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Van De Kuilen, Jan Willem G

Publication date
2016

Document Version
Final published version

Published in
WCTE 2016 - World Conference on Timber Engineering

Citation (APA)
Van De Kuilen, J. W. G. (2016). Wood properties from roundwood to timber engineering. In J. Eberhardsteiner, W. Winter, A. Fadai, & M. Pöll (Eds.), *WCTE 2016 - World Conference on Timber Engineering* (pp. 60-69). Vienna University of Technoclogy.

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WCTE 2016
World Conference on
Timber Engineering
August 22-25, 2016 | Vienna, Austria

WOOD PROPERTIES FROM ROUNDWOOD TO TIMBER ENGINEERING

Jan-Willem van de Kuilen[1]

ABSTRACT: Measuring and assessing wood properties during the production chain is getting more and more important for an optimal use of the resource. Over the years, research has been performed with the focus on establishing important wood properties, with the final goal of an optimized use in timber engineering. It is recognized that not all research results are easily translated into applications or code provisions. Timber grading, the conversion of grading results into strength class assignments, mechanical properties perpendicular to the grain of soft- and medium dense hardwoods are presented. The influence of density and fastener steel grade on the load carrying capacity of joints is discussed. Time-to-failure behaviour of joints is presented and it is shown that slightly more penalizing duration of load factors are required for joints behaving more brittle. Numerical modelling of joints is shown applying a modified Hill-criterion and a continuum damage mechanics model.

KEYWORDS: wood technology, density, joints, safety factors, grading, hardwoods.

1 INTRODUCTION

Ekki

During the transformation phase, a number of technological parameters need to be addressed before it can be applied as part of an engineered wood structure. As structures are designed according to principles and rules in design codes, the timber industry needs to provide structural engineers with sufficient and reliable data of their products. Much of this data is retrieved during the production processes in sawmills and manufacturing plants that produce products like sawn timber, glulam (GLT) or cross laminated timber (CLT). Consequently, both grading and gluing are two essential steps during the production. However, grading alone is not sufficient, as grading often only serves to assign boards and beams to strength classes, such as those listed in EN 338 [49]. This standard now not only contains classes based on bending strength, but contains additional tables for wood that is primarily loaded in tension. This option allows for further optimization of material use. Most important parameters for classification are strength (bending or tension), modulus of elasticity and density. The latter is necessary as input for the determination of the embedding strength in order to be able to determine the load carrying capacity of timber joints.

As the central European timber industry is exploring the use of other species than spruce and pine, other strength values are also becoming more and more interest. As species like beech, ash, chestnut and oak are becoming more and more available, niche markets are being created where these medium dense hardwood can be used. Beech, as one of the most widely available species with a good as they have potential in areas where spruce is at the limit of its possibilities. Examples are curved members where stresses perpendicular to the grain may develop or in trusses where the higher strength of these hardwoods may be used to create smaller members and connections. At the same time, the processing of these hardwoods require new knowledge with respect to the transformation processes from tree to engineered wood product, especially with respect to gluing. On the other end of the scale, there are tropical hardwoods that combine high durability with high strength, which can provide alternative solutions in certain exposed structures.

2 STRENGTH GRADING

2.1 GRADING

Strength grading is the process of assessing wood parameters with the goal of estimating strength, modulus of elasticity and density. This can be done either visually or by machine, but combinations are also possible. Even though machines perform better, some kind of visual grading is always part of the process, as EN 14081-2/3

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[1] Jan-Willem van de Kuilen, Technical University of Munich, Germany & Delft University of Technology, the Netherlands. vandekuilen@hfm.tum.de

[56] requires visual override rules in case not every strength determining wood feature can be captured by the machine. Efforts to grade roundwood before sawn timber production show promising results when tomographic measurements are performed, even though correlation between features and strength is not as high as for stress wave grading of boards, as the strength predicting feature is primarily based on the relation between knots and strength. This is because the correlation between knots and strength is lower than between modulus of elasticity and strength. Stress wave grading on standing trees so far has not produced convincing yield improvements. Stress wave grading of long and short logs, however, is a possibility to select low quality logs and avoid them from entering the sawmill in the first place. Rais et al. [2] performed an extensive analysis on the Douglas fir production chain and showed that yield in C24 and C30 classes could go up considerably by selecting logs at an early stage.

2.2 VISUAL GRADING

Visual grading is still the main grading method used in large parts of Europe. The grading procedure involves the assessment of growth features of boards on the basis of which a board can be assigned to a certain grade. Unfortunately, there is no European standard for visual grading of softwoods yet, and regionally accepted visual grading standards are the norm. In order to overcome trade barriers, EN 1912 [57] contains a list where national grades can be found in relation to the European strength class table of EN 338. For heavy tropical hardwoods, a European visual strength grading standard is close to being finalized, based on an analysis of existing standards for hardwoods in the UK, France and the Netherlands [61]. Where UK and France have issued pure strength grading standards [47,60], the Netherlands has a mixed standard where both strength grading and grading for specific applications in hydraulic structures are integrated [59]. As these hardwoods generally have little or no knots, the main strength grading feature is slope of grain. This is difficult to quantify and assess and consequently dense hardwoods can only be assigned to a single grade. The main requirement for slope of grain is a ratio of 1:10 and knots may not take up more than 20% of the width on the face where they are visible. This is to avoid serious defects (not restricted to knots) to take up too much of the cross section. Cross grain as shown in Figure 1 may not be more than 1:4. Another defect that can sometimes be present in wood are compression failures or brittleheart. Detection of compression failures (CF) or brittleheart is difficult, but with CT-Scanning, the failures can actually be detected, as shown in Figure 2 [18].

Other grade determining features such as slope of grain and top fall can also be difficult to detect. The latter can be a serious problem in small cross section elements, for instance for the production of I-beams. The tensile strength is seriously reduced. An example of failure is shown in Figure 3. Visual graders need to be specifically focussed on this feature, as boards with a fibre angle of about 60° lead to a serious reduction in tensile strength, which was 7.54 N/mm^2 at a density of 421 kg/m^3 in the case of Figure 3.

Figure 1: *Cross grain in an azobé board loaded in flatwise bending.*

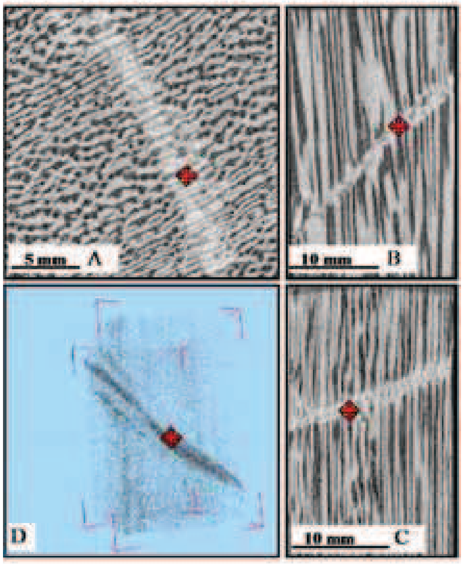

Figure 2: *Brittleheart failure, Sectional images (A, B, and C) and 3D image (D) of a sapupira sample – CF grade 4. Red cross indicates the position in the cube. 3D illustration shows higher densities where 'white' bands are present [18].*

Figure 3: *Top fall causing a serious reduction in tensile strength parallel to the grain.*

2.3 MACHINE GRADING

Assessing the mechanical properties of wood by a machine has clear advantages in terms of the quality of the prediction. Most of the machines currently in use address the dynamic modulus of elasticity by means of a longitudinal vibration measurement. Both wet and dry grading is possible. The correlation between modulus of elasticity and strength is high, but adding parameters such as knot recognition and fibre deviation measurements does bring added value in terms of higher strength class assignments or higher yields. Knot recognition improves the quality of the strength prediction and adds value in the sense that either a higher class assignment can be achieved, or the weakest spot can be recognized and the material can be upgraded by finger jointing. Machines that measure knots and correlate knots with strength have been improved considerably by taking into account the location of the knot in a board when estimating strength. Tension strength can be better predicted than bending strength, but the principle of taking into account the location of the knot with reference to the neutral axis is the same [13,14]. In fact, when grading long boards, the weak spot recognition may lead to downgrading of boards because of the length effect, boards that otherwise would have been assigned to a higher strength class on the basis of the MoE measurements. The effects of this downgrading are analysed in [4] and [5] for visually graded material and in [6] for machine graded material. A clear length effect is present in the material [7-9], but length is so far not specified as a minimum requirement for selecting test samples, nor are machine approvals restricted to the maximum length tested during the approval procedure.

2.4 STATISTICAL VARIATIONS IN GRADED TIMBER

During the grading process, errors can be made. Errors can either lie in the quality of measurements by visual graders, but also by machines when estimating the strength wrongly. However, apart from these errors, uncertainties are present and the 5-th percentile strength level depends very much on the machine settings that has been used to grade a batch. Machine settings can be derived for a certain strength class to be graded in a single pass, or they can be derived for a combination of classes. In Figure 4 and 5, the variation in 5-th percentile values is shown for a batch that has been graded by a machine in two different ways. Firstly, all boards have been graded for strength class assignment C24. Secondly, the batch has been graded for strength class combination C35/C24. After grading, it can be observed that strength class combination C35/C24 results in higher characteristic values for the C24 boards. Apart from that, the steepness of the cumulative frequency distribution indicates that the scatter in material properties (the coefficient of variation) is smaller for the samples that have been graded in the combination, thus allowing for smaller safety factors. The difference shows itself in a much longer tale in the frequency distribution when graded for C24 and better. This results in different safety levels for structures made with C24 timber and consequently different safety factors should be applied for the different grading options. As this is not considered feasible, a more restrictive approach to grading policies should be incorporated in the relevant standards [1], [3]. For the two grading options of the same batch as presented here, a safety factor γ_M of 1.45 could be applied for material graded in the combination C35/C25, whereas a safety factor of $\gamma_M > 2$ would be required for the C24 class graded on its own. The estimation of the γ_M factors is performed using the method specified in ISO 2394 [48].

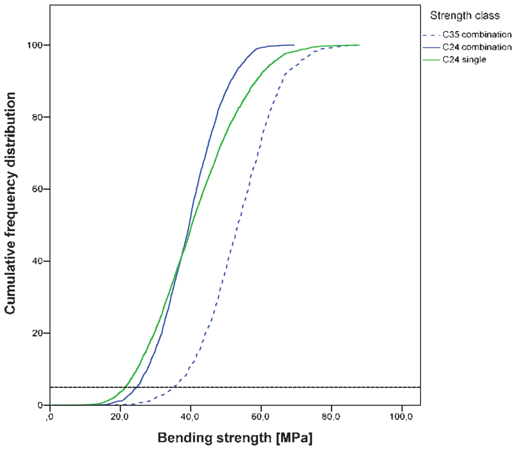

Figure 4: *CFD´s for a batch of timber, graded to C24 and C35/C24 respectively.*

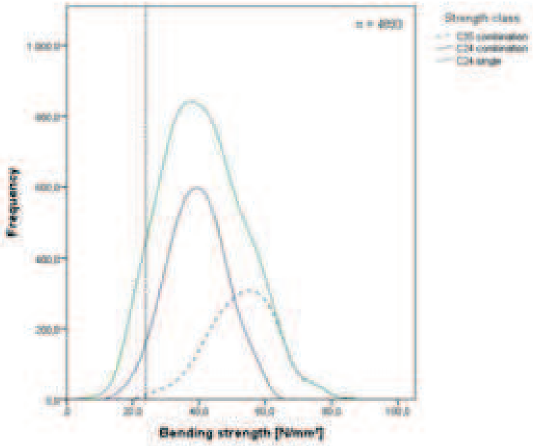

Figure 5: *Frequency distributions a batch of timber, graded to C24 and C35/C24 respectively.*

3 MEDIUM DENSE HARDWOODS

The strength profiles as specified in EN 338 are primarily based on known relationships between the different strength and stiffness values for softwoods [51]. In order to verify these relationships for other species that come onto the market, additional testing is required in order to obtain a better insight, as well to understand better the market potential. As economical production of beech and ash lamellas is difficult to achieve [11], a successful introduction on the market of these species will rely on niche applications, for instance curved glulam members that need local reinforcements for tensile perpendicular to the grain. Therefore, in a research program additional testing has been performed [12] on beech and ash grown in southern Germany and in accordance with EN 408 [52]. Some of these tests however require considerable specimen preparation before testing can commence, especially in the case of shear and tension perpendicular to the grain testing. In Figure 6, the cumulative strength and modulus of elasticity distributions are shown of a test series on ash and beech, with spruce as a comparison.

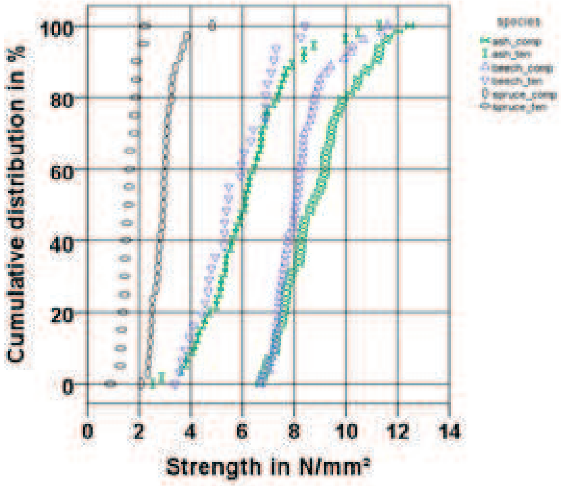

Figure 6: *Cumulative frequency distributions of the strength of ash and beech loaded perpendicular to the grain.*

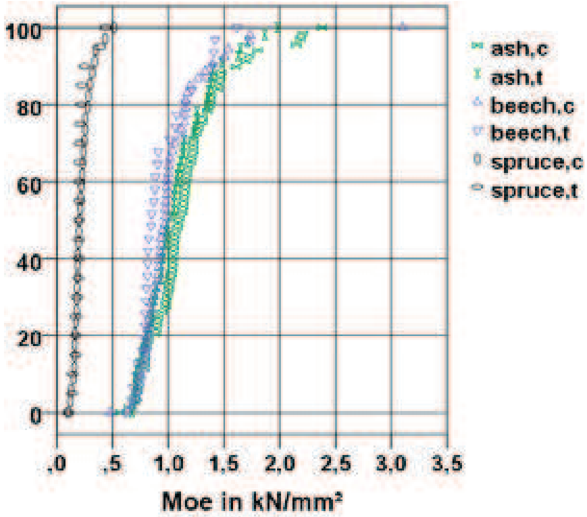

Figure 7: *Cumulative frequency distributions of the modulus of elasticity of ash and beech loaded perpendicular to the grain*

From the strength distributions it can be concluded that on the 5% level, there is not as great a difference between tension and compression for ash and beech as compared to spruce. With about a factor 3.5 higher tensile strength perpendicular to the grain, it is clear that ash and beech have potential in dedicated structural applications. The difference between ash and beech is hardly noticeable. With regard to the stiffness perpendicular to the grain, ash and beech also show very little difference. With regard to the density, Figures 8 shows that correlation between density and strength is there, when looked generically over the density range. On individual species level there is a slight increase in compression strength for spruce with increasing density, otherwise there is no correlation between density and strength, neither in tension nor in compression. From the test results, it can be concluded that the published values in EN 338:2016 [50] for the tensile strength perpendicular to the grain are too conservative, and consequently hinder the introduction of curved glued laminated timber on the market.

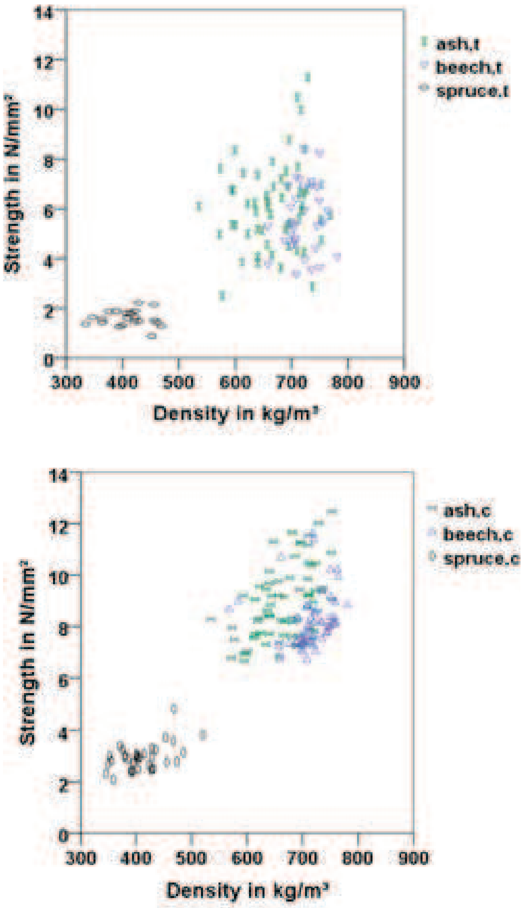

Figure 8. *Relationship between density and tensile strength (upper) and density and compression strength (lower) perpendicular to the grain.*

4 TIMBER JOINTS

4.1 RELATIONSHIP BETWEEN DENSITY AND LOAD CARRYING CAPACITY OF JOINTS

In most design codes, the load carrying capacity of connections is related to the density of the material. In order to assess the correlation between the two, standard procedure is to measure the density of the wood parts of the connection, and correlate the mean value of the density to the load carrying capacity of the connection.

For the design of joints using the Johansen equations, the embedding strength of the material under the fastener is an essential feature. The value is directly related to the density of the timber and consequently the characteristic density of the timber is one of the three essential parameters for strength class assignment. With respect to the embedding strength, in Figure 9 the characteristic density values for the class boundaries of EN 338 are shown as vertical dotted lines for a dataset of 804 embedding tests [19].

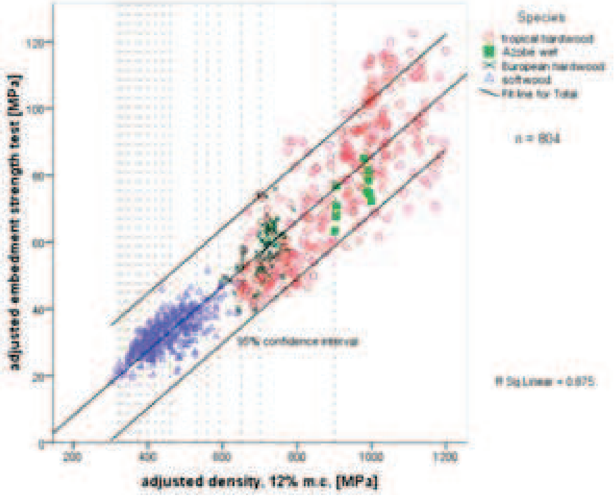

Figure 9: *Embedding strength and strength class boundaries of EN 338.*

In Figure 10, the correlation between density and load carrying capacity for dowelled joints of spruce is shown [20]. For the most widely used species spruce, an additional graph is shown for connections with spruce, where a slight positive influence seems to be present. This positive relationship is rather fictitious however. The density is measured for each member of the joint, and the load carrying capacity is correlated to the mean density. A clear positive relationship is found. In practice however, boards are not measured for density (in visual grading) or the mean density of a board is measured and used as a grading property and (machine grading) only the overall mean board density is measured. It means that the scatter in wood densities near the fasteners increases. For another research project [21, 27-28], not the density of the wood of the connection was measured, but 5 meter boards were matched for density in order to obtain parts for middle and side members with approximately equal density. In order to verify the influence, the load carrying capacity of a number of different joint types was measured and correlated to the mean board density of the two boards used for the middle and side members respectively. From Figure 11 it can be seen that no correlation is present between density and load carrying capacity for the three joint types. The tested joint types were nailed joints, with a high yielding failure and toothed-plate and split-ring joints that fail in a more brittle mode. Sandhaas [22] confirmed that a dependency on density is also the absent for connections with species like beech and azobé (*lophira alata*) covering a density range from 300 to 1100 kg/m^3, see Figure 12.

4.2 HIGH STRENGTH STEEL FASTENERS

Until recently, steel dowel grades were regulated in the sense that dowels were generally ordered in accordance with European standard EN 10025, allowing a maximum grade of S355. As Eurocode 5 has no restrictions, higher steel grades are also possible. In order to see whether the load carrying capacity of timber joints with dowel type fasteners can be increased just by changing the steel grade, a number of tests have been performed with varying dowel diameter and on species like spruce, beech, azobé and beech-LVL.

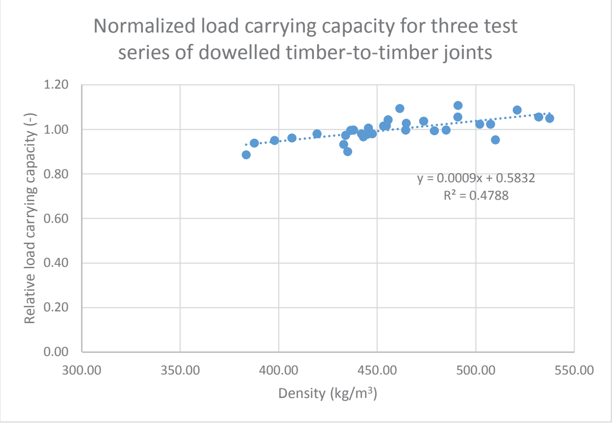

Figure 10: *Correlation between density and normalized load carrying capacity of timber joints, modified from [20].*

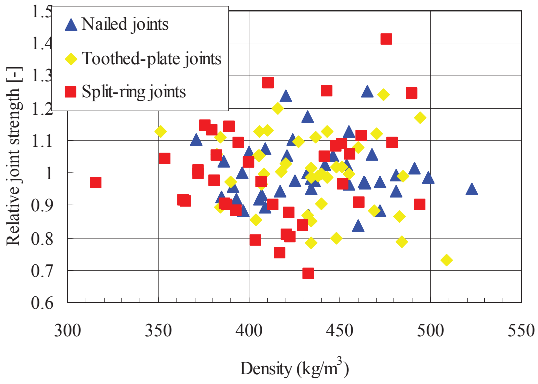

Figure 11: *Influence of spruce density on the normalized load carrying capacity of three different types of joints [21].*

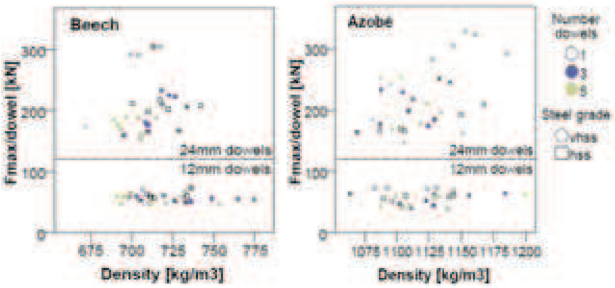

Figure 12: *Influence of wood density on the load carrying capacity of 12 and 24 mm dowelled joints of beech and azobé [22].*

A considerable increase in load carrying capacity can be observed in all cases, both wood-wood joints as well as joints with slotted –in steel plates. With increasing fastener strength, the load carrying capacity increases considerably, and the applicability of the European Yield Model could be confirmed. With respect to beech-LVL there was an interesting finding in the fact that in single dowel tests with ordinary steel, the steel load carrying capacity in shear was reached, before failure of the wood parts. This happened in a number of tests, showing that the high load-carring capacity of the beech-LVL might induce failure modes that are generally not checked by structural engineers as they are not standard practise to verify. In the test series, shear failure of the steel was observed only in single dowel tests. With regard to the steel grade, it was confirmed that ordinary steel had considerable higher than expected strength values [23].

4.3 TIME-TO-FAILURE ANALYSIS OF JOINTS

Eurocode 5 indicates that for the long term strength of timber joints, the same modification factors for duration of load can be applied as for wood and glued laminated timber. At the same time, creep factors for joints are a factor two higher than for timber. In a large research project initiated in 1962 by Vermeyden, the influence of load level on the time-to-failure of joints has been under investigation [27-29]. Only recently, in 2014, the project terminated when the last nailed joints were removed from the test-rig and short term tested for residual strength.

***Figure 13**: Beech LVL joints: slotted in steel plate with single dowel. Upper: HSS grade 12.9, Lower: ordinary steel grade S235. (Steel plate not shown).*

The results of the time-to-failure tests presented in Figure 14 cover load levels between 50% and 90% of the mean strength, normalized to make the different joint types comparable. The test results show a considerable difference between nailed joints on the one hand and toothed-plate and split-ring joints on the other. The nailed joints show a favourable time-to-failure behaviour and the reason for this is found in the fact that the large number of nails are able to redistribute loads from one to another, whereas for toothed-plate and split-ring joints this is hardly possible. Equation (1) relates the constant load with the time to failure in a test:

$$\frac{\sigma}{f_s} = C_1 - C_2 \log t_f \tag{1}$$

where σ = stress, f_s = mean short term strength and t_f = the time-to-failure. A linear relationship is generally found and also the basis for the k_{mod} factors in Eurocode 5. For wood, C_1 generally takes a value of around 0.9 to 1, whereas C_2 has a value of around 0.063 to 0.065 [30-31]. Hoffmeyer and Fridley [32-37] found comparable values, but for non-constant climatic conditions the time-to-failure lines needed to be shifted to the left. Sometimes a delay time could be observed, indicating that a number of deformation processes is acting with different time-scales, similar to creep.

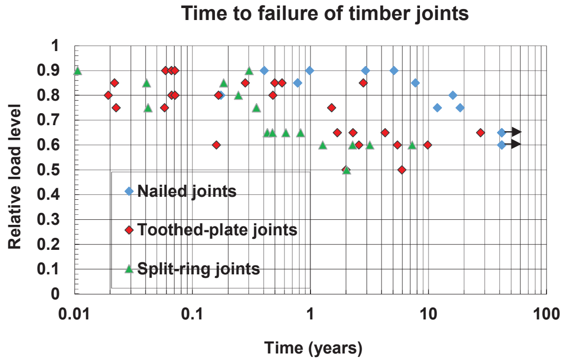

***Figure 14**: Results of time-to-failure tests on nailed, toothed-plate and split-ring joints.*

In [29] a thorough analysis of the results of Figure 14 was performed, both on joint type level as well as on the comparison. As considerable scatter can be observed in the time-to-failure results, but the performance of nailed joints is clearly advantageous over the more brittle joints. The scatter for nailed joints is much smaller, and as load redistribution is possible (for instance when crack formation under the nails initiates), not only a more favourable safety factor is justified, also more favourable k_{mod} factors. In Table 1, the scatter for the short term strength is analysed in relation to the required safety factor in accordance with ISO 2394 [48]. In Eurocode 5, the safety factor for connections is 1.3, so for toothed-plate joints and split-ring joints this seems to be reasonable, whereas for nailed joints this seems to be punishing.

***Table 1:** Statistical data of joints and safety factors*

Joint type	Average strength (kN)	St. dev. (kN)	COV (%)	Safety factor (ISO 2394) γ_M
Nailed	47.1	4.4	9.3	1.17
Toothed-plate	32.7	4.1	12.5	1.28
Split-ring	28.2	4.5	16	1.44

On the basis of the time-to-failure results of the three different joint types, it can be concluded that k_{mod} factors for nailed joints are on the conservative side, but for toothed-plate joints and split-ring joints that behave rather brittle, it seems they are on the unconservative side. From Figure 14 it can be seen that on a 50% load level, failures do occur within the first 10 years of loading. In order to account for the reduced long term strength, factor C_2 in equation (1) should be around 0.07 or 10% higher than that for wood [29]. Depending on the type of regression and the delay time chosen, the regression equation can be written as:

$$\frac{\sigma}{f_s} = 0.95 - 0.07\log t_f \qquad (2)$$

with t_f in hours, and proposed k_{mod} values are given in Table 2.

***Table 2:** Duration of load factors for timber joints*

Load duration class	Accumulated load time	t_f in eq. (2)	k_{mod}
Permanent	> 10 yrs.	50 yrs.	0.55
Long term	6 mth. – 10 yrs.	10 yrs.	0.60
Medium term	1 week – 6 mth.	6 mth.	0.70
Short term	less than 1 week	1 week	0.80

More advanced models to describe the time to failure are available [38-41] and basically improve the models in the sense that the different time dependent processes inside the material are incorporated in a single model. Applying a non-linear time-to-failure line to the data of the toothed-plate joints results in Figure 15. A threshold level for t = ∞ below which no damage accumulates is estimated at around 0.35 on the basis of the available data.

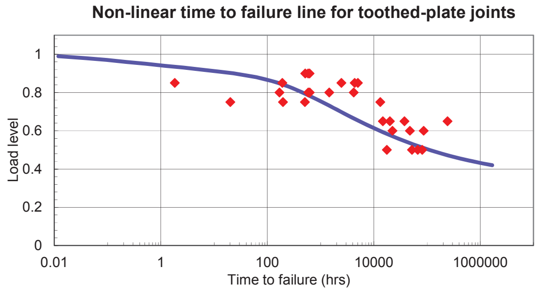

***Figure 15.** Non-linear time to failure line for toothed-plate joints.*

Using the chemical kinetics approach, the rate of bond breaking can be expressed as damage and a damage accumulation can be calculated. This is shown in Figure 16 for the same data of Figure 15. Depending on the approach taken (linear or non-linear time-to-failure processes) a different damage accumulation curve is found. The differences are partly based on the assumed process, and partly on the statistical regression differences.

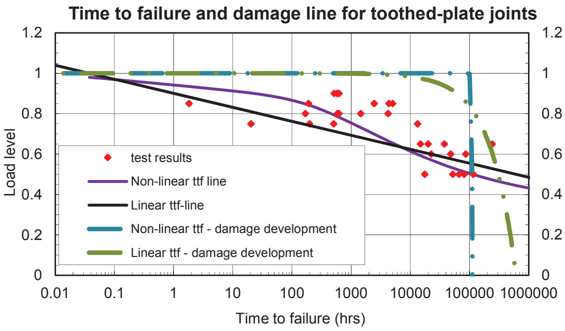

***Figure 16.** Time-to-failure and damage accumulation lines for toothed-plate joints (damage development calculated at 50% load level).*

The damage accumulation lines, in whatever form, can be a good tool to analyse the long-term behaviour of structures under varying loads and when structural degradation takes place. It is clear however from the damage curves that the actual failure process near to collapse, takes place in a very short time span. In [42] this is referred to as so-called ´killer loads´. The damage concept is also applicable on fatigue problems and similar curves can be found based on the amount of load cycles instead of on the time. The advantage is that different threshold ratios can be defined for different stress states. In wood, the fatigue life in compression is clearly favourable compared to tension, and thus a higher threshold can be defined for t=∞. Also, a ratio R=-1 is more damaging than a ratio R=0.1, indicating a lower threshold value. A qualitative graph of what is possible with non-linear fatigue curves based on a bond breaking / damage accumulation approach is shown in Figure 17. The arrows indicate how the parameters of the model may influence the location of the curve and its shape. This depends among others on the delay time, the threshold level and the amount of damage per cycle.

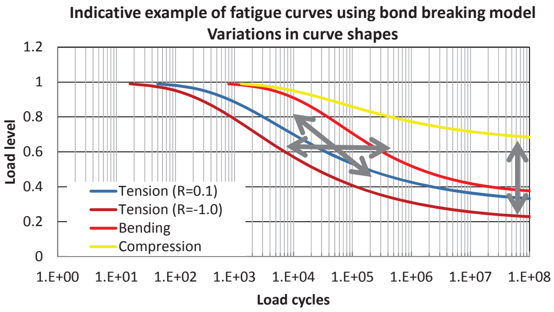

***Figure 17.** Non-linear fatigue curves for wood. Arrows indicate how model parameters may influence the location and shape of the curves.*

The problems with fatigue modelling of wood and joints are however numerous. As wood is a non-linear visco-elastic material and highly anisotropic, it is not only the amount of cycles and the stress range that govern the fatigue life. Apart from the stress range R (= minimum stress / maximum stress), also the stress ratio r (= stress level / short term strength) and the frequency play a role in the fatigue behaviour. This makes it difficult to estimate the values of the parameters that determine the location and the shape of the fatigue curves properly.

5 NUMERICAL MODELLING OF TIMBER JOINTS

Numerical modelling of the mechanical behaviour of wood and joints is complicated. The application of a Tsai-Wu criterion is clearly not sufficient if non-linear effects have to be taken into account. Stress-strain diagrams of the materials are non-linear and this has to be incorporated, especially when failure loads are to be predicted. Different modelling approaches exist, all having their specific benefits and associated problems, from classical continuum mechanics, through fracture mechanics, hybrid approaches and lattice models. Lattice models can be used to model wood as a material and structural wood products, are however not appropriate for the modelling of joints with contact problems. Fracture mechanics is applicable when brittle failure or crack development needs to be modelled, such as notched beams, or joints with load transfer perpendicular to the grain. Sandhaas discussed the issues extensively [22]. Already at low load levels, a non-linear behaviour can be observed, indicating plastic deformations. Therefore, the load-displacement curves of tests are difficult to reproduce when the test protocol is in accordance with EN 26891 [62]. Especially, the load cycle between 10% and 40% of the estimated failure load requires an appropriate FEM model with logical values for the wood properties. Clearly, the stress-state around a fastener is extremely complicated and requires either adaptive meshes, or adaptive material properties. In order to be able to follow the load path, the yield criterion needs to be adapted as below the fastener there is a mixture of stresses in compression (parallel and under an angle to the grain), shear, tensile perpendicular to the grain and friction. Friction also depends on the steel grade and the smoothness of the surface of the dowels [25, 26].

In Figure 18, the load-slip curve from a test and a simulation is shown covering both softwood and hardwood behaviour [44]. The good approximation was obtained by applying the Hill criterion [46] where the yield stress was related to the embedding strength of wood.

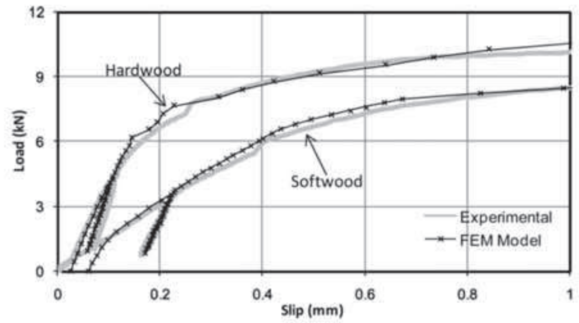

***Figure 18.** FEM-model simulating the load slip curve including the loop in EN 26891 [44].*

Sandhaas developed a numerical model based on continuum damage mechanics and analysed the behaviour of timber joints with 1, 3 and 5 dowels in a row, with spruce, beech and azobé as species and two steel grades and diameters [22]. An example of the damage evolution under a dowel in a joint that is failing plastically is shown in Figure 19, showing the potential of a CDM approach.

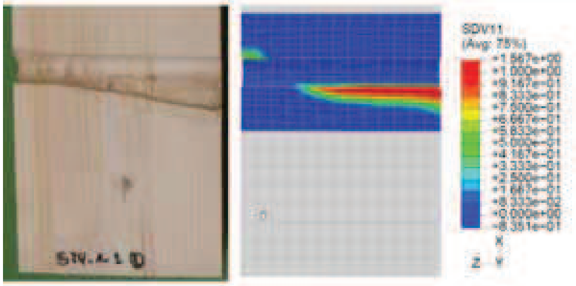

***Figure 19.** Damage model showing compression damage parallel to the grain under a fastener [44].*

The approach is based on damage and state variables that allow for plastic deformations in compression and brittle failure under tension and shear. Multiple stress states are accounted for, so modelling of complex 3D problems is possible. Splitting sensitivity will be partly influenced by the way the rays are located inside the specimen, relative to the fastener direction. These differences will also express themselves when testing entire joints as a more complex stress states are obtained. In Figure 20, the damage parallel and perpendicular to the grain is shown for a beech specimen with three 24 mm dowels in a row. In the parallel state, the damage state can be clearly seen, but is not evenly distributed over the three fasteners. In the perpendicular state, the interaction between the fasteners is clear, as in between the fasteners a clear splitting damage develops. In [19], some clear differences between wood species have been highlighted when embedding tests are performed.

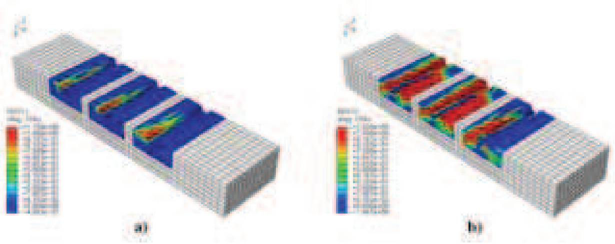

***Figure 20.** Parallel to the grain damage (a) and perpendicular to the grain damage (b), for a joint made of beech with three 24 mm high strength steel dowels [44].*

6 OUTLOOK

Grading, hardwoods, joints incl. long term strength and numerical modelling of joints have been addressed in this contribution. Issues for further research leading to more species independent design codes have been highlighted. Generally, research that leads to a better understanding of wood as a material, will lead to better structural performance and a larger variety of applications. This also implies that a better understanding is necessary on how wood density and hardwoods perform in areas where traditionally softwoods are used. Design standards for timber structures have room for improvement, not the least because of the wide variety in species and products entering the market. For a highly anisotropic and variable material like wood, the number of parameters in the models is high and developed models are generally suited for specific problems. Capturing nature´s complexity in understandable models remains a challenging task, whereas at the same time the models must be comprehensible and applicable for engineers as well. They should be neither too complex, nor too simple.

ACKNOWLEDGEMENT

This paper is the result of the work many both colleagues and students. Without pretending of being complete, I do like to express my sincere gratitude to Marco Ballerini, Alfredo Dias, Wolfgang Gard, Andrea Misconel, Peter Stapel, Andreas Rais, Geert Ravenshorst, Carmen

Sandhaas and Peter de Vries for the fruitful discussions over the years during the research projects partly presented and discussed in this contribution.
The organisation of WCTE 2016 is sincerely acknowledged for the opportunity given to present this paper.
Last but not least, many thanks go to Tom van der Put, Hans Blass and Ario Ceccotti for laying the foundations, for their spirit, for the years of cooperation and the stimulating talks and discussions we´ve had over the years.

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