the course of this study reveal systematic effects of specimen dimensions on tensile strength. On the one hand tensile strength decreases with increasing test length, which corresponds to the statistical approach according to the Weibull theory. On the other hand an increase in tensile strength with increasing specimen width was apparent. This observation contradicts the Weibull theory and can be attributed to the influence of decreasing knot ratio concomitant with the increase in specimen width.

31-12-1 B Källsner, O Carling, J Johansson
Depth factor for glued laminated timber-discussion of the Eurocode 5 approach

Abstract
In Eurocode 5 the bending strength of glued laminated timber depends on the depth of the beam. This paper gives comments on some European investigations of glued laminated timber of spruce (Picea Abies) where the influence of depth has been studied. The test results indicate that the depth factor k_h based on 5-percentiles should be lower than given in Eurocode 5. Further, there is a tendency of lower coefficient of variation in the bending strength of deep beams than of shallow beams. This has an effect on the partial safety factor which could be handled by reducing k_h. Based on this it could be argued that the depth factor k_h should be removed from Eurocode 5.

Conclusion
The Austrian and the Norwegian investigations indicate that the depth factor kh based on 5-percentiles should be about 1,06. In both investigations machine graded timber was used.

A Swedish investigation with well-matched laminations on the tension side of the beams indicates that k_h may be even lower when there is a low variation in the strength of the laminations. This means on the other hand that k_h may be somewhat higher for visually graded timber where there is a somewhat higher variation in the strength of the timber.

There is a tendency towards a lower coefficient of variation in the bending strength for deep beams than for shallow ones. This has an effect on the partial safety factor which could be handled by reducing k_h by about 3 %.

Based on the discussion above it is proposed that the depth factor k_h should be removed from EC5.

For other wood species than spruce (Picea Abies) where other sawing patterns are used and where large single traversing knots appear in the laminations, it cannot be excluded that the depth factor k_h is different.

2.13 TENSION PERPENDICULAR TO GRAIN

25-6-4 H Petersson
On design criteria for tension perpendicular to grain

Introduction

For notched beams, curved beams and beams with openings, cracking and tension perpendicular to the grain is an important matter in design. A standard procedure is to calculate the tensile stresses caused by the loads by using linear elastic assumptions. Normally some formula is used in a hand calculation for the determination of the load effects or, alternatively, a finite element analysis is performed. The stress obtained is compared with some allowable stress value, i.e. some form of stress criterion is used. We may write that

$$\text{calculated stress value} < f_{material} \tag{1}$$

This is a practical approach as long as the material parameter $f_{material}$ can be considered as independent of the size of the structure. However, in a large number of experiments on different types of wooden structures the results clearly indicate a strong influence of size effects.

From the proposal for Eurocode No. 5 [1] the basic rule for ultimate limit state and tension perpendicular to the grain is of special interest for this paper. For solid timber

$$\sigma_{t,90,d} \le f_{t,90,d} \tag{2}$$

and for glued laminated timber

$$\sigma_{90,d} \le f_{t,90,d}\left[\frac{V_0}{V}\right]^{0,2} \tag{3}$$

where V in m^3 is assumed to be an equivalent stressed volume and V_0 the reference volume of 0.01 m^3.

If the size of V can be defined by the length l, the depth h and the width b, then the size effect is

$$\left[\frac{V_0}{V}\right]^{0,2} = \left[\frac{l_0}{l}\right]^{0,2}\left[\frac{h_0}{h}\right]^{0,2}\left[\frac{b_0}{b}\right]^{0,2} \tag{4}$$

and relates to a Weibull model. This model results in failure when the "weakest link" is broken, and does not distinguish between cases where the maximum load is reached, and cases where the load-bearing capacity is substantially larger than (3) indicates due to stress redistributions. Further, (3) does not work well in case of stress concentrations.

It may be questioned whether the stress criterion discussed above should not be replaced by a fracture mechanics criterion in combination with a proper selection of a crack surface. The benefit of such an approach would be a better prediction of the load-bearing capacity of the structure. Another advantage of a fracture mechanics criterion combined with a fracture analysis is that it may offer a tool to predict what happens when the predicted load is obtained. The question is whether there will be a sudden collapse with a running crack, or if the loading can be increased further due to stable crack propagation. A design method that can give an answer to this would be valuable for the design engineer if it is not too complex to use.

The consequences of changing from the present proposal according to formula (2) or (3) to a fracture mechanics approach could for a typical example of crack opening perpendicular to the grain, say for a curved beam, result in

$$\sigma_{t,90,d} \le f_{material} k_l k_h k_b$$

$$k_l = \begin{cases} 1 & \text{stable crack growth} \\ 0,5 & \text{risk of running crack with sudden loss of load-bearing capacity} \end{cases}$$

(5)

$$k_h = \sqrt{\frac{h_0}{h}} \tag{6a}$$

$$k_l = \begin{cases} 1 & \text{no risk of side cracks} \\ \sqrt{\dfrac{b_c}{b}} & \text{risk of side cracks (remaining uncracked width is } b\text{)} \end{cases} \tag{6b}$$

Some theoretical and experimental support for the validity of the proposed design rules will be presented

Concluding remarks

Cracks that run in the fibre direction and open perpendicular to the grains are of major concern in the design of wooden structures. The use of fracture mechanics seems quite natural. In this paper a number of applications have been treated to provide a background for the discussion of size effects. As a summary we may, for the sake of simplicity, conclude that the stress σ calculated by elementary theory should be compared with the fracture material parameter $\sqrt{G_c E}$.

28-6-6 T Canisius
End conditions for tension testing of solid timber perpendicular to grain

Abstract
It is specified in the draft CEN Standard prEN1193 that for tension tests perpendicular to grain of solid timber, the platens be pinned with the pin axis parallel to the grain direction. This is in contrast to the requirement to have 'clamped' ends for the testing of glulam specimens. In this paper, the reason for this difference in requirements is explained with a finite element stress analysis.

Conclusions
- For tension testing perpendicular to grain in solid timber, it is preferable to have pinned end conditions than 'clamped' conditions. Such a choice results in a reduction in stress-concentration related failures at or near the specimen ends.
- In the case of glulam specimens, no significant difference can be expected between results from pinned and clamped end conditions. This is because of their effective vertically symmetric overall nature.

31-6-2 H J Blass, M Schmid
Tensile strength perpendicular to grain according to EN 1193

Introduction
The tensile strength according to EN 384 is derived on the basis of a relationship between tensile strength perpendicular to grain and density:

$$f_{t,90,k} = 0{,}001\ \rho \tag{1}$$

Equation (1) leads to higher characteristic tensile strength values for higher strength classes according to EN 338. In the German timber design code DIN 1052 the tensile strength perpendicular to grain is independent of the strength class. The same applies to the German National Application Document for EC5: a constant value of $f_{t,90,k}$ = 0,2 N/mm^2 for all classes is assumed (for the lowest class S7 $f_{t,90,k}$ = 0). The National Design Specification (1991) of the USA specifies in 3.8.2 " ... designs that induce tension stresses perpendicular to grain shall be avoided whenever possible. When tension stress perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist all such stresses shall be considered …".

Ehlbeck and Mirth developed a test method later introduced in EN 1193. The corresponding test arrangement is shown in figure 1 and figure 2.

The tested volume has a cross-section of 45 mm x 70 mm and a depth of 180 mm. Intermediate timbers stressed parallel to grain are bonded to the specimens. Two inductive measuring gauges are attached diagonally.

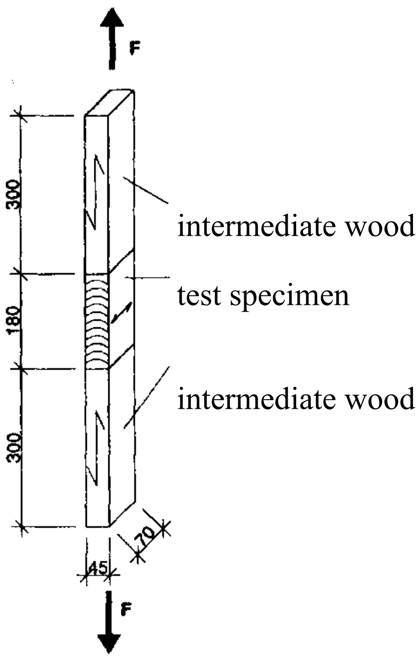


Figure 1: Test specimen for tensile strength perpendicular to grain

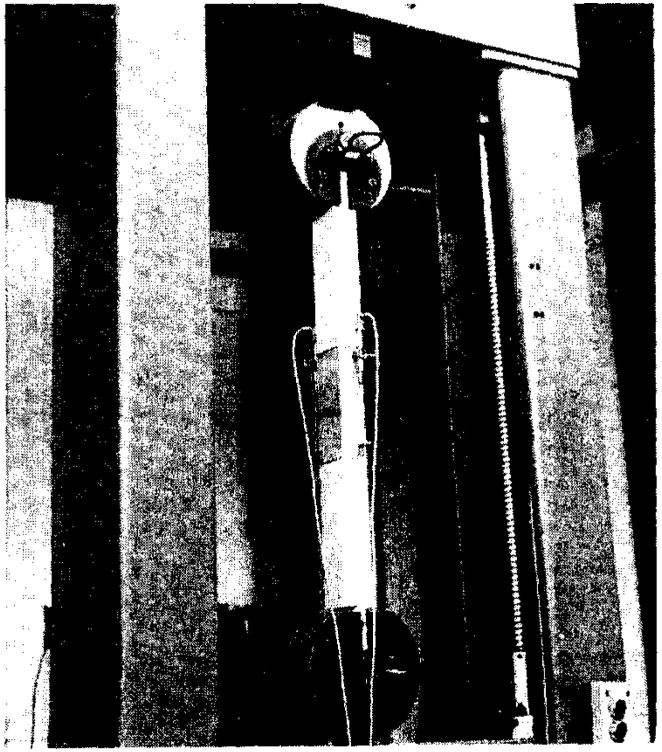


Figure 2: Test arrangement

Summary

The tensile strength perpendicular to grain of 187 wood specimens was determined using the test arrangement of EN 1193 (see figures 1 and 2). 53 % of these specimens failed right at the bond line at the intermediate wood. By Finite Element calculations it was shown that stress intensities are evoked at the bond line and the stress distribution in the specimen is non-uniform.

An important aim of the project was to determine the correlation of tensile strength perpendicular to the grain with other properties, especially with the density. By separating the largest group of the tangential specimens by the location of rupture, the result was a comparatively high coefficient of correlation for specimens which failed at the joint and a very small coefficient of correlation for those with failure in the wood. One possible hypothesis for this result is based on Weibull's probabilistic fracture theory.

It has to be emphasised that the tests were done with nearly clear wood specimens without visible cracks or splits, which often occur in structural size timber. Especially because of cracks, the tensile strength perpendicular to grain in structural timber will be lower than the values given in table 5.

32-6-4 H J Blass, M Schmid
Tensile strength perpendicular to grain of glued laminated timber

Introduction

The tensile strength perpendicular to grain of glued laminated timber according to the European standard EN 1194 is based on the following equation:

$$f_{t,90,g,k} = 0{,}2 + 0{,}015\, f_{t,0,l,,k} \qquad (1)$$

where $f_{t,90,g,k}$ is the glulam strength perpendicular to grain and $f_{t,0,l,k}$ is the tensile strength parallel to grain of the planks. According to Eurcode 5 this characteristic strength $f_{t,90,g,k}$ is related to a reference Volume of V_0 = 0,01 m^3. A test method for determining the tensile strength perpendicular to grain of glulam using this reference volume V_0 is given in EN 1193.

Test series according to this test procedure are reported by Aicher und Dill-Langer. The material they used was especially produced for their research project. The planed planks of strength class C40 had a constant thickness of 33 mm. Planks including pith were excluded (so called 3x-log sawing, centre boards with pith were sorted out).

The aim of the project presented here was to determine the tensile strength of glued laminated timber of different strength classes commonly used in Germany. The material of the test specimens should forma representative sample taken out of numerous beams manufactured by different glulam producers.

Conclusions

The tensile strength perpendicular to the grain shows no significant correlation with wood density and does not differ for different glulam strength classes. This contradicts EN 1194. The lowest values determined in tests are close to zero. Structural details with tensile stresses perpendicular to the grain should be avoided or the timber should be reinforced perpendicular to the grain.

More important than the volume effect according to Weibull seems to be the presence of macroscopic defects as for example ring shake. Macroscopic defects lead to failures governed by stable crack growth.

Based on the test results the volume effect seems to be stronger than assumed by EC5. The exponent k was determined as $k = 3$ compared to $k = 5$ given in Eurocode 5.