12th International Conference on Applications of Statistics and Probability in Civil Engineering, ICASP12
Vancouver, Canada, July 12-15, 2015

Aspects of Code Based Design of Timber Structures

Jochen Kohler
Associate Professor, Dept. of Structural Engineering, Norwegian University of Science and Technology (NTNU), Trondheim, Norway

Gerhard Fink
Research Scientist, Empa, Swiss Federal Laboratories for Materail Science and Technology, Dübendorf, Switzerland

ABSTRACT: The European timber design standard is under development and a new version will be issued at the end of this decade. In this paper the present design standard is critically assessed in regard to its ability to identify design solutions with a consistent level of reliability. The main issues to enhance the current standards are identified and discussed.

1. INTRODUCTION

Sustainable development is the important requirement and goal for modern society and the international research community is in demand to find solutions that provide the foundation for this aim. The role of structural engineering research is thereby of significant importance. The development of methodologies and principles that allows for the optimal allocation of resources into the structural performance and their implementation into the daily engineering practice constitute the major challenge for ongoing and future research in the field of structural engineering.

The broad implementation of newly developed principles requires their proper transition into rules and regulations that constitute the basis for the daily work of practicing engineers. Thus, rules and regulations as structural design codes constitute the mayor interface between structural engineering research and practical application and it is of utmost importance that structural design codes are up to date with the best scientific information available and, at the same time, are simple enough for straight forward application.

This challenge outlined above is general for the entire structural engineering research and professional community. Here, timber and timber based materials might be attributed with a special status since timber as a natural grown material plays an important role in the safe, cost efficient and sustainable development of our future build environment because of its beneficial properties. Timber is an efficient building material, not least in regard to its mechanical properties but also because it is a highly sustainable material considering all phases of the life cycle of timber structures: production, use and decommissioning.

Timber is a widely available natural resource; e.g. with proper management, there is a potential for a continuous and sustainable supply of raw timber material in the future. Because of the low energy use and the low level of pollution associated with the manufacturing of timber structures the environmental impact is much smaller than for structures built in other materials.

However, besides the beneficial properties of timber the confident use of timber as a load-bearing material is particularly challenging compared to other common structural materials as steel and concrete. One of the main reasons for this is that timber is a highly complex material; the proper use in structures actually requires a significant amount of expertise in structural detailing.

Another main reason is that any prediction of the structural performance of timber is associated with large uncertainties. Timber is by nature a very inhomogeneous material. The material properties depend on the specific wood species, the geographical location and furthermore on the local growing conditions over the entire lifetime of the tree. Timber is an orthotropic material, i.e. it consists of "high strength" fibres/grains which are predominantly orientated along the longitudinal axis of a timber log/ tree and packed together within a "low strength" matrix. After a log is sawn into pieces of structural timber, irregularities, such as grain direction or knots, become, in addition to the orthotropic characteristics mentioned above, highly decisive for the load-bearing capacity of a timber structural element. Consequently, the properties of solid timber cannot be designed or produced by means of some recipe but may be ensured to fulfil given requirements only by quality control procedures implemented during the production process for sawn timber. Timber material for structural purpose is generally associated to a certain grade or strength class. However, there are various different ways how quality control is implemented in the production process and the properties of timber of a certain strength class are highly sensitive to the quality control scheme applied to the timber.

Timber is a viscoelastic and hygroscopic material. When using timber as a load-bearing element in a structure it is of high interest how the load-bearing performance is developing over time, i.e. how the building environment with its variable loads, temperature and moisture is influencing the timber structural element.

The high importance of structural timber and timber products for the sustainable development of our build infrastructure together with the fact that many features of the structural behaviour of timber are not known with accurate precision underlines the urgent need for extensive and coordinated research in this field. Furthermore it is necessary that current and future knowledge about timber and timber based materials load- bearing behaviour is represented in the current design standards in a sensible way.

In Europe the design of structures is regulated by the Eurocodes, a suite of consistent standards for structural design covering all relevant load scenarios and building materials. They were developed under the supervision of the European Committee of Standardization (CEN) and regulate to a large extent the performance criteria of the build environment being reliability, serviceability and safety of structures. The Eurocodes had been introduced in the 1980s and are by now compulsory for structural engineering design in most European countries. Until 2020 a revision and update of the Eurocodes is planned. Thus, this constitutes an excellent opportunity to critically reflect the design procedures prescribed in the Eurocode 5 – "Design of Timber Structures" in the light of recent scientific developments.

2. BASIC PRINCIPLES OF RELIABILITY BASED CODE CALIBRATION

Modern design codes, such as the Eurocodes (2002), are based on the so-called load and resistance factor design (LRFD) format. Next, the principle of LRFD is explained for the case of two loads; one that is constant and one that is variable over time. The LRFD equation is given in Eq. (1). Here R_k, G_k and Q_k are the characteristic values of the resistance R, the permanent load G, and the time variable load Q. γ_m, γ_G and γ_Q are the corresponding partial safety factors. z is the so-called design variable, which is defined by the chosen dimensions of the structural component.

$$z\frac{R_\mathrm{k}}{\gamma_m} - \gamma_G G_\mathrm{k} - \gamma_Q Q_\mathrm{k} = 0 \tag{1}$$

The characteristic values for both load and resistance are in general defined as fractile values of the corresponding probability distributions. In Eurocode 5 (2004) the following characteristic values are defined: R_k is the 5% fractile value of a Log-normal distributed resistance, G_k is the 50% fractile value (mean value) of the Normal distributed load (constant in time), and Q_k is the 98% fractile value of the Gumbel distributed yearly maxima of the load (variable in time).

The corresponding partial safety factors can be calibrated to provide design solutions (z) with an acceptable failure probability P_f (Eq. 2). Here R, G, and Q are resistance and loads represented as random variables, $z^* = z(\gamma_m, \gamma_G, \gamma_Q)$ is the design solution identified with Eq. (1) as a function of the selected partial safety factors, and X is the model uncertainty.

$$
\begin{aligned}
& P_f = P\{g(X,R,G,Q) < 0\} \\
\text{with} \quad & g(X,R,G,Q) = z^* XR - G - Q = 0
\end{aligned}
\tag{2}
$$

Often the structural reliability is expressed with the so-called *reliability index* β (Eq. 3). A common value for the target reliability index is $\beta \approx 4.2$ which corresponds to a probability of failure $P_f \approx 10^{-5}$ (JCSS, 2001).

$$\beta = -\Phi^{-1}(P_f) \tag{3}$$

In general, different design situations are relevant; i.e. different ratios between G and Q. This can be considered using a modification of Eq. (1)–(2) into Eq. (4)–(5). α_i might take values between 0 and 1, representing different ratios of G and Q. $\hat{R}$, $\hat{G}$, and $\hat{Q}$ are normalized to a mean value of 1. For each α_i one design equations exists, thus altogether n different design equations have to be considered.

$$z_i \frac{\hat{R}_\mathrm{k}}{\gamma_m} - \gamma_G \alpha_i \hat{G}_\mathrm{k} - \gamma_Q (1-\alpha_i)\hat{Q}_\mathrm{k} = 0 \tag{4}$$

$$g_i(X,\hat{R},\hat{G},\hat{Q}) = z_i^* X\hat{R} - \alpha_i \hat{G} - (1-\alpha_i)\hat{Q} = 0 \tag{5}$$

Afterwards, the partial safety factors (γ_m, γ_G, and γ_Q) can be calibrated by solving the optimisation problem give in Eq. (6).

$$\min_{\gamma} \left[\sum_{j=1}^{n} \left(\beta_\mathrm{target} - \beta_j \right)^2 \right] \tag{6}$$

The reliability based code calibration is briefly introduced to illustrate the influence of uncertainties (load and resistance), in respect to codes. Please find more information in (e.g. JCSS, 2001; Faber and Sørensen, 2003).

The application of the above sketched framework constitutes the basis for reliability based calibration of the partial safety factors of a load and resistance factor design format. And it entirely depends on a realistic representation of loads, resistances and model accuracy by the random variables R, Q, G, and X.

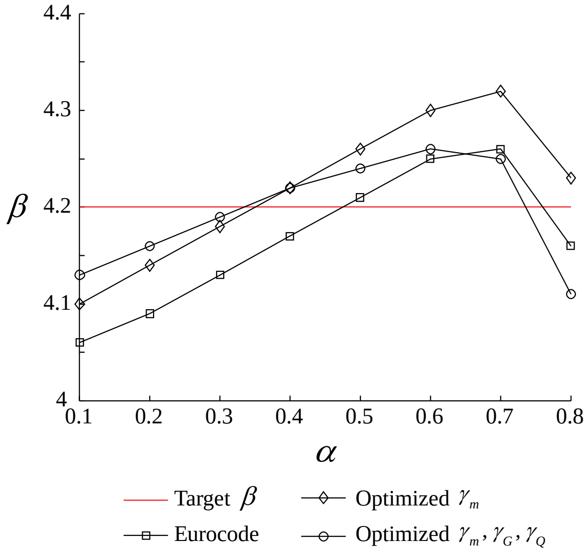

Figure 1: Reliability Index over different design situations alpha for solid timber in bending. The different lines represent different sets of partial safety factors (Kohler and Fink, 2012).

2.1. *Example*

The design equation for a structural component can be calibrated according to the procedure described in above. The chosen variables of Eq. (4) and Eq. (5) are summarized in Table 1. Using this values the situation could represent a solid timber bending beam loaded by constant (e.g. self-weight of beam and installations) and variable (e.g. live load).

In the presented example, which is explained in more detail in Kohler and Fink (2012), the range $\alpha = [0.1, 0.2, ..., 0.8]$ is chosen, to exclude rather unrealistic design situations. The calculations was performed with the software CodeCal (JCSS, 2001). In Figure 1 the chosen target reliability index of $\beta = 4.2$ (red line) is compared with the design solutions for the structural component obtained according to the current version of the Eurocode ($\gamma_m = 1.3, \gamma_G = 1.5, \gamma_Q = 1.5$); represented by the line with squares. The reliability indices of the design solutions according to the Eurocode tend to be too low compared to the target reliability index, especially for small α. The line with the diamonds is obtained when all partial safety factors are subject to optimization: $\gamma_m = 1.29, \gamma_G = 1.30, \gamma_Q = 1.57$. However, it is the philosophy of the Eurocodes that the partial safety factors for the

Table 1: Chosen representation of the model uncertainty X, the bending strength R, the permanent load G and the variable load Q.

	X	R	G	Q
Mean value	1	1	1	1
Standard deviation	0.1	0.25	0.1	0.4
Distribution type	Lognormal	Lognormal	Normal	Gumbel
Fractile	-	0.05	0.5	0.98
Characteristic value	-	0.647	1	2.037

loads are material independent. Thus, γ_G and γ_Q are fixed and the optimization is performed only subject to γ_m. The line with the circles in Figure 1 is representing the corresponding result ($\gamma_m = 1.33$).

The above example demonstrates the validity of the Eurocode 5 (2004) design safety concept for timber load-bearing elements under the assumption that the parameters given in Figure 1 represent the real situation with sufficient accuracy. In the following it will be discussed in which ways the actual load load-bearing behavior derivate from the assumptions in Table 1. It is demonstrated and quantified how the corresponding deviation affects the reliability of design situations and it is discussed how recent research results might integrated in the further developed issue of Eurocode 5 (2004).

3. PARTICULARITIES IN TIMBER MATERIAL MODELLING

3.1. *Different "material properties"*

Timber is a rather complex building material. Its properties are highly variable, spatially and in time. In structural engineering, material properties of timber are in general understood as the stress and stiffness related properties of standard test specimen under given (standard) loading and climate conditions and the timber density. Test configurations are prescribed in e.g. ISO 8375 (1985) and any statement about stress and stiffness related properties of structural timber is conditional to the corresponding test configuration. In general it is distinguished between the different loading modes and "material properties" are given corresponding to the loading direction relative to the main fiber direction of a beam shaped element (Figure 2).

The "material properties" have different statistical properties and when using the design criterion introduced before and applying the same partial safety factor γ_m, as it is practiced in the Eurocode, the reliability of the corresponding design solutions differ.

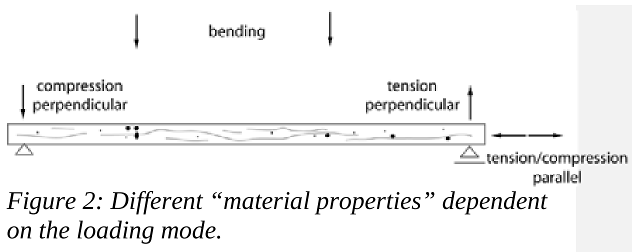

Figure 2: Different "material properties" dependent on the loading mode.

The influence of different "material properties" was investigated in Kohler and Fink (2012). There, the distribution functions and the associated variability for different types of "material properties" were chosen, as recommended in the Probabilistic Model Code JCSS (2006), see also Köhler (2006) for background information. The results are summarized in Table 2. The obtained scatter in partial safety factors suggests a rather differentiated treatment of the different design situations in future developments of design codes.

The most extreme deviation from the values proposed in the Eurocode $\gamma_m = 1.30$ is obtained for the load case tension perpendicular to the grain $\gamma_m = 3.05$. This also indicated that if a structural element for this load case is designed with the current safety factor of $\gamma_m = 1.30$, very low reliability indices, in the order of magnitude of 3.1 are obtained. However, the results concerning this particular load case have to be considered with spe-

Table 2: Calibrated partial safety factors for the resistance, for constant $\gamma_G = 1.35$ and $\gamma_Q = 1.50$ (from Kohler and Fink, 2012).

Ultimate limit state	γ_m
Bending strength	1.33
Tension strength parallel to the grain	1.40
Tension strength perp. to the grain	3.05
Compression strength parallel to the grain	1.24
Compression strength perp. to the grain	1.20
Shear strength	1.33

cial care. In fact the material capacity under this loading mode is specified by EN 338 (2010) with a nominal value that does not correspond to the 5%-fractile value taken from the statistical distribution that is derived from test data for the same loading mode. It is rather a value well below the 5%-fractile value. Furthermore, in best practice timber engineering design this loading mode at its limit is avoided due to the high sensibility to aspects that are not directly controlled in design, as e.g. moisture induced stresses and macro and micro cracks in the timber.

3.2. Timber as a graded material

Due to the special way timber material properties are ensured by means of grading in the production line, special considerations must be made when modeling their probabilistic characteristics. Previous work on this subject is reported in (e.g. Rouger, 1996; Pöhlmann and Rackwitz, 1981). Further assessment of the probabilistic modelling on the properties of graded timber material was presented in Faber et al. (2004); Sandomeer et al. (2008). In the latter references it is reported that the scatter of strength related material properties is highly sensitive to the grading procedure applied and to the properties of the original ungraded material. This observation is confirmed by a large experimental campaign that took place recently in Europe in connection to the Gradewood project. Here a large number of graded samples have been tested and a large between sample variability has been observed. Furthermore it has been shown that it is highly uncertain whether a sample that is graded to a specific grade actually meets the corresponding requirements in regard to minimum 5% fractile values of strength properties.

It is continued along the example introduced above, assuming that the grading accuracy directly affects the coefficient of variation of the timber bending capacity. The material partial safety factor are calibrated for different grading schemes that correspond to different coefficients of variation in the range from 0.2 – 0.4. The corresponding partial material safety factors rage between $\gamma_m = 1.2 - 1.65$ depending on the applied grading procedure. These results suggest a better differentiation of the grading procedure in future design codes. Alternatively, if no information about the accuracy of timber grading is utilized a larger coefficient of variation for representing the bending capacity should be used.

3.3. Non linear design equations

For common design equations a linear comparison of load effects and component resistance as in Equation (1) is not sufficient. One example is the design of slender columns where strength and stiffness properties and creep effects play an important role for assessing the stability. For the analysis of single members, standards generally give simplified calculation models that do not require a 2^{nd} order ultimate limit state analysis. However, for the analysis of more complex systems like unbraced frame structures, a 2^{nd} order structural analysis is more appropriate and accurate and an alternative design procedure is given e.g. in the Eurocode (2004). Compared to the simple design format as presented in Equation (1), the design equations for slender columns are more complex containing uncertain properties as strength, stiffness and load eccentricity in non-linear combination. The problem was addressed in Köhler et al. (2008) and quite uneven reliabilities for different column slenderness have been reported. Figure 3 the reliability index of design solutions with different slenderness-ratio are presented. The different colors correspond to different design frameworks; I. EN 1995-1-1 (2004), 2^{nd} order method with the stiffness considered as the mean modulus of elasticity;. II. DIN 1052 (2004), 2nd order method with the stiffness consid-

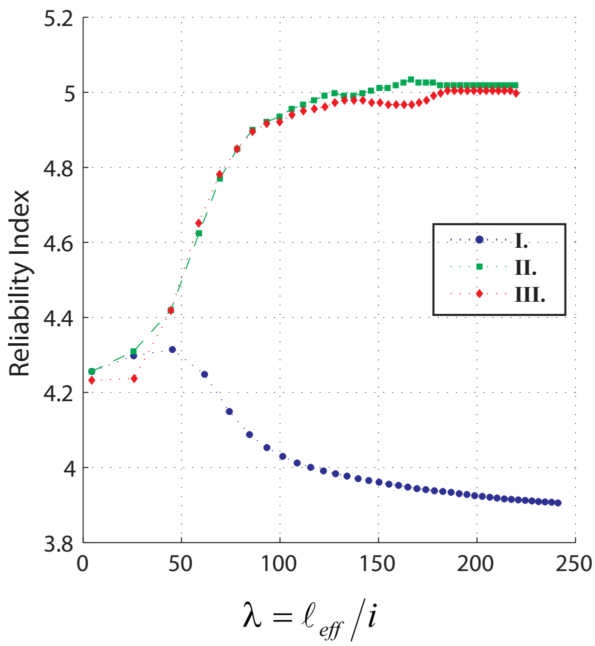

Figure 3: Reliability Index over slenderness for design solutions according to different design formats (Köhler et al., 2008).

ered as the 5% fractile of the modulus of elasticity, and; III. EN 1995-1-1 (2004) / DIN 1052 (2004) according to the so called simplified equivalent length approach. For more details compare Köhler et al. (2008).

In future code safety formats design strength and stiffness should be clibrated in order to obtain consistent reliability levels for different design situations.

3.4. *Duration of load effect and moisture induced stress*

The capacity of a timber structural element is highly dependent on the time duration of the load effect to which it is exposed to. E.g. the capacity of a bending beam continuously loaded is only 60% of that of a similar beam exposed to an instant load (Wood, 1947).

Timber is a hygroscopic material, i.e. it adsorbs and desorbs moisture from the surrounding air. Variations in moisture content in the surrounding air will, with a corresponding time lack, lead to variations in moisture content in the timber, this affects the mechanical properties of the timber but more importantly it will induce stresses due to shrinkage and swelling alongside the moisture gradients in the timber. These moisture induced stresses have been a matter of intensive discussion in the timber engineering community in the last years.

Both, duration of load effect and moisture induced stresses are highly relevant phenomena to take into account in structural design. They are also challenging phenomena since the underlying physical mechanisms are not fully understood and empirical evidence is scarce. However, in practical design, as in the Eurocode 5 (2004), the effect of moisture on the duration of load effect is considered with the joint modification factor k_{mod} which is given for different climate exposures in design codes. Values for this factor are prescribed in a matrix for three different so-called service classes, i.e. different climate scenarios, and five different load classes, i.e. load scenarios.

This format appears to be oversimplified and further research and enhancement of the level of detail in structural design should be developed.

3.5. *Volume and length effects*

One major topic that is continuously discussed within the research community is the appropriate representation of size effects on strength in solid timber. For most loading modes as tension parallel or perpendicular to grain, shear or bending, timber predominately presents brittle failure behaviour. A (perfect) brittle material is defined as a material that fails if a single particle fails (see e.g. Bolotin, 1969). The strength of the material is thus governed by the strength of the 'weakest' particle; therefore the model for ideal brittle materials is also called the weakest link model (Weibull, 1939). This model was applied to the different failure modes in timber, the model parameters have been calibrated based on experimental evidence on the different failure modes. A literature review can be found e.g. in Kohler et al. (2013). There it is concluded that the size effects in timber are better represented with a model that takes into account the multi scale variability of structural timber and a corresponding model framework is suggested.

In present code formats size effects are often not completely taken into account or neglected. This is particularly critical when large scale engineered timber sections are used in modern timber construction. In a revision of the codes this aspect should earn appropriate attention and current research results should be implemented.

3.6. *Joints*

For timber structures, the structural performance depends to a considerable part on the connections or joints between different timber structural members; joints can govern the overall strength, serviceability and fire resistance. Despite their importance timber joint design frameworks are not based on a consistent basis compared to the design regulations of timber structural components.

Explanations for this difference in progress of design provisions for members and joints can be found in the relative simplicity of characterizing mechanical behaviour of members, as compared to connections. A diversity of joints types is used in practice and these types have infinite variety in arrangement. This usually precludes the option of testing large numbers of replicas for a reliable quantification and verification of statistical and mechanical models. The main and most important group of joints corresponds to the joints with dowel type fasteners, i.e. joints with dowels, nails, screws and staples belong to this group.

Different failure modes can be observed for dowel type fastener joints and the modes are partly captured by a simple mechanical model based on the works of Johansen (1949); Meyer (1957). These models build the basis for the current European design framework for dowel type connectors in the Eurocode. However, different failure modes correspond to different failure behavior and consequences (brittle or ductile). In Köhler (2006) is has also been observed that model uncertainty and model bias for the different failure modes is significantly different. This is not considered in the current version of the European design standard and should be subject for further investigation.

3.7. *Consequence classes*

In the previous chapter it was mentioned that different failure modes in dowel type fastener joint lead to different magnitudes of consequences. This is in principle true for all failure modes in timber structure. In Chapter 3.1 different failure modes of timber components have been compared to the same target reliability, implying that the consequences for all failure modes are classified uniformly. However, if a failure scenario for tension or bending failure is visualized and compared with a typical failure scenario for compression perpendicular to the grain, it might be agreed that the consequences are quite different and correspondingly the target reliability should be defined separately for the different cases.

4. CONCLUSIONS

Timber will play an important role in the future developments towards a more sustainable building sector. However, many stakeholders are still skeptical when it comes to the technological maturity of the material, especially compared to concrete and steel. The structural design regulations in general can be seen not only as the main interface connecting the state of knowledge in the engineering research community with the implementation of the real build environment; design standards are also the precondition for the implementation of building material on a high technological level.

In the present paper the major challenges for the future development of timber design standards have been highlighted from a European perspective; i.e. taking the Eurocodes as references. The challenges are hereby related to both, the further development of the knowledge basis for the behavior of timber in structures and the implementation of this knowledge into practicable rules in the future standards.

5. REFERENCES

Bolotin, V. V. (1969). *Statistical Methods in Structural Mechanics*. Holden-Day Series in Mathematical Physics.

DIN 1052 (2004). "Bemessungsregeln für Holzkonstruktionen. Normenausschuss Bauwesen (NABau) im DIN (Deutsches Institut für Normung e.V.), Berlin, Germany.

EN 1990 (2002). "Eurocode 0: Basis of structural design. European Committee for Standardization, Brussels, Belgium.

EN 1995-1-1 (2004). "Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings; German version. European Committee for Standardization, Brussels, Belgium.

EN 338 (2010). "Structural timber – Strength classes; German version. European Committee for Standardization, Brussels, Belgium.

Faber, M. H., Köhler, J., and Sørensen, J. D. (2004). "Probabilistic modelling of graded timber material properties." *Structural safety*, 26(3), 295–309.

Faber, M. H. and Sørensen, J. D. (2003). "Reliability based code calibration – The JCSS approach." *Applications of statistics and probability in civil engineering : proceedings of the 9th International Conference on Applications of Statistics and Probability in Civil Engineering, San Francisco, USA, July 6–9*, 927–935.

ISO 8375 (1985). "Solid timber in structural sizes: Determination of some physical and mechanical properties. International Standards Organization, Geneva, Switzerland.

JCSS (2001). "Probabilistic Model Code Part I - Basis of Design.

JCSS (2006). "Probabilistic Model Code Part III - Resistance Models (3.05 Timber).

Johansen, K. (1949). "Theory of timber connections." *International Association of Bridge and Structural Engineering, Publication No. 9, pp. 249-262, Bern, Switzerland*.

Köhler, J. (2006). "Reliability of timber structures." Ph.D. thesis, ETH Zurich, Zurich, Switzerland, ETH Zurich, Zurich, Switzerland.

Köhler, J., Andrea, F., and René, S. (2008). "On the role of stiffness properties for ultimate limit state design of slender columns." *Proceedings of the 41th Meeting, International Council for Research and Innovation in Building and Construction, Working Commission W18 - Timber Structures, St. Andrews, Canada, CIB-W18, Paper No. 41-1-1*.

Kohler, J., Brandner, R., Thiel, A., and Schickhofer, G. (2013). "Probabilistic characterisation of the length effect for parallel to the grain tensile strength of central european spruce." *Engineering Structures*, 56, 691–697.

Kohler, J. and Fink, G. (2012). "Reliability Based Code Calibration of a Typical Eurocode 5 Design Equations." *World Conference on Timber Engineering 2012 (WCTE 2012) : The Future of Timber Engineering : Auckland, New Zealand : 15-19 July 2012. Volume 4*, P. Quenneville, ed., Red Hook, NY, Curran Associates, Inc., 99–103.

Meyer, A. (1957). "Die Tragfähigkeit von Nagelverbindungen bei Statischer Belastung." *Holz als Roh- und Werkstoff*, 15(2), 96–109.

Pöhlmann, S. and Rackwitz, R. (1981). *Zur Verteilungsfunktion der Festigkeitseigenschaften bei kontinuierlich durchgeführter Sortierung*. Materialprüfung 23, Hanser, Munich, Germany.

Rouger, F. (1996). "Application of a modified statistical segmentation method to timber machine strength grading." *Wood and Fibre Science*, 28(4).

Sandomeer, M. K., Köhler, J., and Faber, M. H. (2008). "Probabilistic output control for structural timber - modelling approach." *Proceedings of the 41th Meeting, International Council for Research and Innovation in Building and Construction, Working Commission W18 - Timber Structures, St. Andrews, Canada, CIB-W18, Paper No. 41-5-1*.

Weibull, W. (1939). *A statistical theory of strength of materials*. Nr. 151, Royal Swedish Institute for Engineering Research.

Wood, L. (1947). "Behaviour of wood under continued loading." *Eng. News-Record*, 139(24), 108–111.